AN INVESTIGATION INTO CURRENT PROCEDURES FOR ESTIMATING HEAVE POTENTIAL IN CLAYS

by

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DECLARATION

I, the undersigned, declare that the dissertation hereby submitted by me for the degree *Master 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 dissertation in favour of the Central University of Technology, Free State.

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ABSTRACT

An investigation into current procedures for estimating heave potential in clays

Several low-cost housing developments in South Africa are suffering major structural failures due to heaving clays. Despite geotechnical investigations and various precautionary measures, this remains an on-going trend.

The aim of this study was to review the current procedures used in South Africa to estimate heave potential in view of either improving the current procedures or suggesting alternatives. To this end, the research question was as follows: *Are the current procedures used in South Africa to estimate heave potential acceptable?* In this context, the most popular procedure used in South Africa, van der Merwe's method, was broken into parts and studied.

The research question was answered in the variance of laboratory results obtained from seven leading commercial laboratories which proved that reliable input parameters to van der Merwe's empirical method are not obtainable. Typically the Atterberg limits and clay fraction results varied significantly producing heave potential classifications that do not accurately reflect the soil characteristics of the samples studied.

On this basis, an investigation into methods of estimating heave potential, which are not considered in current South African codes of practice, were studied in order to provide a foundation for future research. A weighed system is proposed to judge the heave potential of soils using various prediction models as a foundation for future research.

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LIST OF ABBREVIATIONS

ASTM	American Society for Testing and Materials
BS	British Standards
CBR	California Bearing Ratio
COLE	Coefficient of linear extensibility
CSIR	Council for Scientific and Industrial Research
CUT	Central University of Technology
CVS	Constant volume swell
DCP	Dynamic Cone Penetration
FSI	Free swell index
FSR	Free swell ratio
GMBV	Gross Methylene Blue Value
ISO:17025	International Organization for Standardizing Standard 17025
LAB	Laboratory
MBV	Methylene blue value
nr	number
pp	page
SANAS	South African National Accreditation System
SANS	South African National Standards
SWCC	Soil-water characteristic curve
TMH	Technical Methods for Highways
USA	United States of America
USCS	Unified Soils Classification System

LIST OF SYMBOLS

$	Dollar (monetary) – Typically American Dollar
° C	degrees Celcius
µm	micron meter, unit of length, 1 x 10^{-6}meters
Å	Ångstrom unit of length, 1 x 10^{-10}meters
A_C	Activity
c	Clay content (smaller than 0.002mm)
cm^3	cubic centimetre, unit of volume, 1 x 10^{-6} cubic meters
g	gram, unit of weight
I_P (%)	Plasticity Index
K (%)	Swell potential after Savage (2007)
kPa	kilo Pascal, unit of pressure, 1 x 10^3 Pa
m	meter, unit of length
m^2/g	Square meters per gram, specific surface area measurement
mm	millimetres, unit of length, 1 x 10^{-3}meters
N/mm^2	Newton per square millimetre, a unit of pressure
$P_{0.002}$ (%)	Percentage passing 0.002mm, percentage smaller than 0.002mm
$P_{0.425}$	Percentage passing 0.425mm, percentage smaller than 0.425mm
p_s	Swelling pressure
psi	Pounds per square inch (a unit of pressure measurement)
q_i	Initial surcharge
R	Plasticity ratio (W_L/W_P)
S%	Expected swell
T_S	Surface tension
u_0	Initial pore pressure
u_f	Final pore pressure
V_d (cm^3)	Volume of distilled water in cubic centimetres
V_k (cm^3)	Volume of kerosene in cubic centimetres
W_L (%)	Liquid limit
W_{LS} (%)	Linear Shrinkage

W_P (%)	Plastic limit
W_{SL} (%)	Shrinkage limit
γ_d	Dry unit weight (kN/m²)
ΔH	Total heave
ρ_d	Dry density (kg/m³)
σ_v	Vertical stress
ψ_i	Initial suction

CHAPTER 1: INTRODUCTION

1.1 Problem Statement

Expansive soils are widely distributed in arid and semi-arid regions of the world, including South Africa. A study by Puppala and Cerato (2009) attributed thirteen billion dollars (not adjusted for inflation) of annual damage to infrastructure in the United States of America due to expansive soils. A third of the reported damage was recorded in residential and commercial structures with the remainder to roads, bridges and dams (Wray and Meyer, 2004). The financial loss to property exceeds those of earthquakes, floods, hurricanes and tornadoes combined (Jones and Holtz, 1973; Handy, 1995). A similar trend has been identified in the Free State Province and the Northern Cape Province with a large number of structural failures recorded at low cost housing developments due to expansive soils.

In Africa, there exists a widespread problem of providing economic housing for lower income communities. In South Africa the government attempts 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.

Expansive soils swell upon wetting and shrink upon drying due to seasonal changes (Chen, 1975; Groenevelt and Grant, 2004; Erzin and Erol, 2007; Zhan, Chen and Ng, 2007). Hydrophilic minerals such as montmorillonite and illite typically influence volume change in expansive soils with the change in water content.

Once the swelling potential of expansive soils is restrained, a counterforce is generated. This counterforce is typically referred to as swelling pressure. The swelling pressure will be exerted on infrastructure such as housing developments or roads and may lead to extensive damage if countermeasures are not taken (Fredlund, Xing, Fredlund and Barbour, 1995). Reliable estimation of the swelling pressure as well as the ground heave of expansive soils is important in order to design the infrastructure to accommodate expansive soils and alleviate the damages (Hongyu, 2015).

Nelson and Miller’s standard text on expansive soils (Nelson and Miller, 1992) considers three general approaches to heave prediction: the use of the oedometer, the use of shrinkage and or suction curves and the use of empirical methods. Oedometers are not considered to be a viable option for the determination of total estimated heave of active clays for low-cost housing projects due to the cost and time implications thereto. The current procedures typically utilised with low cost housing developments rely on van der Merwe’s empirical method (1964) which determines a heave potential classification based on the Atterberg limits and clay fraction of soils. The heave classification, layer thickness and total non-expansive overburden are then used to determine a total heave prediction per soil profile.

Despite van der Merwe’s method and other empirical methods being utilised to estimate heave potential, the structural failures associated with expansive clays are continually recorded. This prompted the research hypothesis.

1.2 Hypothesis

The current empirical procedures used to determine heave potential for clays are not acceptable.

1.3 Methodology Flow-chart

The following flow-chart (Figure 1.1) indicates the methodology followed towards completing the study and compiling the final dissertation thereon:

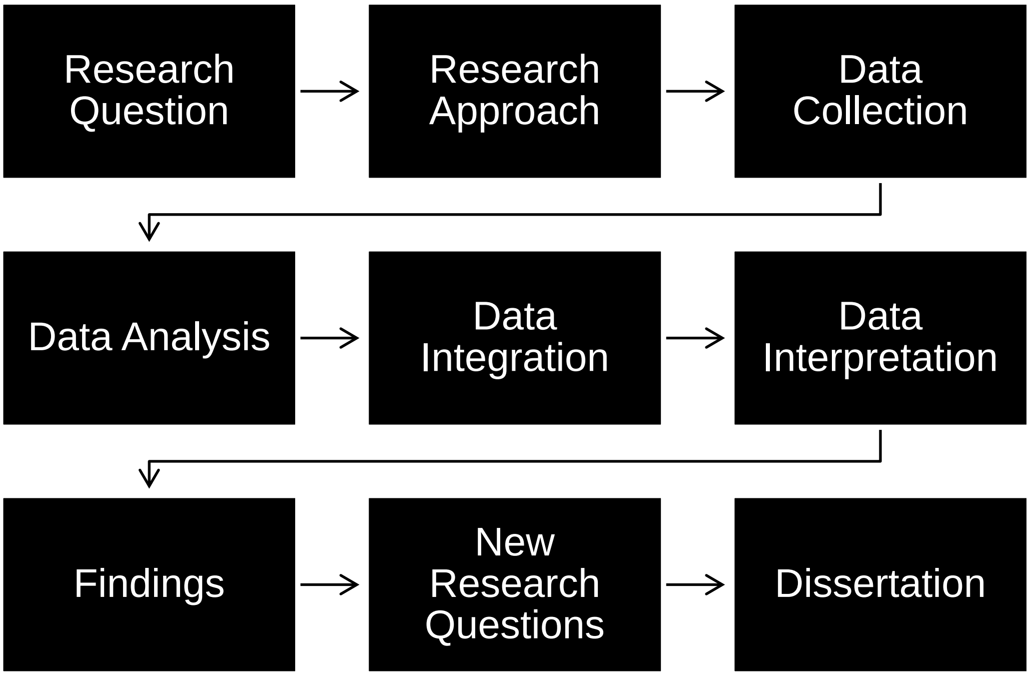


Figure 1.1: Research methodology

Based on several geotechnical investigations and site visits by the author, it became evident that structural failures associated with heaving clays in the Free State Province were increasing with heave potential estimations frequently underestimating the heave potential of *in-situ* soils. This prompts the question: *Are the current empirical procedures used to determine heave potential for clays acceptable?*

A literature study was undertaken (Chapter 2) to determine the research approach and to better define and understand the research problem.

A review of the current procedures used to determine the heave potential of expansive soils was undertaken in Chapter 3.

Soil samples were taken of various clayey soils encountered in the Free State Province and the Northern Cape Province and analysed in a commercial soils laboratory to determine which of the samples are considered to be expansive.

Two expansive materials were split into seven smaller representative samples and sent to seven of the premier commercial laboratories in South Africa (Chapter 4).

The data collected were analysed and integrated into the prediction model initially published by van der Merwe in 1964, improved by van der Merwe in 1976 and adjusted by Williams and Donaldson (1980) and Savage (2007). Based on the data interpretation and findings, new procedures (Chapter 5) were proposed in trying to identify viable methods to determine heave potential in line with the cost and time requirements of low cost housing developments.

The findings of this dissertation are discussed in Chapter 6.

CHAPTER 2: LITERATURE REVIEW

2.1 Expansive Clays

This chapter will focus on expansive clays, their origin, composition and behaviour.

2.1.1 Introduction

Expansive soils have received the attention of many researchers over the years. The behaviour of expansive clays relating to damage caused to buildings, roads and retaining walls have been studied (Mansour, 2011). Expansive soils typically include clays and fine silts which shrink with a decrease in vapour and swell as their moisture contents increases (Chen, 1975).

The amount by which the soil can shrink or expand is determined by the water content in the near-surface zone. The activities up to a depth of three meters are usually significant, unless this zone is extended by the presence of tree roots (Driscoll, 1983; Biddle, 1998).

This chapter aims to provide an understanding of the expansive nature of clays and to give background on the methods used to predict the expansive behaviour of soils.

2.1.2 Origin

According to Donaldson (1969), the parent materials associated with expansive soils are classified into two groups:

a) The basic igneous rocks, where feldspar and pyroxene minerals decompose to form montmorillonite and other secondary minerals; and
b) The sedimentary rocks that contain montmorillonite as one of its elements that break down physically to form expansive soils.

Snethen, Townsend, Johnson, Patrick and Vedros (1975) theorised that the origin and distribution of expansive soils are a function of geological history, sedimentation and present local climatic conditions. Weathering, diagenetic alteration of pre-existing minerals and hydrothermal alteration were the conditions, either individually or in combination, regarded to be the sources of the formation of expansive soils.

The weathering process is considered to be the most important source of expansive soils and is considered following three different mechanisms: inheritance, neo-formation and transformation (Eberl, 1984). These reactions are characterised by ion exchange with the surrounding environment and /or layer transformation in which the structure of octahedral, tetrahedral or fixed interlayer cations are modified (Mitchell and Soga, 2005).

2.1.3 Clay minerals

Typically soil classification systems define *clay particles* as having an effective diameter of two microns (0.002mm) or less (Chen, 1975). Particle size alone, however, does not define clay minerals, with mineralogical composition considered to probably be the most important factor (Peck, Hanson and Thornburn, 1974). Materials are considered to be in colloidal state when the electrical forces acting on the surface are much greater than the gravitational force acting on them. The colloid consists primarily of minerals weathered from parent rock or sedimentary rocks as described in section 2.1.2*.* The methods of engineering classification typically recognise the presence of these minerals through their physical or chemical properties (Olson, Krosley, Nelson, Chabrillat, Goetz and Noe, 2000). The three prominent groups of clay in decreasing order of expansiveness are smectite, illite and kaolinite, illustrated in Figure 2.1.

Clay minerals are mainly produced from the weathering of micas and feldspars, forming part of a group of complex alumino-silicates of iron, magnesium and potassium. These are known as *layer-lattice minerals.* They are very small in size and very flaky in shape. They often have very large specific surfaces (Whitlow, 2001).

Snethen *et al*. (1975) classified the clay fraction as follows:

a) Two-layer clays consist of one silica tetrahedral layer bonded to one aluminium octahedral layer such as kaolinite, indicated in Figure 2.1. 1:1 clays – typically not expansive.

b) Three-layer clays have one octahedral layer bonded between two tetrahedral layers such as illite and montmorillonite, indicated in Figure 2.1. 2:1 clays – typically expansive.

c) Mixed-layer clays consist of interstratifications of the two- and three-layer clay minerals.

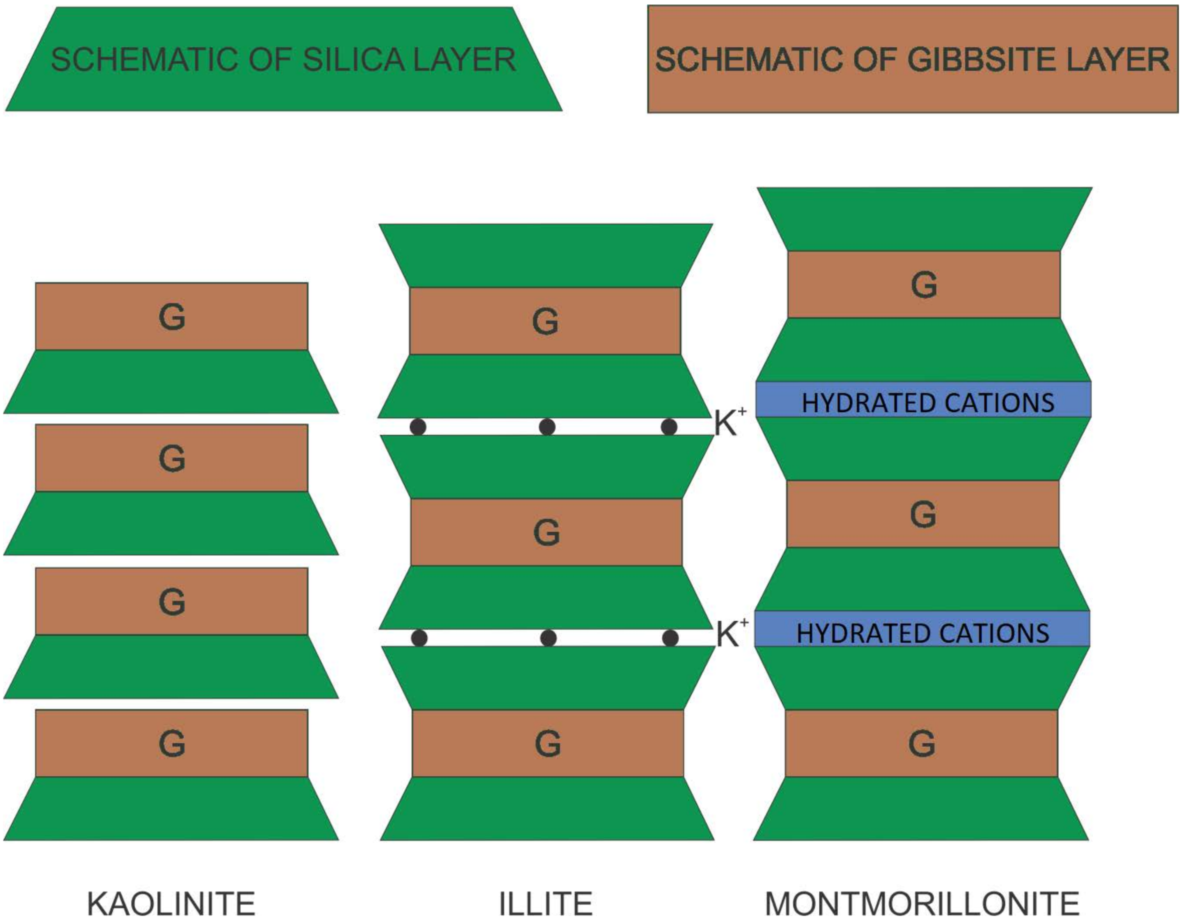


Figure 2.1: Two and three layer silicate clays (Adapted from Whitlow, 2001)

Typically the basic structural units of prevalent clays are a silicon-oxygen tetrahedron and an aluminium-hydroxyl octahedron, as indicated in Figure 2.2. Due to valency imbalances in both the tetrahedron and octahedron units, both have net negative charges. Sheet structures are formed by ion sharing. The tetrahedron units share oxygen ions to form *silica* sheets while the octahedron units combine through sharing hydroxyl ions to form *gibbsite* sheets (Craig, 2004).

Montmorillonite, illite and kaolinite are considered to be the three most important and prevalent clay minerals, all of which are crystalline hydrous aluminosilicates (Donaldson, 1969; Chen, 1975).

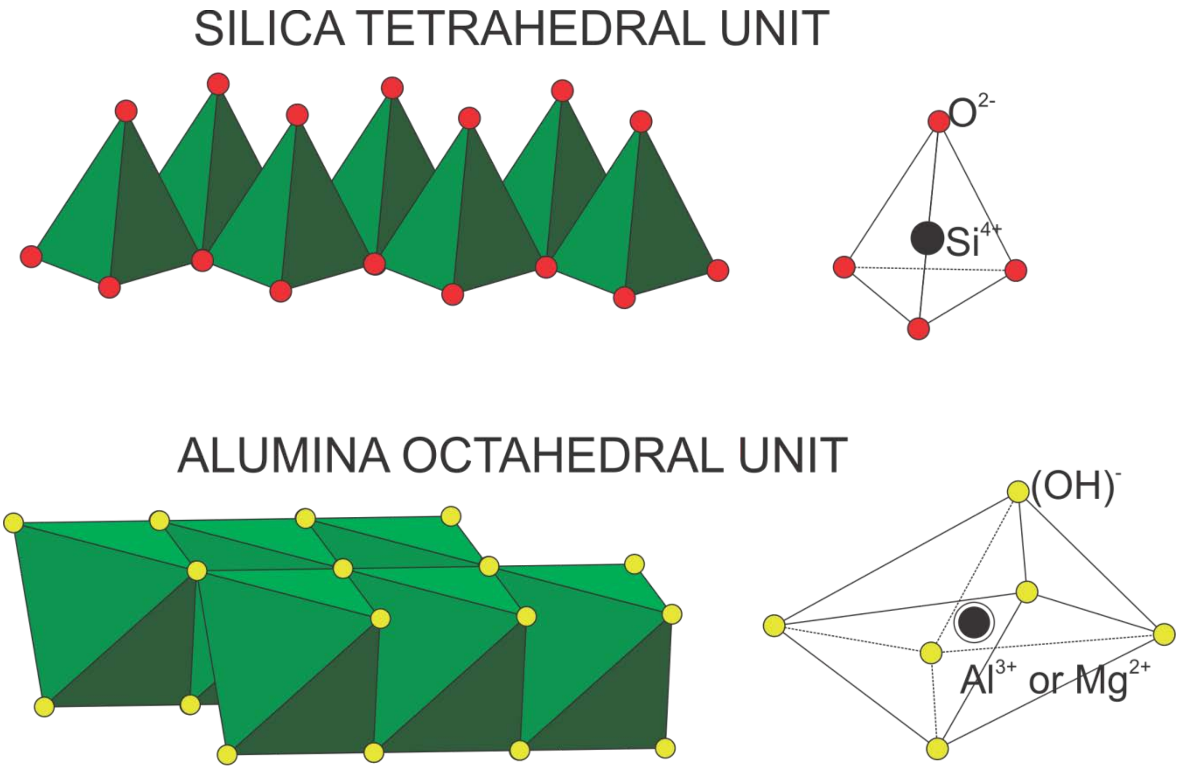


Figure 2.2: Unit elements of clay minerals and typical clay structures (Adapted from Holtz and Kovacs, 1981 and Whitlow, 2001)

Expansive clays show a marked increase or decrease in volume as the water content is increased or decreased (Whitlow, 2001). To understand this nature of clay, one must understand the structure of clay particles. Illite and montmorillonite, both considered to be expansive, have a similar structure consisting of a sheet of gibbsite between two sheets of silica. In illite, there is partial substitution of silicon by aluminium in the silica sheet. In montmorillonite, there is partial substitution of aluminium by magnesium and iron in the gibbsite sheet. The combined sheets are bonded weakly due to non-exchangeable potassium ions for Illite with water molecules transporting exchangeable cations other than potassium, resulting in very weak bonding, for montmorillonite (Craig, 2004). Considerable swelling (heave) of montmorillonite can occur due to additional hydrated cations being absorbed between the combined sheets (Craig, 2004), while considerable settlement may occur if the water adsorbed between the combined sheets is squeezed out due to loading as a result of construction.

Extensive shrinking may occur due to the evaporation of moisture. A continuous electro-static attraction exists between the combined sheets and cations. There is varying suction between the atmosphere and the water which hydrates the cations (Craig, 2004); therefore there will be a variation in moisture content adsorbed between the sheets depending on the existing suction.

Table 2.1: Characteristics of some clay minerals (After Mitchell, 1976)

Mineral Group	Basal Spacing (Å)	Particle Features	Interlayer Bonding	Specific Surface (m²/g)	Atterberg Limits			Activity = PI/clay content
					W_L (%)	W_P (%)	W_{SL} (%)	
Kaolinite	14.4	Thick, stiff 6-sided flakes 0.1 to 4 x 0.05 to 2µm	Strong hydrogen bonds	10-20	30-100	25-40	25-29	0.38
Illite	10.0	Thin stacked plates 0.003 to 0.1 x 1.0 to 10µm	Strong potassium bonds	65-100	60-120	35-60	15-17	0.9
Montmorillonite	9.6	Thin, filmy flakes >10Å x 1.0 to 10µm	Very weak Van der Waal's bonds	700-840	100-900	50-100	8.5-15	7.2

2.1.4 Classification and identification

Identification of potential expansive or shrinking soils is vitally important in order to select the appropriate foundation type (Hamilton, 1977). Today, many test methods have been developed or modified for estimating differential settlement in expansive soils, whether by direct or indirect measures. Direct methods provide actual measurements of swelling or shrinkage which can then be related to estimate differential settlement on the study area where indirect methods involve the use of soil properties and classification schemes to estimate differential settlement (Nelson and Miller, 1992).

Three methods may be utilised to identify expansive soils: (Snethen *et al*., 1975)

a) Indirect methods in which relative intrinsic properties of the soil are measured that may give an indication of the volume change potential and magnitude. These may include soil composition, particle size distribution, physiochemical, physical and index properties.

b) Direct methods may be applied to measure volume change in a sample in an oedometer testing apparatus. These tests are typically grouped into swell or swell pressure tests depending on the parameters required by the geotechnical engineer. A measurement of the swell or shrink of the sample may then be used to classify the soil according to potential expansiveness. Suction tests are also considered to be direct measurement tests.

c) Combination methods in which direct measurements and indirect properties are correlated or statistically evaluated to give a better estimation on the probable severity.

2.2 Soil Properties and Environmental Conditions that Influence Shrink-Swell Potential

Nelson and Miller (1992) summarised the soil properties that influence shrink-swell potential as follows: (Table 2.2 and Table 2.3)

Table 2.2: Soil properties that influence shrink-swell potential (after Nelson and Miller, 1992)

Factor	**Description**	**References** (as cited by Nelson and Miller, 1992)
Clay mineralogy	Clay minerals which typically cause soil volume changes are *montmorillonites, vermiculites,* and some *mixed layer minerals.* Illites and Kaolinites are infrequently expansive, but can cause volume changes when particle sizes are extremely fine (less than a few tenths of a micron).	Grim (1968); Mitchell and Raad (1973); Mitchell (1976); Snethen, Johnson and Patrick (1977).
Soil water chemistry	Swelling is repressed by increased cation concentration and increased cation valence. For example, Mg^{2+} cations in the soil water would result in less swelling than Na^{+} cations.	Mitchell (1976)
Soil suction	Soil suction is an independent effective stress variable, represented by the negative pore pressure in unsaturated soils. Soil suction is related to saturation, gravity, pore size and shape, surface tension and electrical and chemical characteristics of the soil particles and water.	Aitchison (1965); Olsen and Langfelder (1965); Johnson (1973); Fredlund and Morgenstern (1977); Snethen and Johnson (1980).
Plasticity	In general, soils that exhibit plastic behavior over wide ranges of moisture content and that have high liquid limits have greater potential for swelling and shrinking. Plasticity is an *indicator* of swell potential.	Nelson and Miller (1992)
Soil structure and fabric	Flocculated clays tend to be more expansive than dispersed clays. Cemented particles reduce swell. Fabric and structure are altered by compaction at higher water content or remolding. Kneading compaction has been shown to create	Seed, Mitchell and Chan (1962); Johnson and Snethen (1978)

	dispersed structures with lower swell potential than soils statically compacted at lower water contents.	
Dry Density	Higher densities usually indicate closer particle spacings, which may mean greater repulsive forces between particles and larger swelling potential.	Komornik, Wiseman and Ben-Yaacob (1969); Chen (1973)

Table 2.3: Environmental conditions that influence shrink-swell potential (after Nelson and Miller, 1992)

Factor	**Description**	**References** (as cited by Nelson and Miller, 1992)
Initial Moisture condition	A desiccated expansive soil will have a higher affinity for water, or higher suction, than the same soil at higher water content, lower suction. Conversely, a wet soil profile will lose water more readily on exposure to drying influences, and shrink more than a relatively dry initial profile. The initial soil suction must be considered in conjunction with the expected range of final suction conditions.	Nelson and Miller (1992)
Moisture variations	Changes in moisture in the active zone near the upper part of the profile primarily define heave. It is in those layers that the widest variation in moisture and volume change will occur.	Johnson (1969)
a.) Climate	The amount and variation of precipitation and evapotranspiration greatly influence the moisture availability and depth of seasonal moisture fluctuation. The greatest seasonal heave occurs in semi-arid climates that have pronounced short wet periods.	Holland and Lawrence (1980)

b.) Groundwater	Shallow water tables provide a source of moisture and fluctuating water tables contribute to moisture change.	Nelson and Miller (1992)
c.) Drainage and manmade water sources	Surface drainage features, such as ponding around a poorly graded house foundation, provide sources of water at the surface; leaky plumbing can give the soil access to water at greater depth.	Donaldson (1965); Krazynski (1980)
d.) Vegetation	Trees, shrubs, and grasses deplete moisture from the soil through transpiration, and cause the soil to be differentially wetted in areas of varying vegetation.	Buckley (1974)
e.) Permeability	Soils with higher permeabilities, particularly due to fissures and cracks in the field soil mass, allow faster migration of water and promote faster rates of swell.	De Bruijn (1965); Wise and Hudson (1971)
f.) Temperature	Increasing temperatures cause moisture to diffuse to cooler areas beneath pavements and buildings.	Hamilton (1969); Johnson and Stroman (1976)
Stress conditions a.) Stress History	An overconsolidated soil is more expansive than the same soil at the same void ratio, but normally consolidated. Swell pressures can increase on aging of compacted clays, but amount of swell under light loading has been shown to be unaffected by aging. Repeated wetting and drying tend to reduce swell in laboratory samples, but after a certain number of wetting-drying cycles, swell is unaffected.	Kassiff and Baker (1971); Mitchell (1976)
b.) *In-situ* conditions	The initial stress state in a soil must be estimated in order to evaluate the probable consequences of loading the soil mass and/or altering the moisture environment therein. The initial effective stresses can be roughly determined	Nelson and Miller (1992)

	through sampling and testing in a laboratory, or by making *in-situ* measurements and observations.	
c.) Loading	Magnitude of surcharge load determines the amount of volume change that will occur for a given moisture content and density. An externally applied load acts to balance inter-particle repulsive forces and reduces swell.	Holtz (1959)
d.) Soil Profile	The thickness and location of potentially expansive layers in the profile considerably influence potential movement. Gratest movement will occur in profiles that have expansive clays extending from the surface to depths below the active zone. Less movement will occur if expansive soil is overlain by non-expansive material or overlies bedrock at a shallow depth	Holland and Lawrence (1980)

2.3 Indirect Methods to Identify Expansive Soils

2.3.1 Index properties and Activity

Atterberg limit states were formally developed in 1948. A. Atterberg had earlier conceptualised that clay soil can exist in four distinct states of consistency depending on its moisture content. The boundaries of the distinct states are liquid limit, plastic limit and shrinkage limit in order of moisture content (Nelson and Miller, 1992). By subtracting the plastic limit from the liquid limit of a soil sample (equation 2.1), the plasticity index is determined which is considered to be one of the most important indicators in classifying expansive clays (Nelson and Miller, 1992).

Skempton (1953) defined Activity (equation 2.2) as the ratio of plasticity index to the clay fraction and identified three classes of clay according to activities: *inactive* for activities below 0.75; *normal* for activities between 0.75 and 1.25; and *active* for activities in excess of 1.25 (Skempton, 1953; cited by Nelson and Miller, 1992).

$$\boldsymbol{I_P = W_L - W_P} \tag{2.1}$$

Where:

$\boldsymbol{I_P}$ denotes plasticity index (often denoted as PI)
$\boldsymbol{W_L}$ denotes liquid limit
$\boldsymbol{W_P}$ denotes plastic limit

$$A_C = \frac{I_P}{P_{0.002}} \tag{2.2}$$

Where:

$\boldsymbol{A_C}$ denotes Activity
$\boldsymbol{I_P}$ denotes plasticity index
$\boldsymbol{P_{0.002}}$ denotes the percentage of material smaller than 0.002mm (the clay fraction)

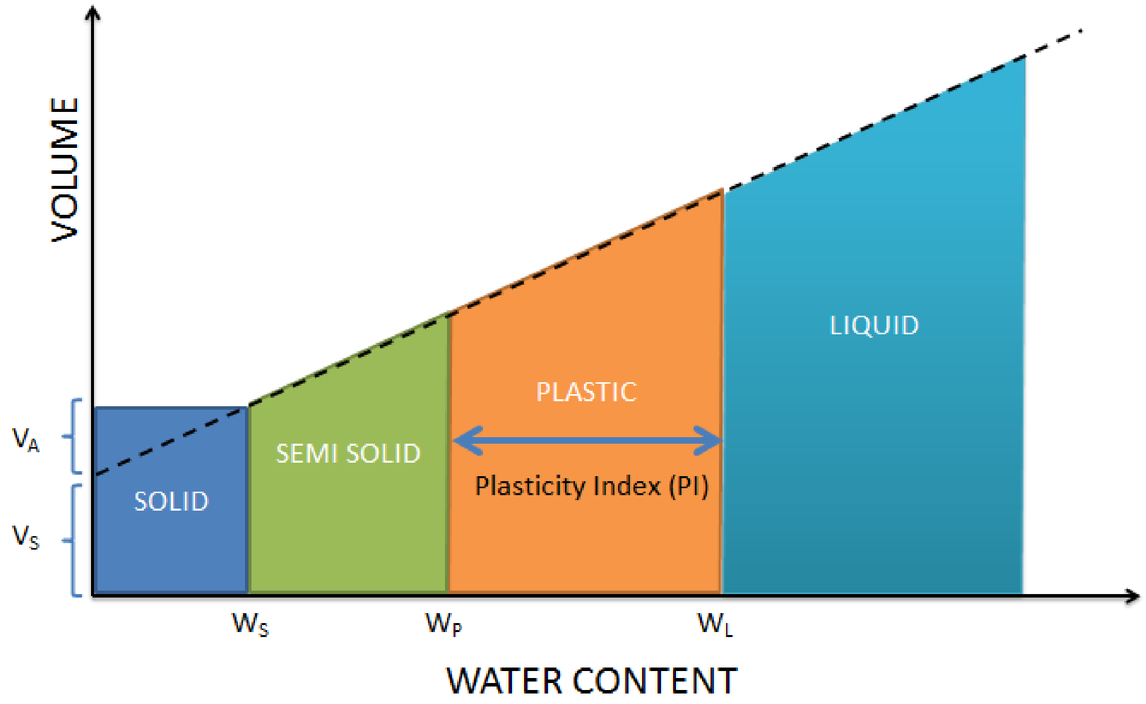


Figure 2.3: Atterberg limit states of soil (adapted from Casagrande, 1948)

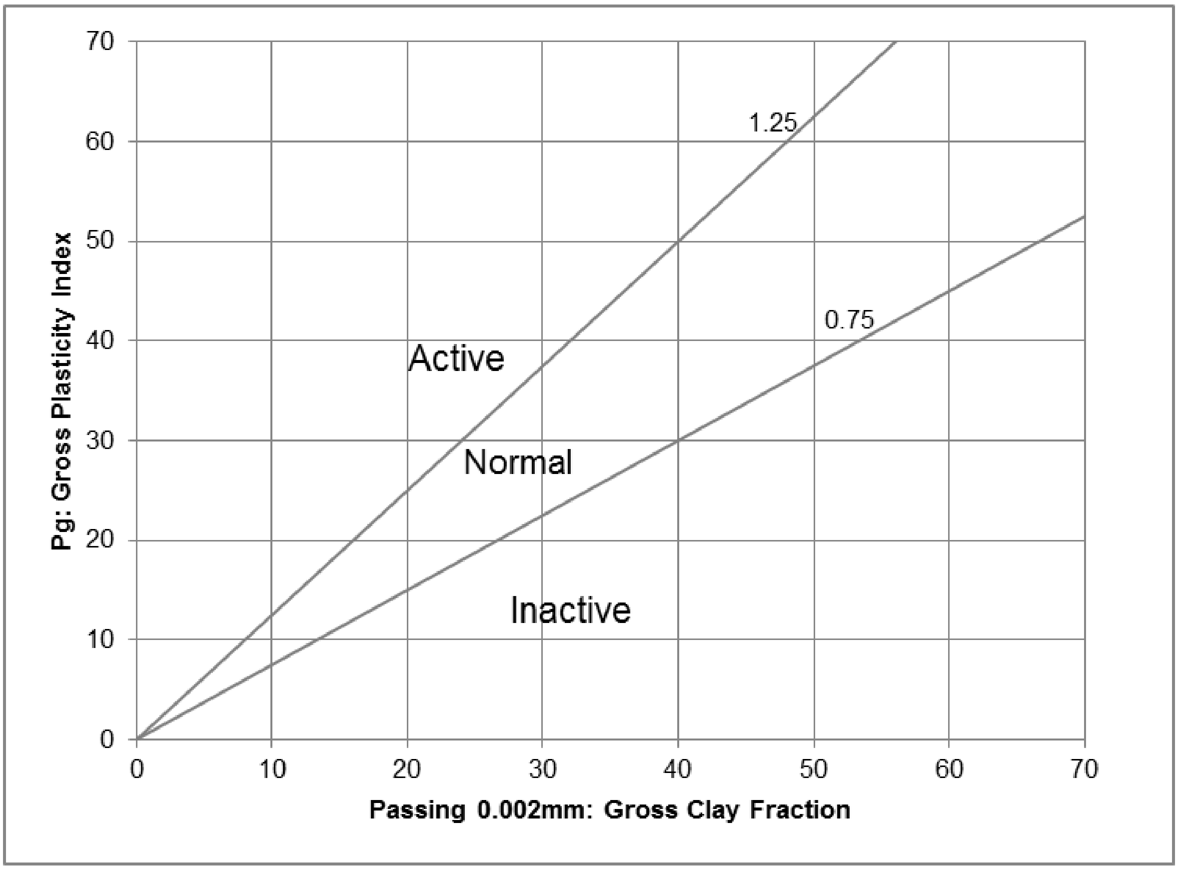


Figure 2.4: Activity chart (adapted from Skempton, 1953)

Casagrande's plasticity chart (Casagrande, 1948; cited by Holtz and Kovacs, 1981) is often used as an indicator of expansiveness potential, and as a prediction model on the mineral composition of clayey materials (Holtz and Kovacs, 1981). Figure 2.5, taken from Holtz and Kovacs (1981), indicates that material nearing the U-line of high to extremely high plasticity is typically considered to be montmorillonite, with clays just above the A-line considered being illite and materials below the A-line considered to be kaolinite. Expansive potential is then based on the estimated mineralogy of the soil.

Although Atterberg limits have the advantage of being a relatively simple, cheap and commercially available test, they do not give a direct measurement of heave potential. It may give an indication of how much water may be absorbed into the clay matrices by mechanical mixing, but gives no measure of the suction (kPa) generated by the soils that may give an estimation of the swelling pressure generated which causes heave.

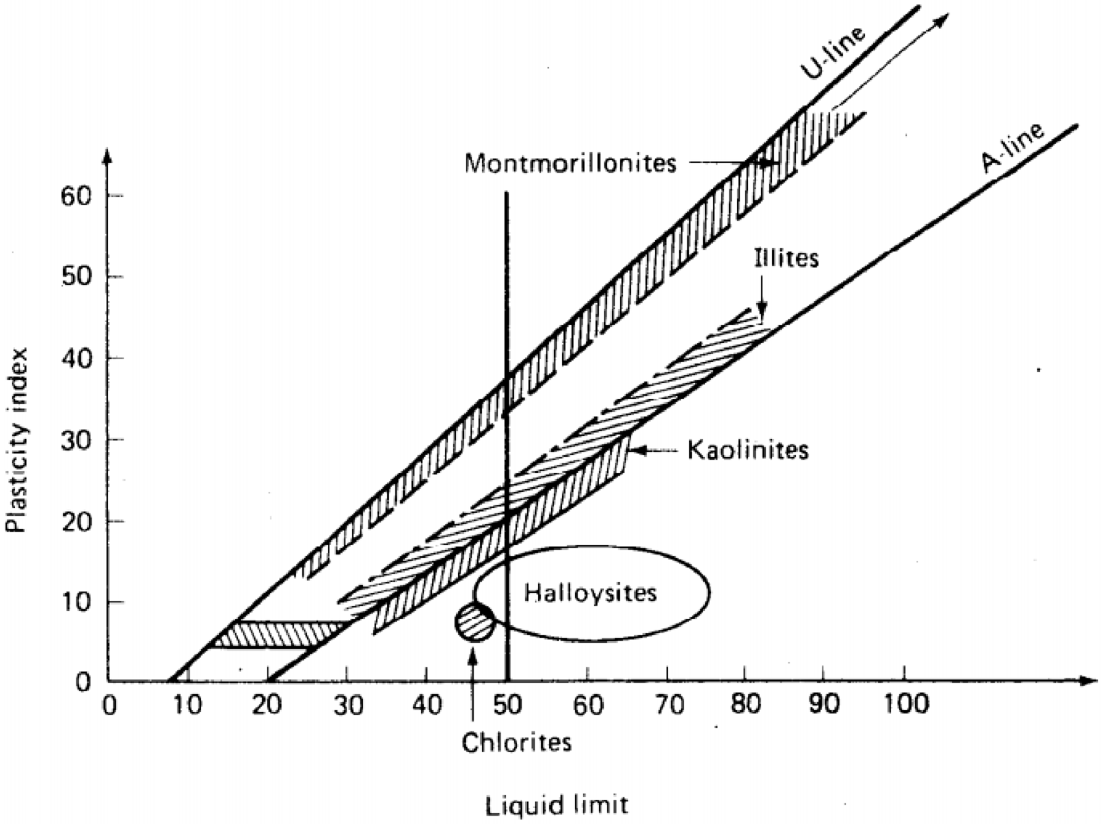


Figure 2.5: Location of common clay minerals on Casagrande's plasticity chart (developed from Casagrande, 1948, and data in Mitchell, 1976. Taken from Holtz and Kovacs, 1981 pp89)

Several classification systems to identify expansive behaviour of soil are based on index properties. In the early 1970's, the Federal Housing Administration of America (FHA, 1973) classified the degree of expansiveness based on its index properties. The plasticity index and liquid limit values were used to differentiate between "non-expansive", "marginal", "moderately expansive", "highly expansive" and "expansive claystone". This classification is summarised in Table 2.4:

Table 2.4: FHA/HUD Classification of expansive soils (FHA, 1973)

Classification	**Plasticity Index**	**Liquid Limit**	**Soil Group**
Non-expansive	0 – 6	0 – 25	A
Marginal	6 – 10	25 – 30	B
Moderately expansive	10 – 25	30 – 50	C
Highly expansive	25 – 50	50 – 70	D
Expansive claystone	>50	>70	E

Chen (1975) proposed the following classification system (Table 2.5):

Table 2.5: Classification of expansive soils after Chen (1975)

Degree of expansion	**Liquid Limit**	**Plasticity Index**
Low	<30	0 – 15
Medium	30 – 40	10 – 35
High	40 – 60	20 – 55
Very High	>60	>35

Snethen *et al.* (1977) proposed a classification that included the liquid limit and plasticity index which would yield an indication of natural suction and potential swell. The classification system is summarised in Table 2.6.

Table 2.6: Classification of expansive soils after Snethen *et al.* (1977)

Potential swell classification	**Liquid Limit (%)**	**Plasticity Index (%)**	**Natural Soil Suction (kPa)**	**Potential Swell (%)**
Low	<50	<25	<1.5	<0.5
Marginal	50 – 60	25 – 35	1.5 – 4	0.5 -1.5
High	>60	>35	>4	>1.5

Kay (1990) indicated that liquid limit is a good indicator of shrink-swell response for natural soils and classified soil expansiveness based on liquid limit (Table 2.7).

Table 2.7: Liquid limit range and site classification (after Kay 1990)

Site Classification	**Liquid Limit Range**
Slightly expansive (S)	<20
Moderately expansive (M)	20 – 40
Highly expansive (H)	40 – 70
Extremely expansive (E)	>70

2.3.2 Linear shrinkage

Although stated as a test method to determine the suitability of materials for the use of wearing course on road construction projects, most commercial laboratories in South Africa analyse linear shrinkage as a measure of verifying Atterberg limit states. Geotechnical engineers consider linear shrinkage as an additional indicator of heave potential, with several reporting linear shrinkages in excess of 7 as expansive clays. Cerato and Lutenegger (2006) demonstrated that appropriate use of linear shrinkage apparatus can give a good indication of the shrinkage limit of soils.

Stott and Theron (2015) state that materials of high plasticity typically bend and shatter resulting in questionable results when conducted according to the South African standard test methods, while the method suggested by Cerato and Lutenegger (2006) gave an excellent indication of shrinkage limit. Blight and Leong (2012) noted that it is well known that drying (even air-drying) of some soils affects their properties that may lead to an underestimation of the liquid and plastic limits. It is expected that drying will have an influence on the linear shrinkage of samples in a similar manner. Air drying samples to near the shrinkage limit of soils and then oven-drying at 105 to 110 degrees Celsius may prevent the bending and shattering (Blight and Leong, 2012; Stott and Theron, 2015), but it is still unclear if the linear shrinkage values will give a good indication of anything other than the shrinkage limit of soils as it gives no indication of the forces which could lead to changes in volume.

2.3.3 Shrinkage limit

The shrinkage limit of soil is defined as the moisture content at which no further decrease in volume occurs when drying a soil sample (Chen, 1975). Many authors have researched the relationship between the liquid limit and shrinkage limit of soils believing that high liquid limit in relation to shrinkage limit is indicative of expansive soils, but Chen (1975) found that there was no conclusive evidence of a correlation between the shrinkage limit of soils and its swelling potential.

Altmeyer (1955) suggested predicting heave potential of soils using the linear shrinkage and shrinkage limit of soils. The proposed classification system is summarised in Table 2.8.

Table 2.8: Classification of expansiveness of soils based on linear shrinkage and shrinkage limit (after Altmeyer, 1955)

Degree of expansion	**Shrinkage limit as a percentage**	**Linear shrinkage as a percentage**
Critical	<10	>8
Marginal	10 – 12	5 – 8
Non-critical	>12	<5

2.3.4 Clay fraction

Frequently used as an indicator of potential expansiveness, clay content is one of the key considerations for many prediction models, including those by Skempton

(1953), van der Merwe (1964), Nayak and Christensen (1974) and several others. Primarily the clay fraction of soil samples is determined by hydrometer analysis where the clay fraction is determined using sedimentation, de-flocculation and Stoke’s law to determine the finer fractions. It is a relatively cheap and commercially available test. Typically the fraction of soils smaller than 0.002mm is considered to be the clay fraction, while some classification systems refer to the fraction smaller than 0.005mm as the clay fraction. Particle site classes commonly used in civil engineering are indicated in Figure 2.6.

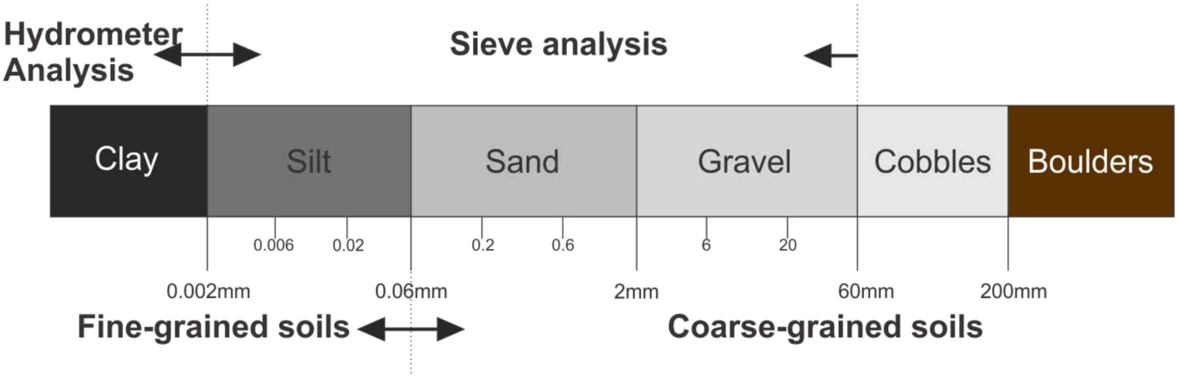
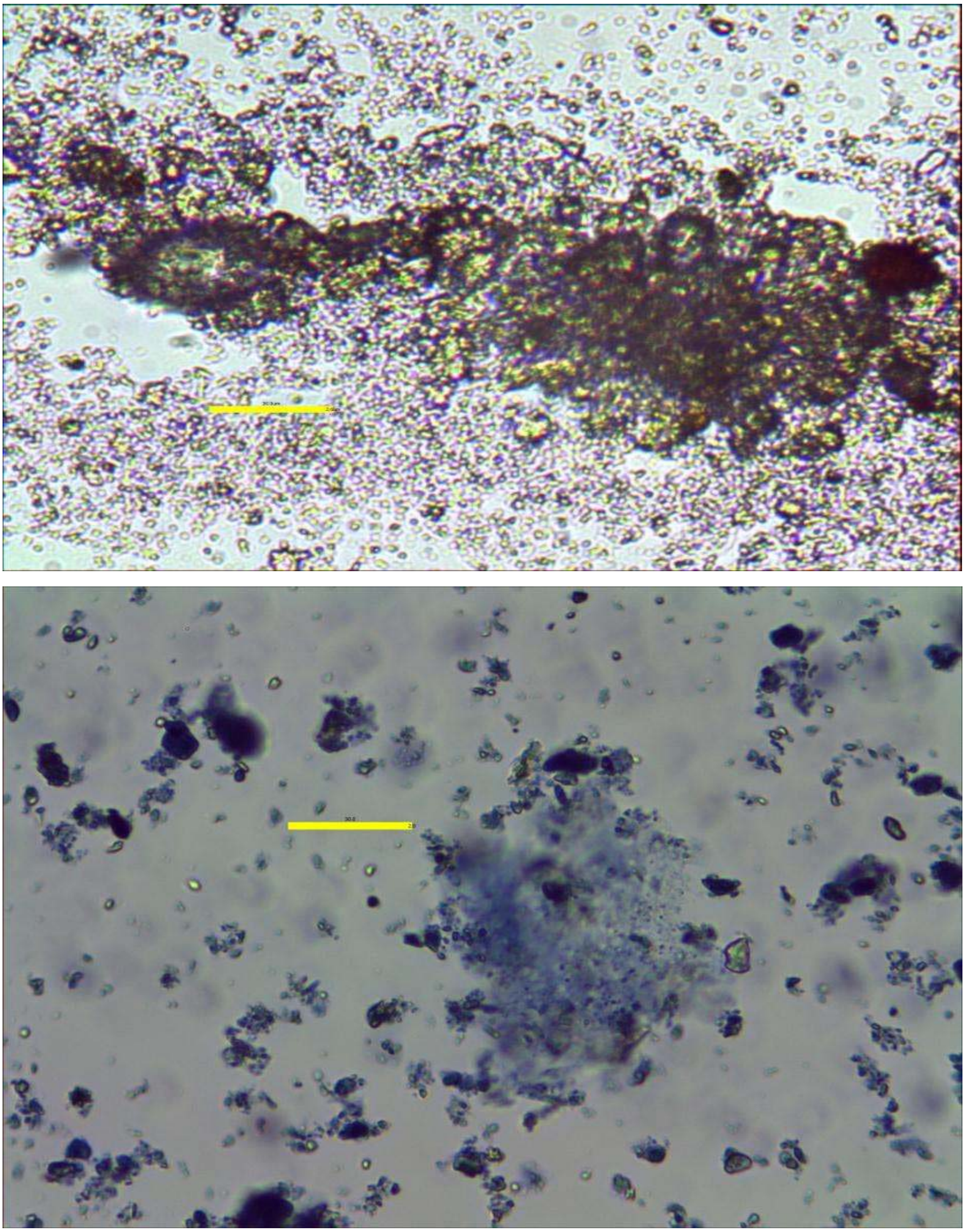


Figure 2.6: Particle size classes commonly used in civil engineering

Rolfe, Miller and McQueen (1960); Nettleship, Cisko and Vallejo (1997), Savage (2007) and Stott and Theron (2015) raised questions on the reliability of hydrometer analysis to determine the finer fraction.

Stott and Theron (2016) highlighted the shortcomings of the hydrometer method extensively confirming Savage’s (2007) suspicion that de-flocculation of many clays is seldom fully achieved. Stott and Theron (2016) used methylene blue to indicate that generally good dispersion was not achieved, even when the dispersive agent and dispersion time were increased. Stott and Theron (2016) claims that the hydrometer method is unreliable for critical analysis such as heave prediction. Figures 2.7a and b indicate two poorly dispersed samples with massive agglomerations. Methylene blue was added in order to mark the clay. These figures were taken from Stott and Theron (2016).



Figures 2.7a and b: Using Methylene Blue to "flag" clay minerals after dispersion, taken from Stott and Theron (2016)

Holtz and Gibbs proposed the following criteria for the classification of expansive materials (Table 2.9):

Table 2.9: Expansive potential of soil (after Holtz and Gibbs, 1956)

Degree of Expansion	Probable expansion (% of total volume change)	Colloid content %	Plasticity Index %	Shrinkage Limit %
Low	<10	<15	<18	<10
Medium	10 – 20	13 – 23	15 – 28	10 – 20
High	20 – 30	20 – 31	25 – 41	20 – 30
Very High	>30	>28	>35	>30

2.3.5 Density and moisture content of *in-situ* soil, dry density and optimum moisture content

Many methods used to classify the potential expansiveness of soils rely on the *in-situ* density of soils or the moisture content of the soils. These values are viewed in relation to the liquid limit, plasticity index, clay fraction or other density or moisture measures to obtain an estimation of heave potential (Komornik and David, 1969; Chen, 1973; Nelson and Miller, 1992).

Soils with low densities and high moisture contents and high Activity ratings are considered to be prone to shrinkage while soils with high densities, low moisture contents and high Activity ratings are considered to be highly expansive (Nelson and Miller, 1992).

The *in-situ* moisture content compared with the index values of soils is considered to give insight into whether soil is expected to shrink or swell.

2.3.6 California bearing ratio test (swell percentage)

The California bearing ratio (CBR) test is a popular test among commercial laboratories as it is regularly used with road construction. The one-dimensional swell under a surcharge pressure of compacted or remoulded samples are measured while the samples are being saturated with water (Kassif *et al.*, 1969; Yoder and Witczak, 1975).

2.3.7 Soil classification systems

Soil classification systems such as the unified soils classification (USCS) categorises soils into different classifications based on particle size distribution and index properties. Certain classifications are associated with heaving clays.

2.3.8 Visual identification

Wayne, Mohamed, and El-Fatih (1984) identified visual indications of expansive materials as a visual estimation of expansiveness potential which includes:

a) Wide and deep shrinkage cracks occurring during dry periods;
b) Soil is rock-hard when dry, but very stiff and sticky when wet;
c) Damages on the surrounding structures due to expansive soil; and
d) Field findings may be verified by laboratory testing that may give an indication of the soils mineralogy and particle size.

2.4 Direct Methods to Identify Expansive Soils

Direct methods are methods considered to give a direct indication of heave potential. They are predominantly divided between suction-based methods, oedometer-based methods and other methods.

2.4.1 Oedometer-based methods

An oedometer is a one-dimensional consolidation apparatus. It is the most commonly used test to predict heave (Nelson and Miller, 1992). The method proposed by Jennings and Knight (1957) and improved by Jennings *et al.* (1973) is generally used by geotechnical engineers in South Africa and is indicated in Figure 2.10.

Consolidation-swell oedometer tests (indicated in Figure 2.8) involve an initial loading of unsaturated soil to a prescribed stress. The loading remains constant, allowing the soil to swell under the load imposed when water is added. The initial load may represent expected overburden, expected overburden plus structural load or any other design load. After increasing in volume, the soil sample is further loaded and unloaded in the conventional manner. The swell pressure is typically defined as the pressure required to compress a sample to its original volume (Nelson and Miller, 1992).

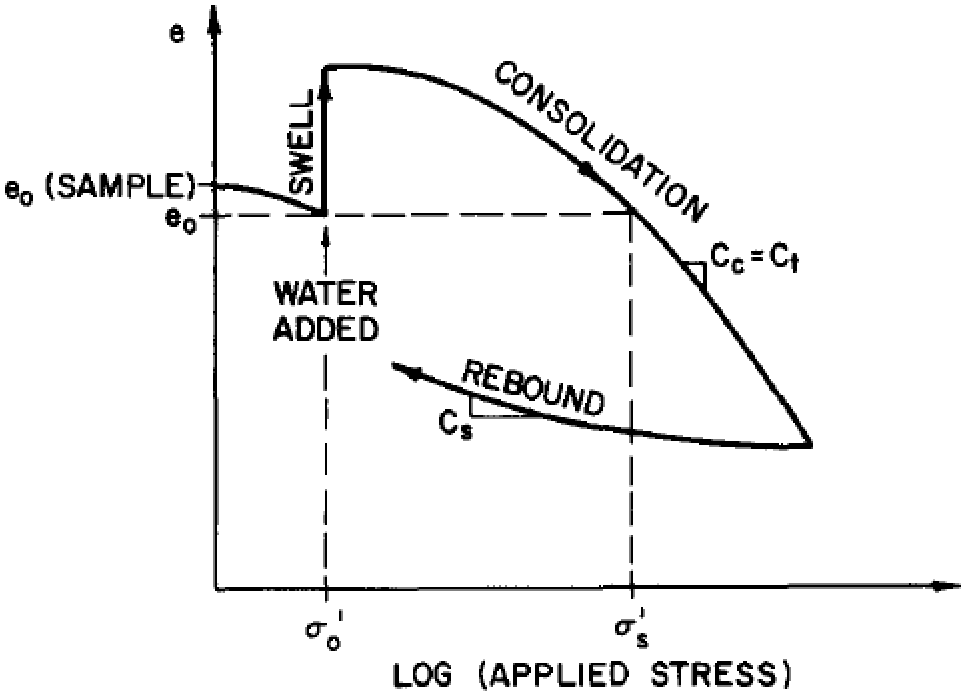
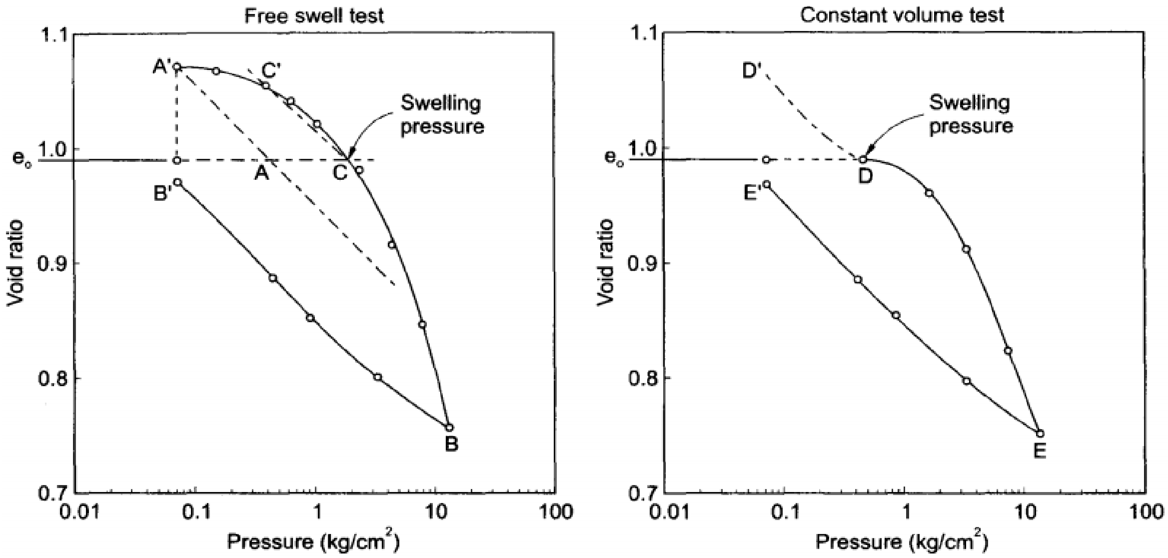


Figure 2.8: Typical consolidation test results on consolidation-swell tests (after Nelson and Miller, 1992)

Constant volume oedometer tests involve submerging the sample in the oedometer apparatus while preventing volume change. The swell pressure is then reported as the maximum stress applied to maintain constant volume (Nelson and Miller, 1992). A typical schematic of the constant volume test (two-dimensional) is displayed in Figure 2.9b.

Free swell oedometer tests are described as being similar to *consolidation-swell* oedometer tests with no initial load. The sample is allowed to swell freely and the volume change is recorded as well as initial and final void ratio (Mansour, 2011). A typical schematic of the Free swell test (two-dimensional) is displayed in Figure 2.9a.

The loading and wetting sequence, sample disturbance and apparatus compressibility must be taken into account when analysing oedometer tests. With conventional consolidometers, only the total stress can be controlled. Matric suction is determined by measuring the initial total stress, inundating the sample and determining the difference in stress (Nelson and Miller, 1992).



Nelson and Miller (1992) lists fourteen heave prediction methods using oedometers, with Hongyu (2015) listing an additional eleven methods. They are summarised in Table 2.10.

Figure 2.9a and b: Typical consolidation test results; a) Free swell and b) Constant volume (after Fredlund, 1969)

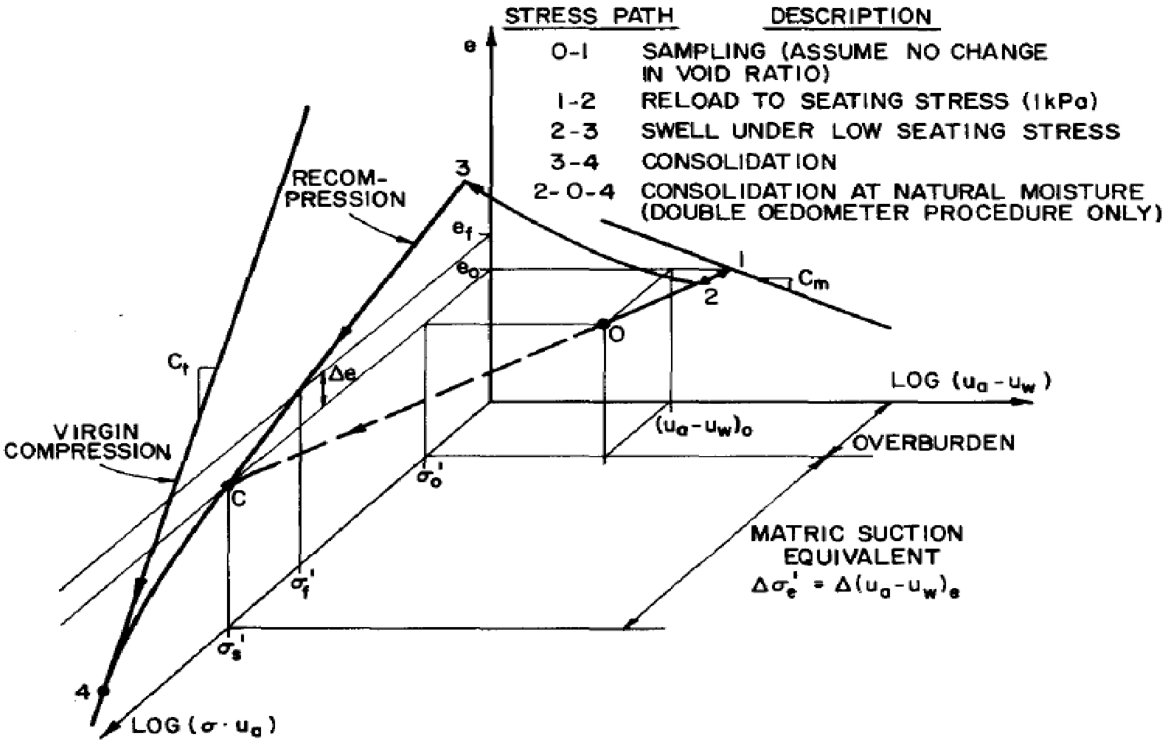


Figure 2.10: An idealised double and simplified oedometer test with stress paths represented in terms of stress-state variables (after Nelson and Miller, 1992)

Table 2.10: Heave prediction using oedometers (adapted from Nelson and Miller, 1992; and Hongyu, 2015)

Test Name		**Description**	**Reference**
1	Double oedometer method	Two tests performed on adjacent samples; a consolidation-swell test under a small surcharge pressure and a consolidation test, performed in the conventional manner but at natural moisture content. Analysis accounts for sample disturbance and allows simulation of various loading conditions and final pore-water pressures.	Jennings and Knight (1957)
2	Volumenometer method	In specialized apparatus, air-dried samples were inundated slowly under overburden pressure.	De Bruijn (1961)
3	Salas and Serratosa method	An oedometer based heave prediction model incorporating the “swelling pressure” of a soil into the equation. The “swelling pressure” is the pressure in an oedometer test required to prevent a saturated soil sample from swelling (Nelson, Chao, Overton and Nelson, 2015).	Nelson *et al.* (2015)
4	Sampson, Schuster, and Budge (1965) method	Two tests performed on adjacent samples to simulate highway cut conditions; a consolidation-swell test under overburden surcharge, and constant volume-rebound upon load removal test.	Sampson, Schuster and Budge (1965)
5	Noble method	Consolidation-swell tests of remoulded and undisturbed samples at various surcharge loads to develop empirical relationships for Canadian prairie clays.	Noble (1966)
6	Sullivan and McClelland method	Constant volume test, samples initially at overburden pressure on inundation.	Sullivan and McClelland (1969)

7	Komomik, Wiseman, and Ben-Yaacob method	Constant volume tests at various depths and swell-consolidation tests at various initial surcharge pressures representing overburden plus equilibrium pore water suction, used to develop swell versus depth curves.	Komornik *et al*. (1969)
8	Holtz Method	Assumed a standard overburden pressure of 0.1MPa with a correction factor applied on the swell potential determined. Three curves were plotted: a.) Expanded and loaded; b.) loaded and expanded; and c.) estimation of load-expansion relationship.	Holtz (1970)
9	Navy method	Swell versus depth curves determined by consolidation-swell tests at various surcharge pressures representing overburden plus structural loads	Navy (1971)
10	Wong and Yong method	Swell versus depth determined as in (7) and (9), but surcharge loads of overburden plus hydrostatic pore water pressures used	Wong and Yong (1973)
11	USBR method	Double sample test, a consolidation-swell under light load (l psi), and a constant volume test	Gibbs (1973)
12	Simple oedometer	Improved from double oedometer test (1). Single sample loaded to overburden, then unloaded to constant seating load, inundated and allowed to swell, followed by usual consolidation procedure	Jennings, Firtu, Ralph and Nagar (1973)
13	Direct model method (Texas State and Highway Dept)	Consolidation-swell tests on samples inundated at overburden or end-of construction surcharge loads	Smith (1973)

14	Mississippi State Highway Dept. method	Consolidation-swell tests on remoulded or undisturbed samples, inundated at overburden surcharge loads	Teng, Mettox and Clisby (1972; 1973) and Teng and Clisby (1975)
15	Controlled strain test	Constant volume swell pressure obtained on inundation followed by incremental, strain-controlled pressure reduction	Porter and Nelson (1980)
16	Univ. of Saskatchewan	Constant volume test. Analysis corrects for sample disturbance and apparatus deflection	Fredlund, Hasan and Filson (1980)
17	Sridharan, Rao and Sivapullaiah method	Free swell oedometer test where a sample is allowed to swell freely while wetted with a token load applied to gradually consolidate the sample back to its original volume. The swelling pressure is defined as the pressure necessary to consolidate the sample to its original volume.	Sridharan, Rao and Sivapullaiah (1986)
18	Erol, Dhowian and Youssef method	Proposed corrections to oedometer heave to account for stress conditions. Indicated that the axial swell strain is stress path dependant. The correction, based on field tests, was in the order of one-third.	Erol, Dhowian and Youssef (1987)
19	Shanker, Ratnam and Rao method	Developed in India. Not researched.	Hongyu (2015)
20	Heave index method	Nelson, Durkee and Bonner (1998) and Nelson, Reichler and Cumbers (2006) introduced the heave index in the heave equation. The heave index is determined from constant volume and constant stress tests. The heave index value is applied over the entire height	Nelson *et al.*(1998)

		of the soil layer and does not take into account the nonlinearity of the strain over the thickness of the layer (Nelson *et al.,* 2015).	
21	Al-Shamrani and Al-Mhaidib method	Suggested that better estimates of vertical swell may be determined using triaxial stress path cells under various loading conditions.	Al-Shamrani and Al-Mhaidib (1999)
22	Basma, Al-Homoud and Malkawi method	The cyclic swelling behaviour of clays were investigated which indicated a reduced swell for every cycle until equilibrium was reached after four or five cycles.	Basma, Al-Homoud and Malkawi (2000)
23	Subba Rao and Tripathy method	Subba Rao and Tripathy (2003) confirmed that the swelling potential of compacted expansive soils decreased with ageing and suggested that particle rearrangement and the formation of bonds with time are the reasons for the increase in strength and decrease in swelling potential.	Subba Rao and Tripathy (2003)
24	Nelson *et al*. 2006 method	Heave prediction by utilising a loaded swell test and a constant volume swell test. Samples are confined at overburden stress corresponding to the depth sampled and specific weight of overburden materials.	Nelson *et al.* (2006)
25	Singhal method	Similar to Nelson *et al.* (2006) method, but instead of using a three dimensional stress path, a surrogate stress path is used to determine the maximum potential heave.	Singhal (2011)

2.4.2 Suction-based methods

The response of soil to suction can be predicted with a manner similar to the reaction of soil to saturated stress changes. The relationship between matric suction and void ratio is comparable to the compressibility index determined by oedometer tests. Predictions of heave are calculated in a similar way to *reverse consolidation* equations used in oedometer test methods (Nelson and Miller, 1992).

Comparison testing using oedometer based methods and suction based methods, showed that suction based methods were simpler, more economical and more expedient (Johnson, 1977).

The concept of suction needs to be understood in order to understand the mechanism involving suction measurements. Suction is often referred to as a free energy state of pore water and is measured as partial vapour pressure of pore water (Richards, 1965). Vapour pressure is best explained in terms of kinetic theory. Molecules in liquid state are in constant motion and collide with each other, at the free surface of the liquid; a molecule can absorb enough energy to facilitate phase change. Molecules changing phases (from liquid to gas) results in the development of vapour pressure of which the magnitude depends on the rate of phase change from liquid to gas. Molecules may phase back from gaseous to liquid form which results in the creation of partial pressure. When the rate of the opposing processes is in equilibrium, the gas above the liquid is saturated with vapour and is termed saturation pressure. The equilibrium vapour pressure is a function of the concentration of solubles and the water pressure and temperature (Theron, 2000).

Soil suction consists of osmotic and matric suction. The total suction is equal to the sum of the matric and osmotic suctions (Mansour, 2011). Matric suction is the attraction of water to the soil particle surfaces and depends on the pore size distribution (Figure 2.11). Osmotic suction of the pore fluid is evaluated as the difference between dissolved salts concentration of the pore water and reservoir water salinity (Linchang and Xin, 2004).

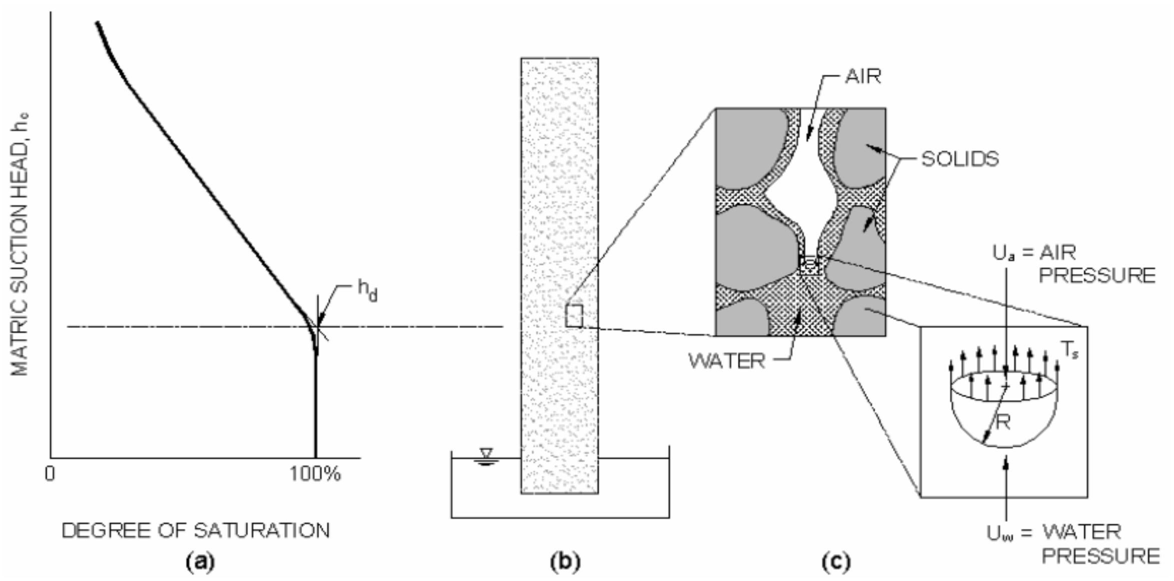


Figure 2.11: Matric suction in soil (after Nelson *et al.*, 2015)

Figure 2.11 (a) indicates capillary head, h_d, which represents the air pressure that would be necessary to displace the water from the pore spaces, while figures 2.11 (b) and (c) indicates a granular column of soil allowed to drain freely and the surface tension, T_s, preventing the water from draining away.

The water vapour pressure is related to the total suction in the air voids of the soil. Factors that may influence water vapour pressure include initial density and moisture content, environmental factors such as described in Table 2.3, permeability and the soil structure (Linchang and Xin, 2004).

To predict the amount of heave that may be expected at a particular time, it is necessary to know what zone of soil is being wetted and what the expansive nature of the soil is. This is dependent on the ground water movement in the soil. Nelson *et al.* (2015) proposed the following definitions regarding the depth of water movement:

a) *<u>Active Zone</u> is that zone of soil that is contributing to heave due to soil expansion at any particular time. The active zone will normally vary with time;*
b) *<u>Zone of Seasonal Moisture Fluctuation</u> is that zone of soil in which water contents change due to climatic changes at the ground surface;*

c) *<u>Depth of Wetting</u> is the depth to which water contents have increased due to the introduction of water from external sources, or due to capillarity after the elimination of evapo-transpiration. The external sources can include such things as irrigation, seepage from ponds or ditches, broken water lines, and others; and*

d) *<u>Depth of Potential Heave</u> is the depth to which the overburden vertical stress equals the swelling pressure of the soil. This represents the maximum depth of Active Zone that could occur.*

Generally, expansive soils are underlain by various strata with a range of permeabilities. Layers with low permeability intercept water moving through the unsaturated zone and form perched water tables. The degrees of saturation below such perched water tables are thus discontinuous with unsaturated conditions above and below perched water tables (Freeze and Cherry, 1979). Durkee (2000) indicated that the migration of a wetting front will continue to move downward with time unless bounded by a truly impermeable barrier.

The soil-water characteristic curve (SWCC) describes the relationship between moisture content and soil suction for a soil specimen (Fredlund and Rahardjo, 1993). The amount of moisture in soil is generally quantified in terms of gravimetric water content, volumetric water content or degree of saturation.

A typical example of a soil-water characteristic curve is indicated in Figure 2.12 indicating the drying curve at a given soil suction is higher than that of the wetting curve. The end point of the wetting curve differs from that of the drying curve due to the entrapment of air in the soil specimen. The contact angle at a receding interface varies from that of an advancing interface. The entrapment of air and the different contact angle are considered to be the main causes for hysteresis in the soil-water characteristic curve (Fredlund and Rahardjo, 1993).

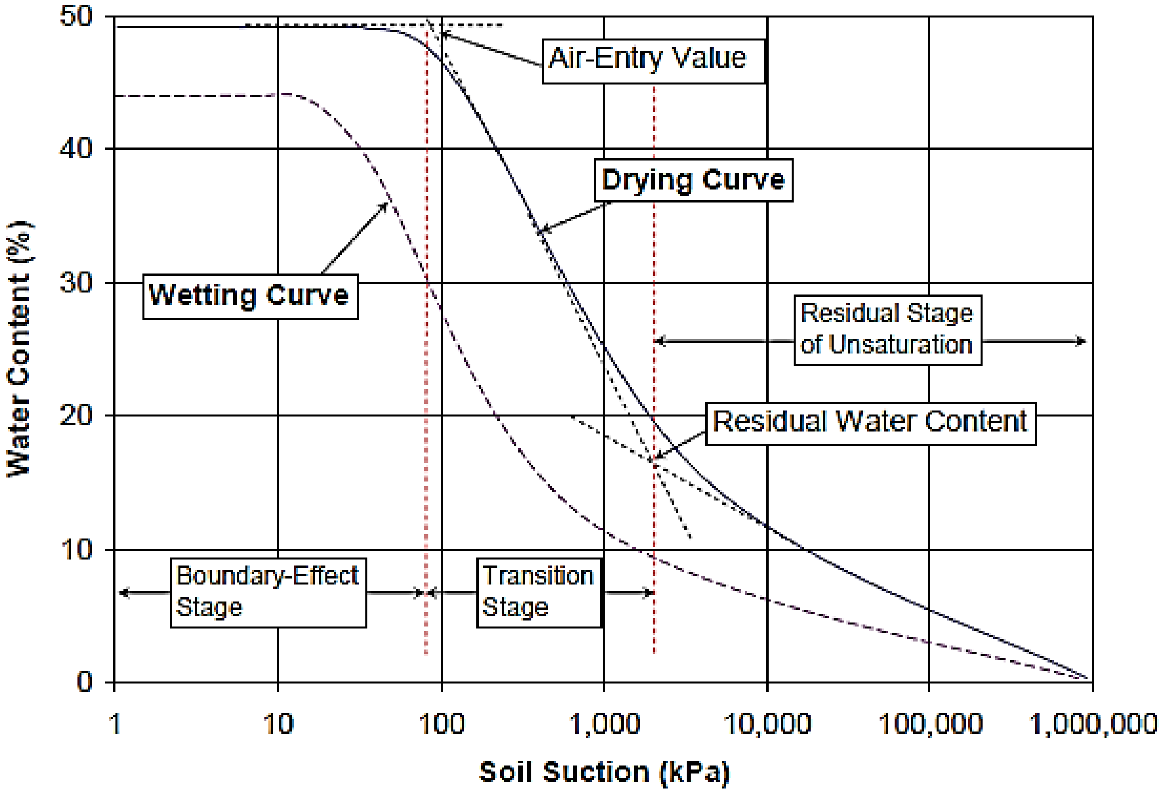


Figure 2.12: Soil-water characteristic curve (SWCC) (after Chao, 2007)

Tests associated with suction provide an insight into the behaviour of expansive soils. Suction tests measure the free energy content of pore pressure in the soil. Suction tests are generally less time exhaustive than oedometer tests and the measured data are applicable to a range of moisture conditions (Erzin and Erol, 2007).

Several soil suction test methods exist, including psychrometers, tensiometers, pressure plate, filter paper and thermal conductivity sensors. Each method has advantages and disadvantages. Psychrometers are sensitive to temperature variations, require frequent recalibration and can only measure total suction. Tensiometers require daily maintenance. Filter paper is considered to be a good method for the measurement of matric and total suctions of soil samples. Pressure plates, membranes and thermal conductivity sensors only measure matric suction (Manosuthkij, Puppala, Nazarian, Saride and Hoyos*,* 2008). Figure 2.13 indicates total, matric and osmotic suction determined using a psychrometer, pressure plate and the squeezing technique. The sum of the matric and osmotic suctions compare favourably with the total suction measured by psychrometer.

The filter paper method is inexpensive and relatively simple, but obtaining equilibrium may take a long time. The moisture in a filter material will reach equilibrium with the surrounding environment either through vapour or liquid flow. Initially, the filter paper should be calibrated (plot a calibration curve) to give either total or matric suction. Once equilibrium is achieved, the moisture content is determined and compared to the calibration curve to determine the suction value according to the ASTM D5298 method (Mansour, 2011).

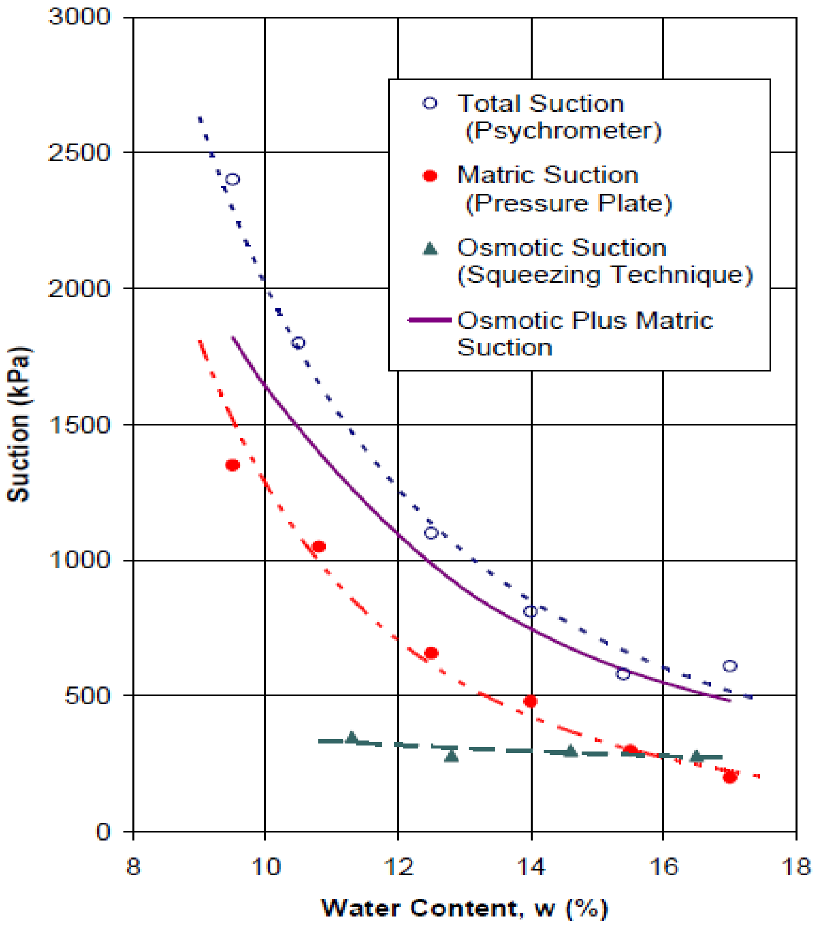


Figure 2.13: Total, matric and osmotic suction (from Krahn and Fredlund, 1971)

2.4.2.1 Fredlund SWCC Device

Designed by Geotechnical Consulting and Testing Systems Incorporated (GCTS) in 2004, the Fredlund SWCC device is an unsaturated soil testing apparatus with flexibility for controlling the matric suction while applying total stress and measuring volume change, allowing various stress paths to be followed (Perera, Padilla and Fredlund, 2004; Chao, 2007). The device, pictured in Figure 2.14, consists of a pressure cell assembly, a pressure panel and a pneumatic loading frame. The pressure panel contains two volume indicator tubes and two pressure gauges allowing graduated readings of the volume of water released from or absorbed into the soil samples during tests. The flow of water into or out of the soil allows for the determination of water content of the soil specimens analysed. The pneumatic loading frame applies normal loads up to 10kN to the soil specimens (Chao, 2007).

Figure 2.14: Fredlund SWCC device with loading frame (after Chao, 2007)

In order to achieve reasonable soil-water characteristic curves, earlier versions of pressure plate apparatus required several soil samples. It is almost impossible to obtain several soil samples representing the same soil structure and stress state conditions, an inconvenience overcome by the Fredlund SWCC device. This device allows the use of a single soil sample to obtain a reasonable soil-water characteristic curve with any number of data points. Overall volume change can be recorded by taking measurements of the load plate movements using an attached dial gauge (Chao, 2007).

2.4.2.2 Filter paper method and calibration curves

The filter paper method (Figure 2.15) was developed in the agricultural soil science discipline and has been used routinely by the Water Resources Division of the U.S. Geological Survey for many years (McQueen and Miller, 1968). The advantage of using filter paper is the wide range of suction values that may be recorded and the simplicity of use, while the degree of accuracy required to weigh the filter paper is a disadvantage (Chao, 2007).

The method is based on the principle that the suction of the filter paper will come to equilibrium with the suction of the soil sample. In this method, a sample of soil is placed in a closed container with a calibrated filter paper. The filter paper and sample are allowed to equilibrate for a period of time at constant temperature. Once equilibrium is achieved, the filter paper is weighed using a balance, accurate to 0.0001grams, before and after oven drying (Chao, 2007).

The relative humidity inside the container prior to equilibrium being reached is controlled by the soil suction. Equilibrium may be reached by vapour moisture exchange between the filter paper and the sample or by liquid exchange. Vapour exchange will give an indication of total suction while liquid exchange will give an indication of matric suction (Chao, 2007).

Calibration curves, such as ASTM D5298-10 may be used to obtain matric suction of soil samples using the filter paper method (Figure 2.16) (Chao, 2007).

Figure 2.15: Filter paper test equipment, including Whatman® no 42 filter paper (from Chao, 2007)

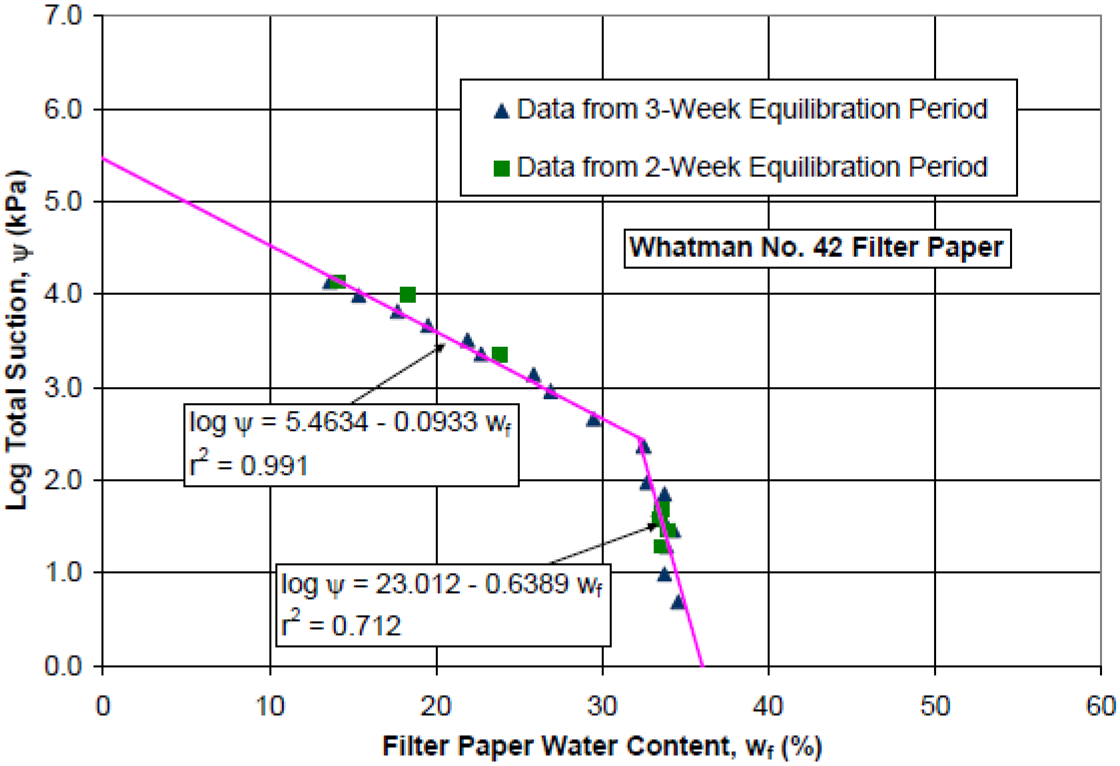


Figure 2.16: Filter paper calibration curve for total suction (after Chao, 2007)

2.4.2.3 WES method – US Army corps of engineers

The United States Army Corps of Engineers Waterways Experiment Station (WES) conducted extensive laboratory analysis and field studies to develop a prediction model for the use of foundation design. Based on the comparisons made between suction tests and oedometer tests and the comparison of other test results, the suction index was not measured but calculated using an empirical calculation equating suction index to the product of the compressibility factor and the specific gravity of solids as a ratio to the slope of suction versus water content curve (Johnson and Snethen, 1978).

The compressibility factor relates to changes in volume and water content which may be determined in a similar way to the shrinkage limit. It may also be calculated similar to the CLOD test described below.

The volumetric compressibility factor generally decreased with increasing confining pressure. Thermocouple psychrometers and pressure membrane methods were used to measure initial suction values and suction versus water content relationships in the WES studies.

2.4.2.2 The COLE and CLOD tests

The coefficient of linear extensibility (COLE) test is routinely used by the National Soil Survey Laboratory in the United States to characterise the shrink-swell potential of soils. Undisturbed soil samples are coated with a flexible plastic resin (Saran resin) that is impermeable to liquid water but permeable to water vapour. The samples (clods) are weighed in air and in water to determine the field density using the Archimedes principle. The clods are then oven-dried and weight and volume measurements are taken again. The change in volume from field density is determined by the relationship between oven dry bulk density and the bulk density at field density (Brasher, Franzmeier, Valassis and Davidson, 1966).
Often used in America and considered to be the best heave indicator test available (Nelson and Miller 1992), the CLOD test suggested by McKeen and Nielsen (1978) employs an unrestrained shrinkage test on an undisturbed soil sample using Saran

resin. Irregular lumps of soil may be used. The change in volume of the sample is determined as a function of its moisture content as it dries, providing a volume change index. The test relies on the Saran resin to form a semi-permeable coating around the sample which allows water vapour to permeate but effectively impermeable to short term water immersion.

The Saran resin is not available in South Africa and the time reported to obtain shrinkage curves are typically lengthy. It is therefore not considered to be a viable test for the use of low-cost housing.

2.4.3 Other direct methods

Mitchell (1993) indicated that using X-ray diffraction, electron microscopy, differential thermal analysis and wet chemical analysis may be used to identify clay minerals in soils. From a mineralogical point these methods may give a good indication of the quantitative expandable clay minerals. These methods are typically very expensive, have special requirements, require expertise that is not always readily available and require expensive equipment; therefore these methods are not routinely used by civil engineers (Chen, 1975).

Physicochemical methods include cation exchange capacity, a measure of ion adsorption properties of clay minerals which gives a good indication of hydration properties of clay. The cation exchange capacity increases from kaolinite to montmorillonite (Snethen *et al*., 1975 and Mitchell and Soga, 2005).

Physical methods include colloidal content from hydrometer analysis, specific surface area of clay particles, soil fabric by electron microscopy and structure by X radiology. None of these methods give a direct measurement of swell potential, but considered with others may give a good indication (Nelson and Miller, 1992).

Index properties, including the Atterberg limits and linear shrinkage of soil samples, are considered to give good correlation to swell potential (Nelson and Miller, 1992).

2.4.3.1 Free swell ratio

Free swell is defined as the ratio of the wet bulk volume to the dry bulk volume as a percentage. A small sample of $10cm^3$ of oven-dried soil passing the 0.425mm sieve is slowly poured into a $100cm^3$ measuring jar filled with distilled water. The change in volume of the soil is noted and reported as a percentage of initial volume. Due to the difficulty in accurately measuring out $10cm^3$, various papers suggest using 10g of material. Holtz and Gibbs (1956) stated that soils with free swells greater than 100% can cause considerable damage while free swell index values below 50% probably do not exhibit appreciable volume changes. In a paper by Dawson (1953), several Texas clays reported free swell index values below 50% and showed extensive expansion. Prakash and Sridharan (2004) propose that the free swell ratio can be used to estimate the clay mineralogy to an acceptable degree for civil engineering purposes.

2.4.3.2 Methylene blue adsorption

The methylene blue test was developed in France to determine the suitability of granular materials for use in the manufacturing of concrete by assessing the presence of clays. Methylene blue powder performs like a cationic colorant when mixed with water. It is identified by the chemical formula $C_{16}H_{18}N_3SCl$. When mixed with soil in solution, the chloride ions take the place of cations in the clay minerals to be adsorbed to the surface of clay minerals. The amount of methylene blue solution adsorbed varies according to the amount of clay minerals and clay type, cation exchange capacity and specific surface area. It is a relatively cheap and easy test that does not require expensive equipment (Türköz and Tosun, 2011). Since methylene blue has a high propensity to be adsorbed into negatively charged clay mineral surfaces, it can give a good indication of the cation exchange capacity of a clay mineral (Çokca and Birand, 1993).

2.5 Empirical Methods

Soil classification parameters such as liquid limit, plastic limit, plasticity index, clay fraction, shrinkage limit, linear shrinkage, moisture content and dry density are used to empirically predict the behaviour of expansive soils (Hongyu, 2015). Empirical relations between the percentage of expected swell, S%, swelling pressure, p_s, and the various soil parameters are summarised in Table 2.11.

Table 2.11: Empirical relationships for swelling pressure and swelling potential adapted after Hongyu, 2015)

Seed *et al.* (1962)	$S\ \% = 0.00216 \cdot I_p^{\,2.44}$ for undisturbed $S\ \% = 0.0036 \cdot I_p^{\,2.44}$ for disturbed soils $S\ \% = 3.6 \times 10^{-5} \cdot \mathrm{A}^{2.44} \cdot \mathrm{c}^{3.44}$ I_p = plasticity index c = clay content (fraction smaller than 0.002mm) A = activity = (I_p / c)
Van der Merwe (1964) and van der Merwe (1976)	$\Delta H = -[F \cdot e^{-0.377 \cdot \mathrm{D}} \cdot (\mathrm{e}^{-0.377 \cdot \mathrm{T}} - 1)]$ ΔH = total heave F = correction factor for degree of expansiveness D = the thickness of non-expansive layer T = thickness of expansive layer
Ranganatham and Satyanarayana (1965)	$S\ \% = 0.000413 \cdot \mathrm{I_s}^{2.67}$ I_s = shrinkage index, ($w_l - w_s$) w_l = liquid limit w_s = shrinkage limit
Komornik and David (1969)	$\mathrm{Log}\ p_s = -2.132 + 0.0208 w_l + 0.000665 \gamma_d - 0.0269 w_i$ p_s = swelling pressure w_l = liquid limit γ_d = dry unit weight w_i = initial moisture content
Nayak and Christensen (1971)	$S\ \% = 0.00229 \cdot I_p^{2.67} \cdot (1.45 \cdot c) / w_i + 6.38$ $p_s\ (\mathrm{psi}) = [(3.58 \cdot 10^{-2})\ I_p^{1.12} \cdot c^2 / w_i^{\,2}] + 3.79$ w_i = initial moisture content p_s = swelling pressure
Vijayvergiya and Ghazzaly (1973)	$S\ \% = (0.44\, w_l - w_i + 5.5) / 12$ $\mathrm{Log}\ (S\ \%) = 0.0526 \cdot \gamma_d + 0.033 \cdot w_l - 6.8$ w_l = liquid limit

	w_i = initial moisture content γ_d = dry unit weight
Schneider and Poor (1974)	$\text{Log}\ (S\ \%) = 0.9\ (I_p\ /\ w_i) - 1.19$ I_p = plasticity index w_i = initial moisture content
McCormack and Wilding (1975)	$S\ \% = 7.5 - 0.8 \cdot w_i + 0.203 \cdot c$ w_i = initial moisture content c = clay content (fraction smaller than 0.002mm)
Brackley (1975)	$S\ \% = (5.3 - (147e\ /\ I_p) - \text{Log}\ P) \times (0.525 \cdot I_p + 4.1 - 0.85 \cdot w_i)$ I_p = plasticity index P = surcharge w_i = initial moisture content
O'Neil and Ghazzally (1977)	$S\ \% = 2.77 + 0.131 \cdot w_l - 0.27 \cdot w_n$ w_l = liquid limit w_n = natural water content
Chen (1975)	$S\ \% = 0.2558 \cdot e^{0.00838 \cdot Ip}$ I_p = plasticity index
Johnson (1978)	$S\ \% = 23.82 + 0.7346 \cdot I_p - 0.1458 \cdot H - 1.7 \cdot w_0 + (0.0025 \cdot I_p) \cdot w_0 - (0.00884 \cdot \mathrm{I_p}) \cdot H$ $S\ \% = -9.18 + 1.5546 \cdot I_p + 0.08424 \cdot H + 0.1 \cdot w_0 - (0.0432 \cdot I_p) \cdot w_0 - (0.01215 \cdot \mathrm{I_p}) \cdot \mathrm{H}$ w_0 = initial moisture content I_p = plasticity index
Weston (1980)	$S\ \% = 0.00411\ w_{lw}^{4.17} \cdot \sigma_v^{-3.86} \cdot w_i^{-2.33}$ w_{lw} = weighted liquid limit σ_v = vertical stress
Bandyopadhyay (1981)	$S\ \% = 0.00114 \cdot A^{2.559} \cdot c^{3.44}$ I_p = plasticity index c = clay content (fraction smaller than 0.002mm) A = activity = $(I_p\ /\ c)$

Picornell and Lytton (1984)	$\Delta H = \sum[fi\cdot(\Delta v / v)\cdot i]\ H$ H = *the stratum thickness* $(\Delta v/v)i$ = volumetric strain fi = factor to include the effects of the lateral confinements
Dhowian (1990)	$\Delta H = (S\ \%)\cdot H$
Basma (1993)	$S\ \% = 0.00064\cdot I_p^{1.37}\cdot c^{1.37}$ I_p = plasticity index c = clay content (fraction smaller than 0.002mm)
Çokça (2002)	$S\ \% = -121.807 + 12.1696\cdot \mathrm{MBV} + 27.6579\cdot \mathrm{Log}\cdot \psi_i$ MBV = Methylene blue value ψ_i = Initial suction
Erguler and Ulusay (2003)	$p_s = -227.27 + 2.14\cdot w_i + 1.54\cdot w_l + 72.49\cdot \gamma_d$ p_s = the swelling pressure w_i = initial moisture content w_l = liquid limit γ_d = dry unit weight
Rao, Phanikumar and Sharma (2004)	$S\ \% = 4.24\gamma_{di} - 0.47w_i - 0.14q_i - 0.06\mathrm{FSI} - 55$ γ_{di} = dry unit weight w_i = initial moisture content q_i = initial surcharge FSI = Free Swell Index
Erzin and Erol (2004)	$\mathrm{Log}\ p_s = -4.812 + 0.01405\cdot I_p + 2.394\cdot \gamma_d - 0.0163\cdot w_i$ $\mathrm{Log}\ p_s = -5.020 + 0.01383\cdot I_p + 2.356\cdot \gamma_d$ p_s = the swelling pressure I_p = plasticity index γ_d = dry unit weight w_i = initial moisture content
Sabtan (2005)	$S\ \% = 1.0 + 0.06\cdot(c + I_p - w_i)$ c = clay content (fraction smaller than 0.002mm) I_p = plasticity index w_i = initial moisture content
Azam (2007)	$S\ \% = 0.6\cdot I_p^{1.188}$ I_p = plasticity index

Yilmaz (2009)	$S\ \% = 2.0981e^{-1.7169\,IL}$ LI = Liquidity index $[(w_l - w) / I_p]$
Sparks (2010)	$\Delta e/(1+e_0)$ *(as percentage)* $= \Delta w_c\% / (1 / G_s + w_0\%/100)$ *Where* $w_0 = 0.266 \cdot (I_p\%/\text{Log } R) \cdot (\text{Log } u_0 - \text{Log } u_f)$ And $\text{Log } R = (G_s . I_p)/(0.918 \cdot (w_l - 5.7))$ Δe = *change in void ratio* e_0 = *initial void ratio* G_s = *specific gravity* I_p = plasticity index u_0 = initial pore pressure u_f= final pore pressure w_l = liquid limit
Türköz and Tosun (2011)	$S\ \% = -57.865 + 37.076 \cdot \rho_d + 0.524 \cdot MBV + \varepsilon$ ρ_d = Dry density MBV = Methylene Blue Value ε = mean-zero (Gaussian random error term)
Çimen, Keskin and Yildirim (2012)	$(S\ \%)_1 = (0.3139 \cdot \gamma_d^{0.3552} - 0.1177 \cdot w_i^{0.4470}) \cdot I_p^{0.9626}$ $(\text{Log } p_s)_1 = 0.0276 \cdot I_p - 365.2118 \cdot \gamma_d^{-2.4616} - 0.0320 \cdot w_i + 2.2292$ $(S\ \%)_2 = (0.4768 \cdot \gamma_d^{0.3888} - 0.0033 \cdot w_i^{1.6045}) \cdot I_p^{0.7224}$ $(\text{Log } p_s)_2 = 0.0239 \cdot I_p - 1285.3723 \cdot \gamma_d^{-3.2768} - 0.0396 \cdot w_i + 2.3238$ $S\ \% = \text{mean } (S\%_1, S\%_2)$ $\text{Log } p_s = \text{mean } [(\text{Log } p_s)_1, (\text{Log } p_s)_2]$
Zumrawi (2013)	$S\ \% = 24.5 \cdot (q^{-0.26}) \cdot (I_p\, c)^{1.26} [F_i - 7.1(q^{0.22}) \cdot (I_p\, c)^{1.26}]$ q = surcharge F_i = initial state factor

Empirical methods were received well by conventional engineering practises due to the ability of most commercial laboratories to conduct the necessary tests to determine the soil parameters needed. These methods were, however, created based on local data and are not universally valid on all types of expansive soils.

CHAPTER 3: A REVIEW INTO CURRENT PROCEDURES USED TO DETERMINE HEAVE POTENTIAL FOR LOW-COST HOUSING DEVELOPMENTS IN SOUTH AFRICA

3.1 Introduction

In order to find a viable approach to resolve the research problem, commercial laboratories were contacted to enquire on the typical approach taken for the determination of potential expansiveness of soils with low-cost housing developments. Most commercial laboratories suggest *foundation indicator* samples be taken and analysed using van der Merwe's empirical method of 1964.

Upon reviewing van der Merwe's method (1964), it became clear that the methods used by the commercial laboratories are an adaptation to van der Merwe's method. Upon further research, it was found that van der Merwe adjusted the method to reflect metric units in 1976. Soillab (Pty) Ltd produced the graph indicating heave prediction from potential expansiveness based on the graphs by van der Merwe in his 1964 and 1976 papers. The classification chart used by commercial laboratories is that of Williams and Donaldson (1980).

For low-cost housing developments in South Africa, van der Merwe's method (1964) is the most popular empirical method with others methods not typically considered. Correspondence with commercial laboratories indicated that potential expansiveness is typically determined by *foundation indicator* tests, which includes the particle size distribution, the Atterberg limits and clay fraction of soils. *Foundation indicators* are used as input parameters to van der Merwe's method (1964). Oedometer tests and tensiometer tests are offered as alternative options, but these tests come with a long waiting period and are typically not considered viable due to the cost and time implications.

The particle size distribution is determined according to TMH1 A1(a) (1986) – "The wet preparation and sieve analysis of gravel, sand and soil samples" or the newer SANS 3001-GR1 (2011) – "Wet preparation and particle size analysis".

The Atterberg limits are determined subsequent to sample preparation by either TMH1 A1(a) or SANS 3001-GR1. The test methods pertaining to the Atterberg limit states are TMH1 A2, TMH1 A3 and TMH1 A4 or SANS 3001-GR10, SANS 3001-GR11 and SANS 3001-GR12.

Hydrometer analysis is performed according to TMH1 A6, SANS 3001-GR3 or ASTM D422.

Several commercial laboratories consider the moisture content to form part of a *foundation indicator*, although several laboratories indicate this as an additional test not necessarily required. The test method governing moisture content of soil samples is TMH1 A17, replaced by SANS 3001-GR20 – “Determination of moisture content by oven-drying”.

The differences between the two sets of test methods are discussed in section 3.3.

3.2 Van der Merwe’s Empirical Method (1964)

The most popular empirical method used in South Africa is that of van der Merwe devised in 1964. Van der Merwe adjusted the method in 1976 to accommodate metric units. Two further improvements were proposed by Williams and Donaldson in 1980 and by Savage in 2007 – but the method remains basically the same. The primary input parameters used by van der Merwe’s method are the Atterberg limits and the particle size distribution of a soil sample (van der Merwe, 1964).

Van der Merwe’s method relies on the starting depth (top) of the particular soil layer, the thickness of the layer, the percentage clay fraction (passing 2µm) and the Atterberg limits of a soil sample. The weighted plasticity index for the entire soil sample is determined and plotted on a graph similar to that of Figure 3.2a**.** The sample is then categorised as having a potential expansiveness of either “*low”, “medium”, “high”* or *“very high”*. After determining the potential expansiveness classification, the corresponding graph illustrated in Figure 3.3 is used to determine total upwards heave that may be expected taking cognisance of the overburden pressure exerted by the material overlying the particular layer. Equation 3.1 below can also be used to calculate the expected heave.

$$\boldsymbol{H} = -[\boldsymbol{F}.\boldsymbol{e}^{-\mathbf{0.377}.\boldsymbol{D}}.(\boldsymbol{e}^{-\mathbf{0.377}.\boldsymbol{T}} - \mathbf{1})] \tag{3.1}$$

Where:

H denotes Heave
F = 0 for “low expansiveness”
F = 0.055 for “medium expansiveness”
F = 0.110 for “high expansiveness”
F = 0.2221 for “very high expansiveness”
D is the thickness (m) of the total non-expansive overburden overlying the specific layer
T is the thickness (m) of the particular layer

Van der Merwe’s method can be summarised as follows: (Figure 3.1)

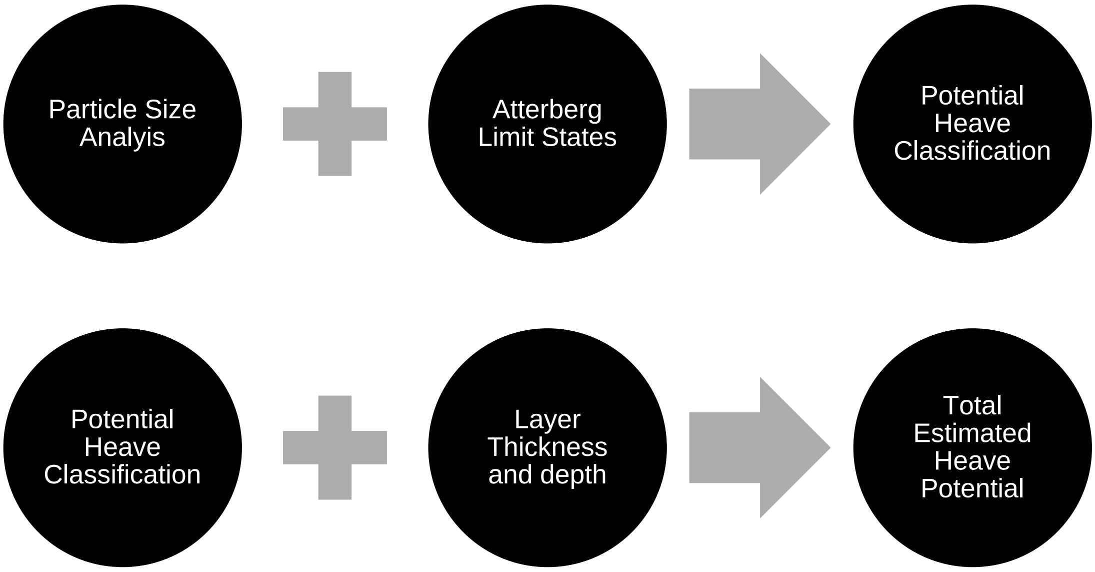
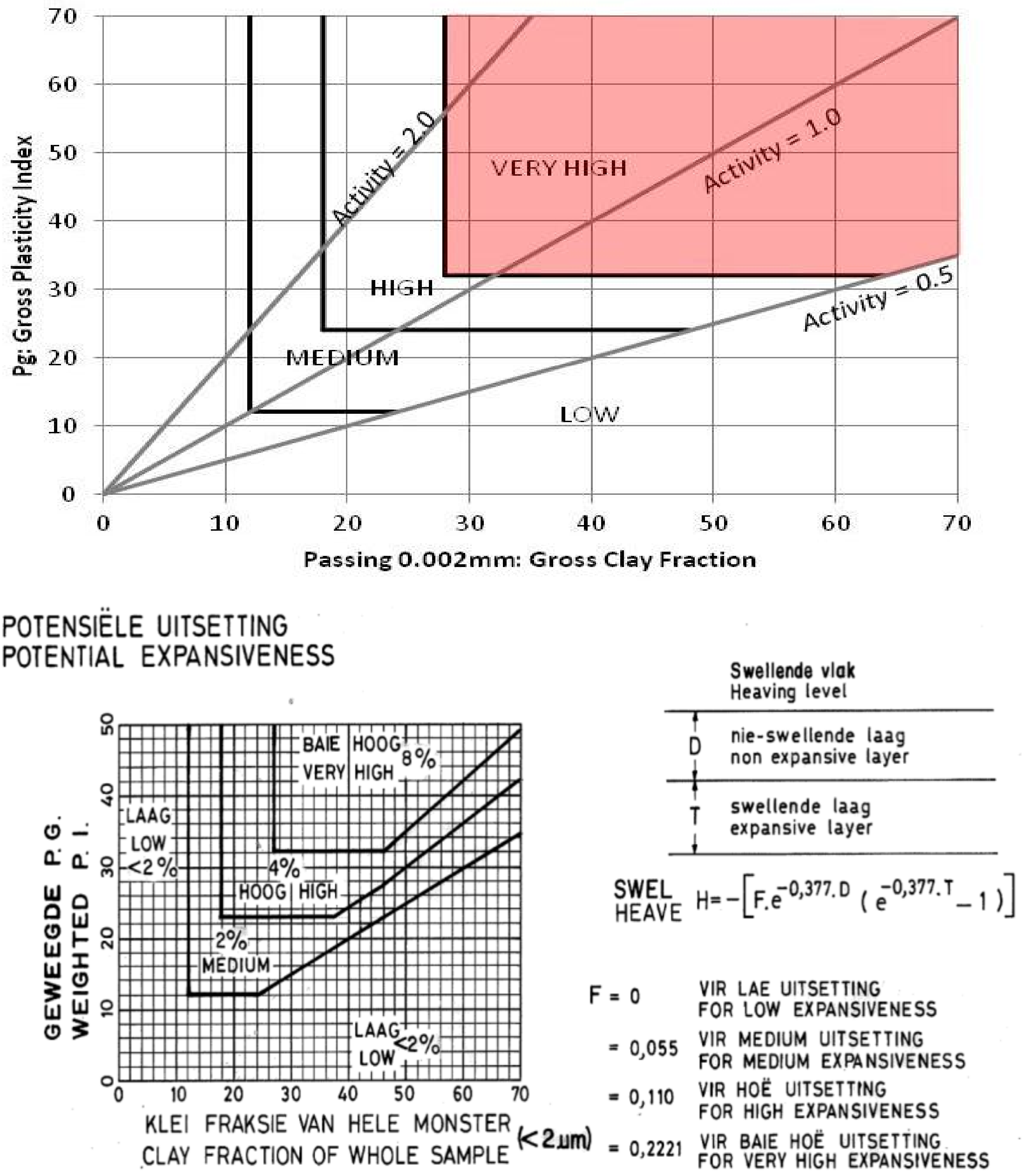


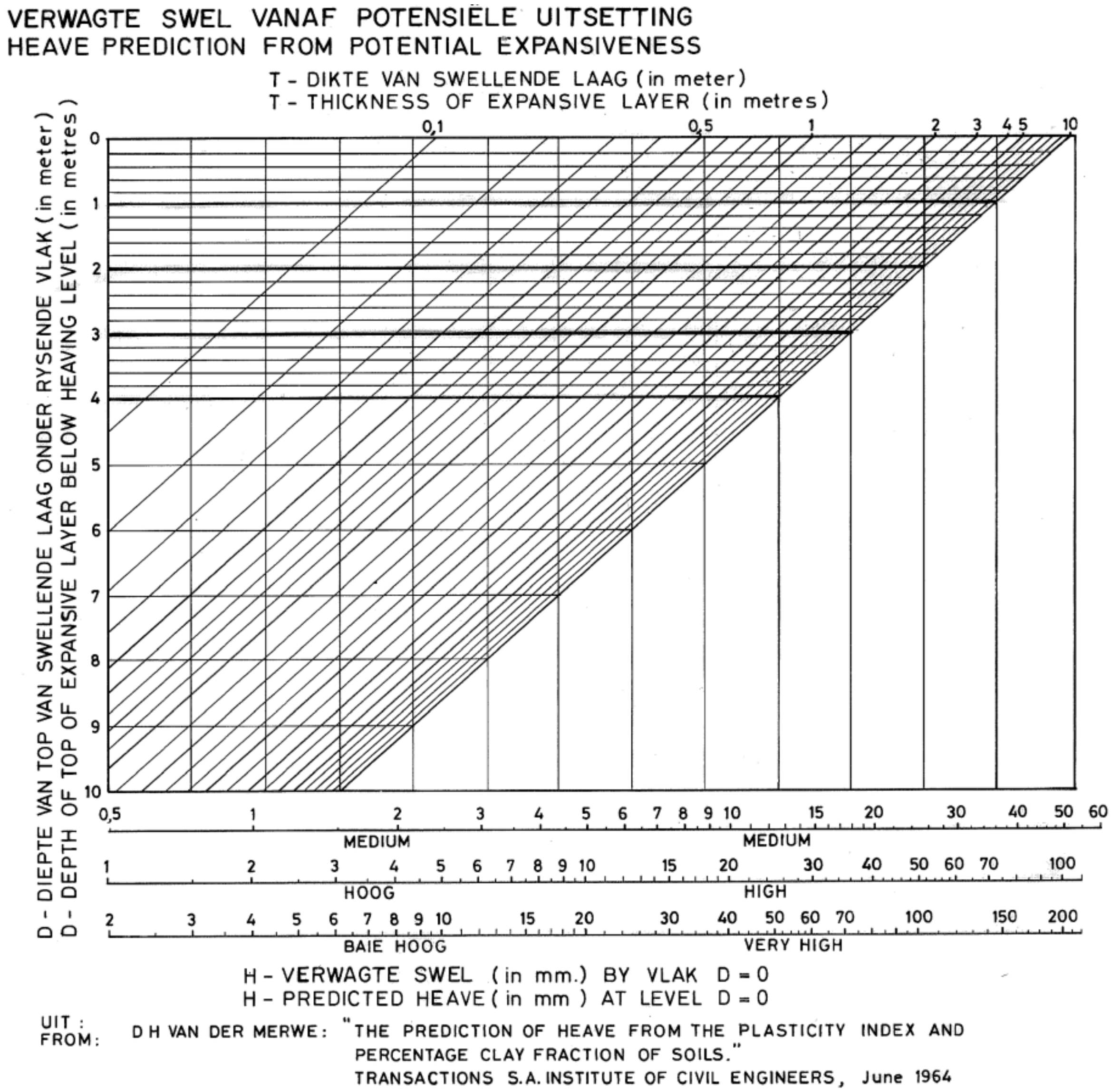
Figure 3.1: Simplified van der Merwe model

A typical example of this method is illustrated in APPENDIX E. As part of the electronic submission of this dissertation, a Microsoft Excel© document used to determine the total estimated heave potential based on the input parameter required by van der Merwe’s method, has been attached. The worksheet relies on Equation

3.1 above. Minor inconsistencies have been noted when comparing the values obtained by Equation 3.1 and the values obtained using the graph illustrated in Figure 3.3, especially when the particular layer has no overburden. A correction factor may be applied under certain conditions and layer thicknesses in order to achieve more comparable results, but this was not done as part of this study.



Figures 3.2a and b: 3.2a) Potential Expansiveness (adapted from Van Der Merwe's 1964 article) 3.2b) as modified by Williams and Donaldson (1980)


VERWAGTE SWEL VANAF POTENSIËLE UITSETTING
HEAVE PREDICTION FROM POTENTIAL EXPANSIVENESS

UIT :
FROM: D H VAN DER MERWE: "THE PREDICTION OF HEAVE FROM THE PLASTICITY INDEX AND PERCENTAGE CLAY FRACTION OF SOILS."
TRANSACTIONS S.A. INSTITUTE OF CIVIL ENGINEERS, June 1964

Figure 3.3: Predicted Heave (in millimetres). Taken from DH van der Merwe: "The prediction of heave from the plasticity index and percentage clay fraction of soils." Prepared by Soillab (Pty) Ltd.

3.3 Shortcomings in the Current Standard Testing Procedures for Assessing Heaving Clays in South Africa (Stott and Theron 2015)

Considering the model indicated in Figure 3.1, it is evident that the quality of input parameters is critical to ensure the quality of output parameters. The input parameters were therefore studied and reviewed.

Prior to this study and through the majority of the study period, the candidate was employed as a branch manager for a commercial soils laboratory. Throughout this period of employment, inconsistencies in testing were noted in comparing visually expansive materials (expansive cracking visual on study areas) and laboratory testing results. This was in part what inspired this study.

Typically the following laboratory tests are conducted in order to predict heave potential of materials encountered during geotechnical investigations for low-cost housing developments:

a) TMH1:1986 – Method A1(a) – The wet preparation and sieve analysis of gravel, sand and soil samples – replaced by SANS 3001 GR1;

b) TMH1:1986 – Method A2 – The determination of the liquid limit of soils by means of the flow curve method – replaced by SANS 3001 GR10, GR11 or GR12;

c) TMH1:1986 – Method A3 – The determination of the plastic limit and plasticity index of soils – replaced by SANS 3001 GR10, GR11 or GR12;

d) TMH1:1976 – Method A4 – The determination of the linear shrinkage of soils – replaced by SANS 3001 GR10, GR11 or GR12;

e) TMH1:1986 – Method A6 – The determination of the grain size distribution in soils by means of a hydrometer. Several commercial laboratories prefer the method stipulated within ASTM D422 to determine the finer fraction of soils by hydrometer analysis. SANS 3001 GR3 was proposed in 2011, but was not eagerly adopted by South African commercial laboratories, with none of the laboratories enjoying accreditation status for the test method currently. (*note: The ASTM D422 test method has officially been withdrawn in 2016);*

f) TMH1:1986 – Method A8 – The determination of the California Bearing Ratio of untreated soils and gravels – replaced by SANS 3001 GR40. A part of this test involves the measurement of swell by saturating a briquette in water for 4 days. This swell measurement is often seen as an indication of heave potential; and

g) TMH1:1986 – Method A17 – The determination of the moisture content of a soil sample – replaced by SANS 3001 GR20.

Most geotechnical engineering consultants would rely on these tests, with some requesting dynamic cone penetration tests as an additional measure, to determine the heave potential of the soils encountered in a study area. The author was involved in several such investigations, including an investigation into why light-weight single storey structures were failing in Botshabelo Section K and Luckhoff Combined School (Figure 3.4). Upon finalising the investigation and reviewing the laboratory results, analysis indicated that the heave potential for the entire study area was “low” according to van der Merwe’s method.

With structural failures indicating visually that the materials on site were without a doubt expansive in nature, this study was undertaken.

Figure 3.4: Expansive failure at a primary school in Luckhoff (taken by author, October 2014)

In examining the various test methods listed before, inconsistencies were noted and the repeatability of some tests proved problematic when analysing clayey samples. Large variations were recorded on the finer fractions of the samples with repetitive testing and unacceptable variances were recorded with the Atterberg limit states tests. In order to determine whether the inconsistencies were an in-house issue, it was decided to send duplicate samples to leading commercial laboratories. Two clay samples were split into seven smaller, representative, samples and sent to six commercial laboratories and tested in-house. The candidate approached several leading institutions for assistance and managed to obtain an interesting sample from Professor SW Jacobsz (University of Pretoria), a researcher previously involved in a similar study. Based on an article submitted to SAICE magazine in April 2008 titled “Are we getting what we pay for from geotechnical laboratories?”, Professor Jacobsz found similar discrepancies in test results obtained from four different laboratories. The sample obtained from the University of Pretoria was labelled “Steelpoort”. Due to the relatively small sample size available, “Steelpoort” was only split into four smaller representative samples.

The three clayey samples were taken to seven leading commercial laboratories (including the in-house laboratory used) to verify the in-house findings that indicated similar variance to that encountered in-house. This study is discussed in chapter 4.

With the variations in mind, the Central University of Technology research group, of which the candidate has been a member throughout the duration of this study, looked at the various test methods used individually, assessing each method in terms of expansive clays. An article by Stott and Theron (2015) resulted from this study which noted the following shortcomings to the current test methods commercially used:

a) None of the test methods take account of the suction and related forces of expanding clays;

b) TMH1 A1(a) deals with the sample preparation of soil samples which includes the steps “3.4 Boiling and washing” and “3.5 Drying and disintegration of fines”, which are often neglected by commercial laboratories to save time and therefore increase production. By not following these steps, internal testing indicated that an under-estimation of the Atterberg limits and clay fraction may be expected;

c) The SANS 3001 series replacing the TMH series offers three sample preparation techniques, GR1 for wet preparation, GR2 for dry preparation and GR5 for wet preparation at low temperatures. The existence of GR5 is recognition that normal procedures may not always be satisfactory (Stott and Theron 2015). There is, however, no indication as to who should make the decision on which sample preparation method should be utilised;

d) The GR5 preparation method is a very time consuming method requiring samples to be dried at 45 degrees Celsius, as opposed to the customary 105 to 110 degrees Celsius, which in the case of high plasticity clays could take anything from three days to a week in the oven. The longer oven duration will lead to lower production from a commercial point of view which makes it an unlikely method to be selected by commercial laboratories;

e) In test method TMH1:1986 Method A2, the determination of the liquid limit of a soil sample, it states that “It has been found that the Liquid Limit of certain materials is influenced by the mixing time.” A mixing time of 10 minutes was decided on. The flow-curve method was the method used when van der Merwe’s method was established, such a mixing time was not stipulated;

f) The TMH methods in 1986 made provision for a one-point liquid limit test using the Casagrande apparatus, a two-point method and a three-point method (the flow curve method). Invariably, commercial laboratories elected to perform the two-point method in preference to the flow curve method and to the fulfilment of accreditation status until the SANS 3001 series separated the liquid limit tests into three test methods: GR10 (one point); GR11 (two point) and GR12 (flow curve). To satisfy the requirements of accreditation, the one-point method is now preferred. During the 1960’s, when van der Merwe’s method was developed, the liquid limit test was done by the flow curve method;

g) A study, in which the candidate took part, published by Stott and Theron (2015) indicated that low plasticity clays showed less time dependence while high plasticity clays indicated large scatter while the plasticity index initially increased rapidly. The longer the mixing time, the more gradual the increase in plasticity index became and the scatter minimised;

h) Typically the liquid limit results obtained by the research group were higher than those obtained from commercial laboratories while the plastic limits were similar. The plasticity index of soils is the difference between liquid limit and plastic limit indicating that typically the plasticity index values obtained by the research group were higher than those of the commercial laboratories. Stott and Theron (2015) indicate that the difference was between 29% and 75%, which is vast; and

i) The test methods pertaining to linear shrinkage dictates that the troughs be immediately inserted into an oven at 105 to 110 degrees upon filling it with prepared soil paste. It was found that following this procedure led to bent and shattered samples, while air-drying before inserting the samples into the oven produced less bending, arching and shattering with linear shrinkage values twice as large as those produced by oven drying directly.

In summary it was found that sample preparation (wet preparation) was a key consideration. Determining the liquid limit of high plasticity should be handled differently than those with low plasticity. Stott and Theron (2015) proposed an amendment which essentially just means adding the expected required amount of water to the sample within the first half of the allotted mixing time and allowing the sample to soak up the moisture for 30 minutes prior to continuing the mixing process. The sample may not have reached equilibrium, but the values recorded would be acceptable in terms of what may be expected in the field.

Stott and Theron (2015) indicated that the linear shrinkage (Figure 3.6) of a soil sample may give an indication of heave potential where other methods fail to do so. The authors further noted that the current testing methods are acceptable for the use of road construction materials, which are predominantly granular, but attention should be given to the preparation and mixing time of the samples when it comes to potentially expansive materials.

A study by Stott and Theron (2016) indicates that the hydrometer method, which is almost exclusively used by commercial laboratories, is unreliable and not fit for use when analysing soils with high clay content. Using methylene blue to label clays, the study proves Savage’s (2007) assumption that clays are typically not dispersed sufficiently (Figures 2.7a and 2.7b).

Figure 3.5: Linear shrinkage tests on various samples (Taken from Stott and Theron, 2015)

3.4 A Review of the Current Testing Methods Used As Input Parameters

A review into the entire process followed from sampling to final analysis has identified several areas of concern as listed below:

3.4.1 Sampling techniques

Sampling techniques – Is the material sampled truly reflective of the material encountered? Often it was found that the material sampled on site is contaminated by adjacent soil layers. This typically occurs when the samples are excavated by backhoes or excavators. Excavations by hand typically offer more control, but that control needs to be exercised and is limited to depths of 1.5 meters.

3.4.2 Sampling preparation

This can be subdivided into two categories:

a.) Subdivision of the sample by means of a riffler or by coning and quartering to reduce the sample size adequately to conduct grading analysis, Atterberg limits and hydrometer analysis on a <u>representative</u> sample. If this is not done correctly the sample tested is not representative of the soil sampled; and

b.) Preparation for grading analysis and Atterberg limits by oven drying materials and boiling materials with water to break down hard lumps of clay. By subjecting soil samples to oven drying at temperatures exceeding 100° Celsius, we are exposing the samples to conditions that will not occur under natural conditions. Blight (2013) indicated that oven drying of samples at 105° Celsius had permanently dehydrated and modified the clay minerals or metallic hydroxides contained by the soil. This will have an impact on the clay minerals in that we are effectively changing the sample that we intend to test. Furthermore, lumps of clay that are not effectively broken up may be recorded as sand or gravel leading to an incorrect particle size analysis which will have further repercussions as the fines obtained from the grading analysis is typically used for hydrometer analysis and Atterberg limit states analysis. A compounding error will invariably lead to incorrect heave potential estimations.

3.4.3 Grading analysis on soil samples up to 75 µm

This test is reliant on material testers to be patient and follow the procedures stipulated in the various test methods. Often it is found that operator bias and impatience leads to incorrect particle size analysis which ultimately lead to a compounding error as explained in point 3.3.2b above.

3.4.4 Atterberg limits

Atterberg Limits – The current method generally used in South Africa (Casagrande cup) is widely believed to be subject to operator bias. The method is reliant on the discretion of the material tester. The test consists of three parts:

a.) Liquid Limit – Determined using the Casagrande cup, often referred to as the Liquid Limit Device. Forty-eight grams (48g) of material (fines typically passing 0.425mm) is weighed out and placed in a porcelain mixing bowl (often steel bowls are used in the industry) and mixed with de-ionized or distilled water in increased quantities until the tester believes that the liquid limit has been achieved. Three-quarters of the material is then placed in the Casagrande cup and a groove is cut through the middle with a grooving tool to certain specifications. The Casagrande cup is then tapped at two taps per second until the grove closes to such an extent that ten millimetres of the sides touch. If the amount of taps required achieving this is between twenty-two (22) and twenty-eight (28) blows, the liquid limit can be determined based on factors tabled as part of the test method. Should closure be achieved in less than twenty-two blows, the material is deemed too wet and mixed further until the material is deemed suitable. This effectively lengthens the mixing time and may impact the liquid limit achieved as the limit state increases with mixing time. A specified mixing time of ten (10) minutes may not be enough as water movement on molecular scale in active clays may take longer. This will have a further impact on the plasticity index as can be seen in Equation 2.2, section 2.3.1 and Figure 3.6.

$$I_P = W_L - W_P \tag{2.2}$$

Where:

I_P denotes plasticity index (often denoted as PI, as indicated in Figure 3.6)

W_L denotes liquid limit

W_P denotes plastic limit

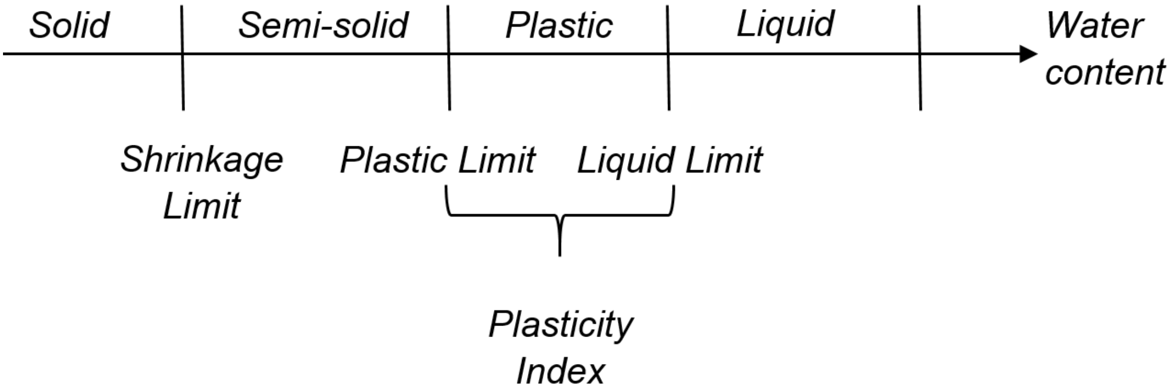


Figure 3.6: Atterberg limit states of soil (simplified from Casagrande, 1948)

b) Plastic Limit – Determined by rolling a thread approximately three millimetres in diameter. Should the material be too dry, the thread will crumble before the target diameter is achieved. Should the material be too wet, the diameter will be achieved without any crumbling visible. Ideally the thread should start to crumble near achieving three millimetres. Operator bias is obviously a factor here. The pressure exerted while rolling the thread will have an impact on the final result achieved.

c) Linear Shrinkage – Typically this method is used as a check value to compare with the plasticity index determined by the difference between the liquid limit and the plastic limit. In the current code of the South African context, it serves only to judge the suitability for materials to be used as wearing course on roads. It does however prove to be a test that is repeatable and generally gives acceptable variation in values. One of the problems with linear shrinkage is that clayey materials tend to arch upwards making it impossible to accurately determine the actual linear shrinkage achieved; the other is the drying rate. During the research conducted for this dissertation, it was found that air drying the material prior to inserting the linear shrinkage troughs into the oven greatly solves this problem, although it is a deviation from the standard testing methods.

3.4.5 Hydrometer analysis

Hydrometer analysis depends primarily on Stokes' law to determine the distribution of finer fractions based on sedimentation. The problem herein lies that Stokes' law assumes spherical particles. Clay is known to be flaky and have large surface areas. The soil sample is sieved to obtain material passing the 2millimetre sieve (often a 0.425milimetre sieve is used). This material is then dispersed by adding a dispersive agent and mixing it thoroughly with a paddle rotating at speeds ranging from low to high frequencies. Research done by Stott and Theron (2015) as part of the research cluster formed at the Central University of Technology found that it is almost impossible to completely disperse the clayey materials. Clayey particles attach themselves to silty or sandy particles. This amalgamation of material is then only partly dispersed leading to clayey fractions being assessed as sand or silt due to their adhesion to larger fractions. This can be verified by adding methylene blue to the already dispersed soil mixture. Clay will absorb the methylene blue and be coloured blue making it easy to identify under a microscope.

Shortcomings are noted in the test methods themselves with TMH 1:1986 Method A6 stating in the prelude to the method that “This method does not give absolute results, but gives data which are comparable and consistent if the method is followed in detail.” The paper by Stott and Theron (2015) refutes this assumption; the results aren’t comparable and consistent, even if the procedure is strictly followed. Furthermore in section 5.1 it states “If absolute results are required the method given in ASTM Designation D442 should be used.” Initial analysis using this method compared to ASTM D442 proved that typically TMH1 1:1986 Method A6 yields a lower estimation of clay particles (measured as 0.005 millimetres and smaller). The ASTM D442 method was officially withdrawn in 2016. Various attempts to utilise the method stated in SANS 3001:GR3 proved unsuccessful to find results that are in-line with TMH1 A6 and ASTM D422. It is suspected that some of the formulae provided with the test method may be incorrect.

It is important to note that any errors made as part of the input parameters will directly impact the final heave potential estimation. Based on the above it is evident that obtaining reliable heave potential results on the basis of the inputs obtained by the specified test methods is unlikely.

3.5 The Importance of Van Der Merwe’s Method

From its inception, van der Merwe’s method proved to be popular amongst geotechnical engineers and structural engineers as it gave them a cost effective alternative to expensive tests such as the double oedometer test. An examination of geotechnical investigation reports dated from 1970 to 2016 shows that this method has been trusted to aid in engineering decisions for more than 40 years.

In geotechnical engineering, it has become common practice to use van der Merwe’s method to determine whether expansive soils should be a geotechnical consideration during the design stage. For larger structures there is usually scope in terms of time and money to conduct more expensive time consuming tests, but with low cost housing no scope exists for such alternatives and van der Merwe's method is relied on almost entirely for the most fundamental design problem – designing foundations for structures that should have a design life of sixty years for most houses.

In 2014, the candidate was involved with a geotechnical investigation for proposed additional buildings to Luckhoff Combined School. Several desiccated patterns were identified on the soil surfaced which is often associated with heaving clays. Structural failure evident on one of the structures (Figure 3.4) verified that expansive clays were an obvious concern of site. The candidate compiled the final geotechnical report on the investigation and was surprised to learn that all the laboratory analysis indicated that heaving clays are not a geotechnical constraint for this site. (Extracts of geotechnical report on Luckhoff Combined School are attached as Appendix F).

A previous geotechnical investigation on Luckhoff Combined School indicated "*low expansiveness*", and the foundations were designed with that consideration in mind. Several examples of the same exist throughout the Free State Province, notably in Botshabelo, Thaba Nchu and Bloemfontein. Such examples have also been identified in Kimberley and other towns in the Northern Cape Province.

In order to determine whether van der Merwe's method is reliable, the method was broken and reviewed in parts.

3.6 Possible Shortcomings in Van Der Merwe's Method

After assessing van der Merwe's method (1964), Paige-Green suggests that the results must be interpreted with caution (Paige-Green, 2004). After almost 50 years it is still used and still widely preferred.

Notable shortcomings in van der Merwe's method are the fact that it relies on accurate plasticity index determination as well as accurate clay fraction determination while not considering the actual (initial) moisture content of the sampled materials. What is especially concerning with regards to the clay fraction is the method used to determine it. The formula used during the analysis of the hydrometer test assumes spherical particles (Stoke’s Law). Clays form agglomerations with other clay, silt and sand particles that form significantly larger particles that are miss-assessed as silt or sand particles. Stott and Theron (2016) published findings of a study, to which the candidate contributed, which used methylene blue to mark clay agglomerations that did not disperse well. This study proved that a significant amount of stirring with dispersive agents (more so than specified in the test methods) did not dissolve the agglomerations formed by clay particles. Underestimations of clay fractions are therefore likely when conducting hydrometer tests on clayey materials.

Van der Merwe provided four categories: low, medium, high and very high (Figure 3.2a). The problem is that each of the other categories shares a boundary with low which means that a small margin of error in the determination of the plasticity index and the clay fraction can lead to an extensively large difference in heave potential. Williams and Donaldson (1980) attempted to address this in a paper (Figure 3.2b), but the margin of error remains very small.

A study made by Savage (2007) indicates that the ratio of Liquid Limit to Plastic Limit indicates the type of clay present in a soil. If the clay fraction can then be accurately estimated without hydrometer analysis, it will at least rule out one of the problem areas associated with the prediction of heave potential. In order to achieve this, Savage has made two suggestions for improving the van der Merwe method. The first uses Skempton's Activity formula to estimate clay fraction, the second is a mathematical formulation of swell potential, giving curves of a similar form to van der Merwe's.

Reducing the error expected from hydrometer analysis by eliminating it from the required tests to be performed may be the first constructive step towards a more accurate estimation of heave potential; however doubt still exists in the methods used for the determination of the plasticity index of a soil sample. Perhaps looking at the British Standards, specifically at the cone penetration device, can solve the uncertainty involved with determining the liquid limit. The cone penetration device is less subject to operator bias, limiting a prominent part of the uncertainty in determining the liquid limit of a soil sample.

Van der Merwe's method can be broken into two parts:

a) Classification – based on the plasticity index of the whole sample and the clay fraction (percentage smaller than 2µm in diameter) the heave potential of the sample is determined to be either "low", "medium", "high" or "very high".

b) Quantification – based on the layer thickness, overburden on the expansive layer and the classification the expected surface heave is calculated using an empirical formula or an empirical chart.

3.7 Savage's Method (Savage 2007)

In his paper dated July 2007, Savage noted Atterberg limit states as a means to estimate the clay minerals present in expansive soils based on the ratios of liquid limit to plasticity index and plasticity index to plastic limit. Because the plasticity index is a function of the liquid limit and the plastic limit, the ratio of liquid limit to plastic limit may be accepted as a clay type indicator, termed R and referred to as "plasticity ratio".

With the values of R increasing from kaolinite (1.50) to montmorillonite (6.00), it is evident that the value R increases with the potential expansiveness of the minerals. Kaolinite is not considered to be expansive while montmorillonite is considered to be highly expansive. Based on Skempton's activity formula, Savage correlated his "plasticity ratio" to relate to Skempton's activity. By relating "plasticity ratio" to activity one is able to determine the percentage material smaller than 2µm, making the hydrometer method to determine the clay fraction of the soil sample redundant. Savage further formulated an alternative to van der Merwe's potential expansiveness graph in which swell potential is defined as K where $(P_{0.002} - 0{,}73K)(Pg - 0{,}16P_{0.002}\ K^{0{,}4}) - K = 0$. K values smaller than or equals to 16 is considered to be of low swell potential while values between 16 and 27 are considered to be of medium swell potential. K values in excess of 27 and 37 are considered to be of high and very high swell potential respectively.

According to Savage, hydrometer analysis may prove to be inaccurate do to several reasons including:

a) Stoke's law assuming spherical particles where clays typically are flaky with large surface areas;
b) De-flocculation of clays is seldom fully completed at the time of testing;
c) Clay particles are partially carried down by larger particles; and
d) A relative density of 2.65 is assumed for all particles, which may not be true.

In conclusion, Savage argues that it is more reasonable to rely on Atterberg limit and clay activity in determining swell potential. It should be noted that Savage's article was primarily intended for the use of road construction materials; it is however considered to be relevant to this study.

3.8 Discussion

Most commercial civil engineering soils laboratories in South Africa regard *foundation indicators* as the reference test to determine heave potential where the scope is insufficient for double oedometer tests or tensiometer tests. According to commercial laboratories, most geotechnical practitioners utilise van der Merwe's method to determine heave potential and total estimated heave.

A review into the input parameters to van der Merwe's method indicates that the output parameters to van der Merwe's method will always be questionable for expansive soils. The method relies on several test methods that were optimised and created for the testing of road construction projects.

The new SANS 3001 test methods have several clauses allowing laboratories to select the most appropriate method for the soil to be tested, but it is unlikely that a commercial laboratory will deliberately choose to conduct a more cumbersome method at the cost of production.

In-house testing indicated that sample preparation is probably the most important part of any test conducted on expansive soils. At times it is very challenging to break down the lumps of clay present in soil samples. Dispersive agents are not always very effective at breaking up clay lumps; clay lumps are therefore often misrepresented as sand or silt particles.

In-house testing at Roadlab Bloemfontein proved to produce more questions than answers when analysing clay samples. In order to clarify the internal findings as part of a problem finding process, three clay samples were sent to some of the leading national commercial laboratories in terms of reputational quality. The results and findings thereof are contained in Chapter 4.

CHAPTER 4: INTER-LABORATORY TESTING ON THE INPUT PARAMETERS USED TO DETERMINE HEAVE POTENTIAL USING VAN DER MERWE'S EMPIRICAL METHOD

4.1 Introduction

With prior laboratory testing and internal testing done at Roadlab Bloemfontein and Roadlab Germiston indicating that repeatability is typically within acceptable limits when analysing road construction materials, it was found that repeatability on test samples became more problematic with a decrease in mean fraction size and an increase in plasticity index. Clayey materials were found to have large discrepancies with repetitive testing.

In order to verify the findings made by internal testing, three samples of clayey materials were sampled and sent to various accredited soils laboratories. The laboratories were selected based on their reputation as reputable soils laboratories, accreditation status and locality.

This chapter focuses on the input parameters used by empirical methods as mentioned above and aims to establish the reliability of the test results received from seven of the premier commercial laboratories in South Africa.

4.2 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 can be summarised as using sedimentation and Stoke's law to determine the finer fractions of a soil sample.

4.2.1 Grading analysis

Two clayey samples were submitted to seven commercial laboratories, while a third was only submitted to four commercial laboratories due to a limited amount of available sample. Generally, the particle size analysis compared favourably between the various laboratories for apertures of 75.0mm to 2.0mm, where it is believed wet preparation may have influenced the finer fractions. Internal testing has indicated that sample preparation has a major impact in assessing the percentage of finer fractions. Where wet preparation was undertaken, the percentage fines recorded were considerably higher than when only dry preparation was undertaken.

Materials finer than 0.075mm were analysed using hydrometer analysis according to methods TMH1: 1986 Method A6 or ASTM D422. Most commercial laboratories prefer following the TMH1: 1986 Method A6, but revised the times of readings to reflect 0.002mm instead of 0.005mm. TMH1:1986 Method A6 has recently been replaced by SANS 3001: GR3, which combines features of both test methods.

Tables 4.1, 4.2 and 4.3 were compiled summarising the laboratory analysis obtained from the various commercial laboratories. These tables are visually represented in Figures 4.2 to 4.10.

The various laboratories are indicated as LAB1 to LAB7 with LAB2 testing according to ASTM D422 and TMH1 A6. The average values, standard deviation from mean, minimum and maximum values were determined for statistical purposes. The percentage difference is calculated as the difference between the minimum and maximum values.

Table 4.1: Particle Size Distribution - Steelpoort Sample

Steelpoort Sample	LAB 1	LAB 2 ASTM	LAB 2 TMH	LAB 3	LAB 4	Average	Standard Deviation	Minimum	Maximum	% Difference
75 mm	100	100	100	100	100	100,0	0,00	100	100	0.0%
63 mm	100	100	100	100	100	100,0	0,00	100	100	0.0%
53 mm	100	100	100	100	100	100,0	0,00	100	100	0.0%
37,5 mm	100	100	100	100	100	100,0	0,00	100	100	0.0%
26,5 mm	100	100	100	100	100	100,0	0,00	100	100	0.0%
19 mm	100	100	100	100	100	100,0	0,00	100	100	0.0%
13,2 mm	100	100	100	100	100	100,0	0,00	100	100	0.0%
9,5 mm	98	99	99	100	100	99,2	0,84	98	100	2.0%
6,7 mm	97	99	99	100	99	98,8	1,10	97	100	3.0%
4,75 mm	97	98	98	99	98	98,0	0,71	97	99	2.0%
2,36 mm	94	97	97	98	98	96,8	1,64	94	98	4.0%
2 mm	94	96	96	96	96	95,6	0,89	94	96	2.0%
1,18 mm	92	95	95	94	92	93,6	1,52	92	95	3.0%
0,425 mm	85	90	90	88	85	87,6	2,51	85	90	5.0%
0,3 mm	82	86	86	87	81	84,4	2,70	81	87	6.0%
0,25 mm	82	83	84	85	77	82,2	3,11	77	85	8.0%
0,15 mm	78	78	78	83	72	77,8	3,90	72	83	11.0%
0,075 mm	74	75	77	76	57	71,8	8,35	57	77	20.0%
0,005 mm	23	61	59	33	47	44,6	16,46	23	61	38.0%
0,002 mm	17	56	54	26	24	35,4	18,22	17	56	39.0%

Table 4.2: Particle Size Distribution - Brandwag Sample

Brandwag Sample	LAB 1	LAB 2 ASTM	LAB 3	LAB 4	LAB 5	LAB 6	LAB 7	Average	Standard Deviation	Minimum	Maximum	% Difference
75 mm	100	100	100	100	100	100	100	100,0	0,00	100	100	0.0%
63 mm	100	100	100	100	100	100	100	100,0	0,00	100	100	0.0%
53 mm	100	100	100	100	100	100	100	100,0	0,00	100	100	0.0%
37,5 mm	100	100	100	100	100	100	100	100,0	0,00	100	100	0.0%
26,5 mm	100	100	100	100	100	100	100	100,0	0,00	100	100	0.0%
19 mm	100	100	100	100	100	100	100	100,0	0,00	100	100	0.0%
13,2 mm	100	100	100	100	99	100	100	99,9	0,35	99	100	1.0%
9,5 mm	100	100	100	100	99	100	100	99,9	0,35	99	100	1.0%
6,7 mm	100	100	100	100	99	100	100	99,9	0,35	99	100	1.0%
4,75 mm	100	100	100	100	98	100	100	99,8	0,71	98	100	2.0%
2,36 mm	99	99	99	99	96	98	99	98,5	1,07	96	99	3.0%
2 mm	99	99	99	99	96	98	98	98,4	1,06	96	99	3.0%
1,18 mm	98	98	98	98	95	95	95	96,9	1,55	95	98	3.0%
0,425 mm	96	96	97	96	92	93	94	95,0	1,77	92	97	5.0%
0,3 mm	96	96	94	92	92	92	92	93,8	1,98	92	96	4.0%
0,25 mm	96	96	92	88	91	92	91	92,8	2,96	88	96	8.0%
0,15 mm	94	94	90	82	89	90	89	90,1	3,91	82	94	12.0%
0,075 mm	85	83	85	72	78	85	80	81,3	4,53	72	85	13.0%
0,005 mm	59	62	56	55	61	62	49	58,0	4,47	49	62	13.0%
0,002 mm	49	58	44	33	56	50	47	49,1	8,15	33	58	25.0%

Table 4.3: Particle Size Distribution - Botshabelo Sample

Botshabelo Sample	**LAB 1**	**LAB 2 ASTM**	**LAB 3**	**LAB 4**	**LAB 5**	**LAB 6**	**LAB 7**	**Average**	**Standard Deviation**	**Minimum**	**Maximum**	**% Difference**
75 mm	100	100	100	100	100	100	100	100,0	0,00	100	100	0.0%
63 mm	100	100	100	100	100	100	100	100,0	0,00	100	100	0.0%
53 mm	100	100	100	100	100	100	100	100,0	0,00	100	100	0.0%
37,5 mm	100	100	100	100	100	100	100	100,0	0,00	100	100	0.0%
26,5 mm	100	100	100	100	98	100	100	99,8	0,71	98	100	2.0%
19 mm	99	100	100	100	98	100	100	99,6	0,74	98	100	2.0%
13,2 mm	99	100	100	100	97	100	99	99,4	1,06	97	100	3.0%
9,5 mm	99	100	100	100	96	100	98	99,0	1,41	96	100	4.0%
6,7 mm	99	99	100	100	96	100	96	98,6	1,69	96	100	4.0%
4,75 mm	98	98	100	97	95	99	95	97,5	1,77	95	100	5.0%
2,36 mm	96	96	98	96	94	96	91	95,3	2,05	91	98	7.0%
2 mm	96	96	97	95	93	96	90	94,8	2,25	90	97	7.0%
1,18 mm	94	96	96	94	92	93	87	93,4	2,92	87	96	9.0%
0,425 mm	92	94	93	92	89	89	83	90,6	3,58	83	94	11.0%
0,3 mm	92	94	87	85	89	88	80	88,4	4,50	80	94	14.0%
0,25 mm	91	93	84	77	88	88	75	86,0	6,80	75	93	18.0%
0,15 mm	86	88	79	72	83	88	69	81,5	7,46	69	88	19.0%
0,075 mm	74	74	76	58	66	83	63	70,8	7,98	58	83	25.0%
0,005 mm	44	59	54	50	48	51	42	50,9	6,29	42	59	17.0%
0,002 mm	35	56	43	29	44	40	35	42,1	9,57	29	56	27.0%

Based on the particle size distribution of Steelpoort Sample, which was considered to be the most expansive of the three samples analysed, the standard deviations increased with the decrease in particle size as indicated in Figure 4.1.

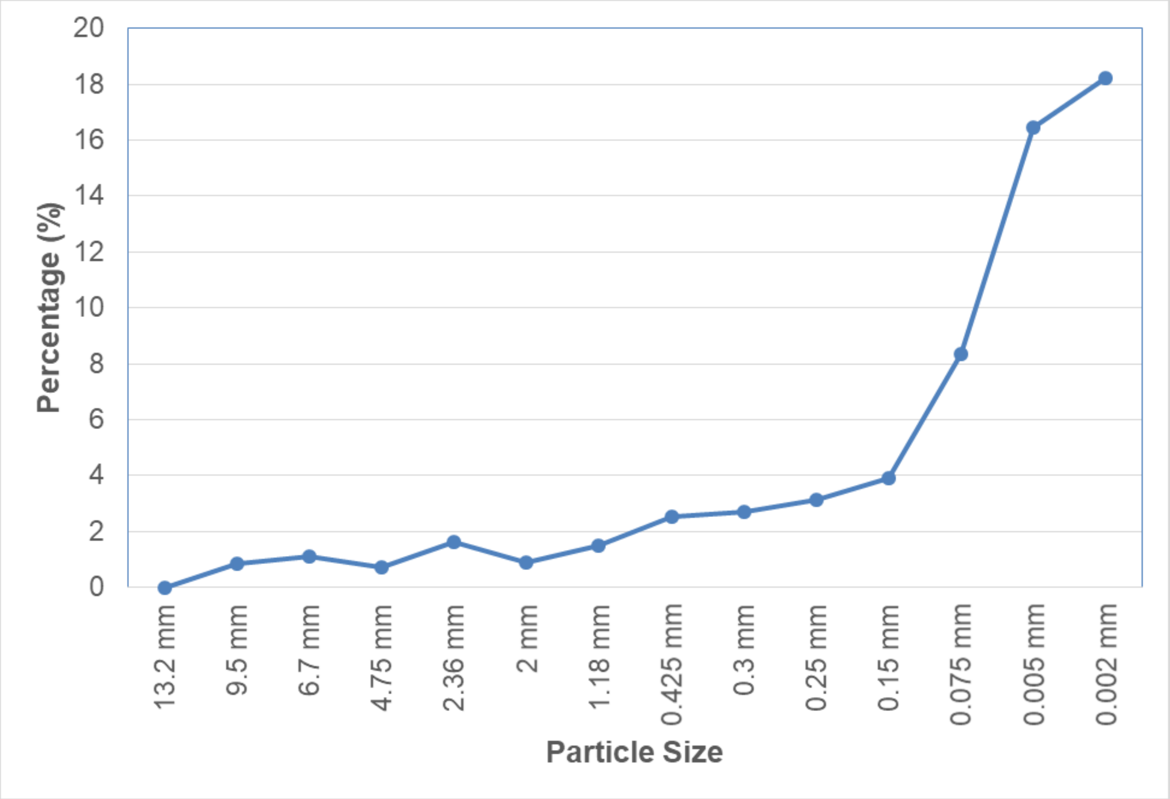


Figure 4.1: The increase of standard deviation with decrease in particle size for Steelpoort sample

The maximum differences recorded between the commercial laboratories are generally acceptable up until the 0.425mm fraction, where the differences between the maximum and minimum values increase from 5.0% at 0.425mm to 39.0% at 0.002mm (clay fraction). Sample preparation is expected to have had a significant impact on the fraction smaller than 1.18mm, with lumps of clay often assessed as sand or silt. Based on the findings made by Stott and Theron (2016), the high differences recorded at the fractions smaller than 0.075mm is expected to be due to the poor dispersion of clays and other factors influencing the reliability of hydrometer analysis.

The increase in standard deviation was similar for Brandwag and Botshabelo, although less exaggerated.

These results indicate that using the clay fraction obtained from commercial soils laboratories for the prediction of heave potential is questionable. Figures 4.2 to 4.10 indicate the particle size distribution obtained with specific emphasis on the fractions done by sieve analysis and the fractions determined by hydrometer analysis. It is evident that the differences are significant for hydrometer analysis.

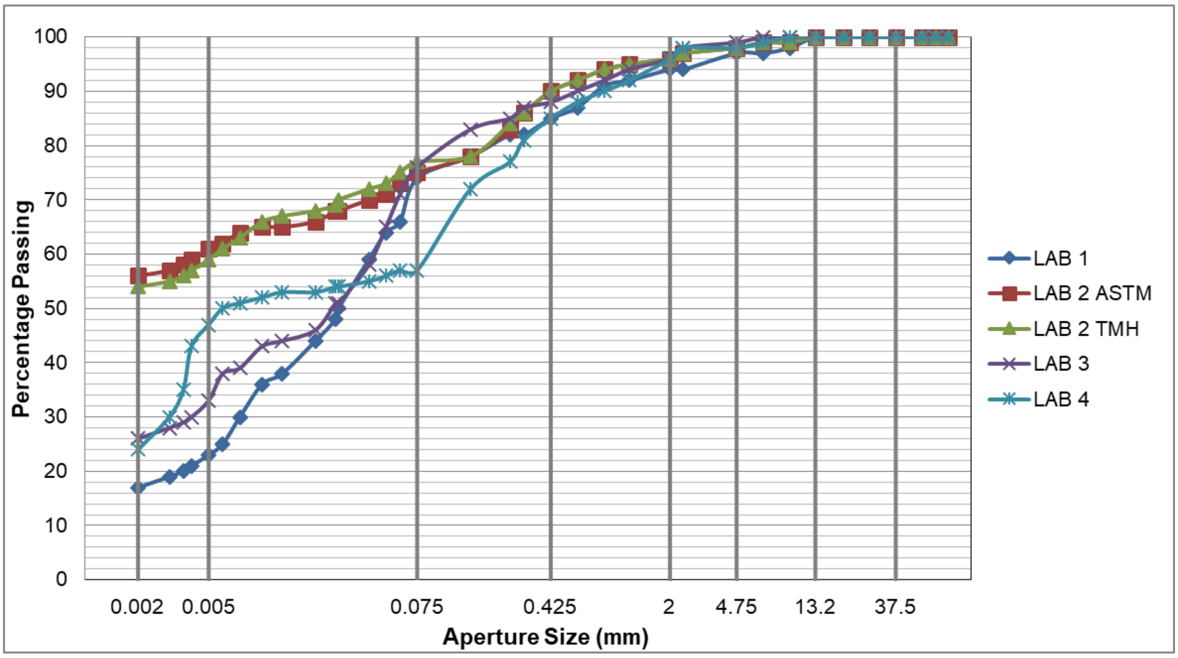


Figure 4.2: Particle size analysis of Steelpoort sample

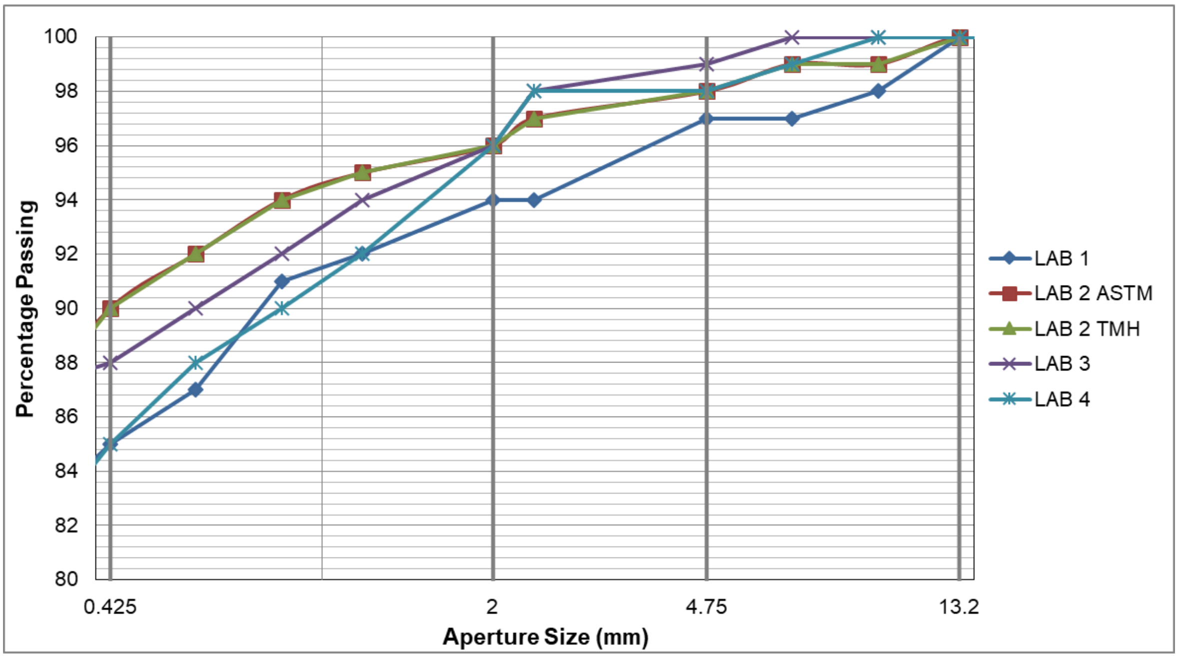


Figure 4.3: Particle size analysis of Steelpoort sample with specific focus on apertures 13.2mm through to 0.425mm

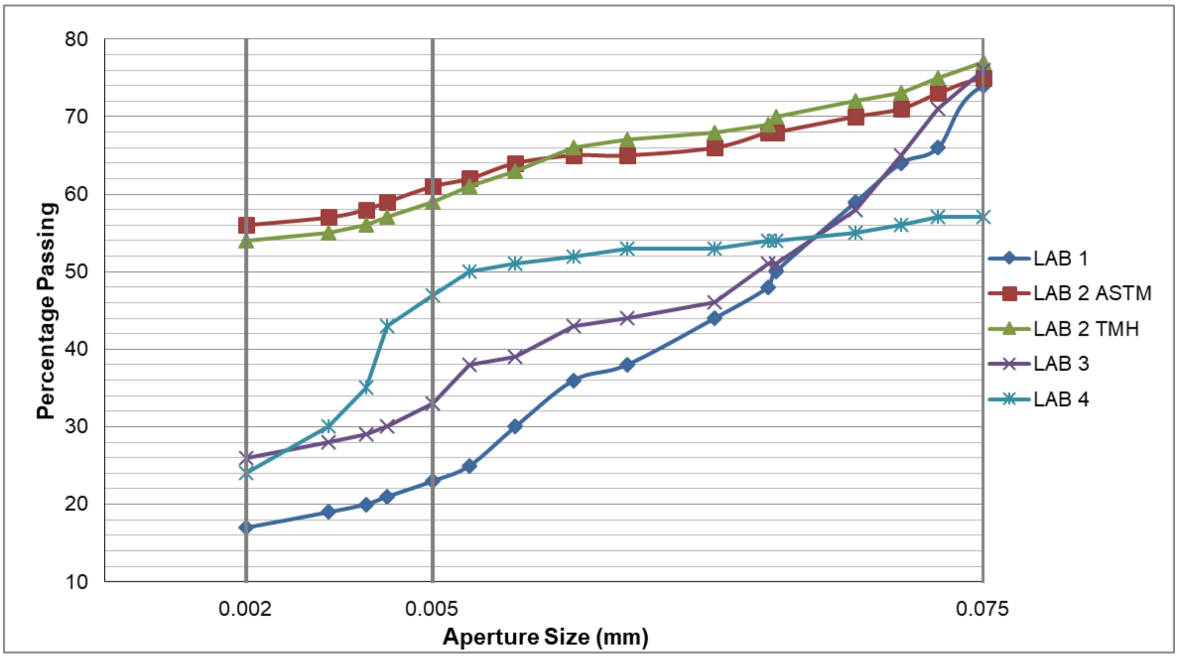


Figure 4.4: Particle size analysis of Steelpoort sample with specific focus on fraction determined by hydrometer

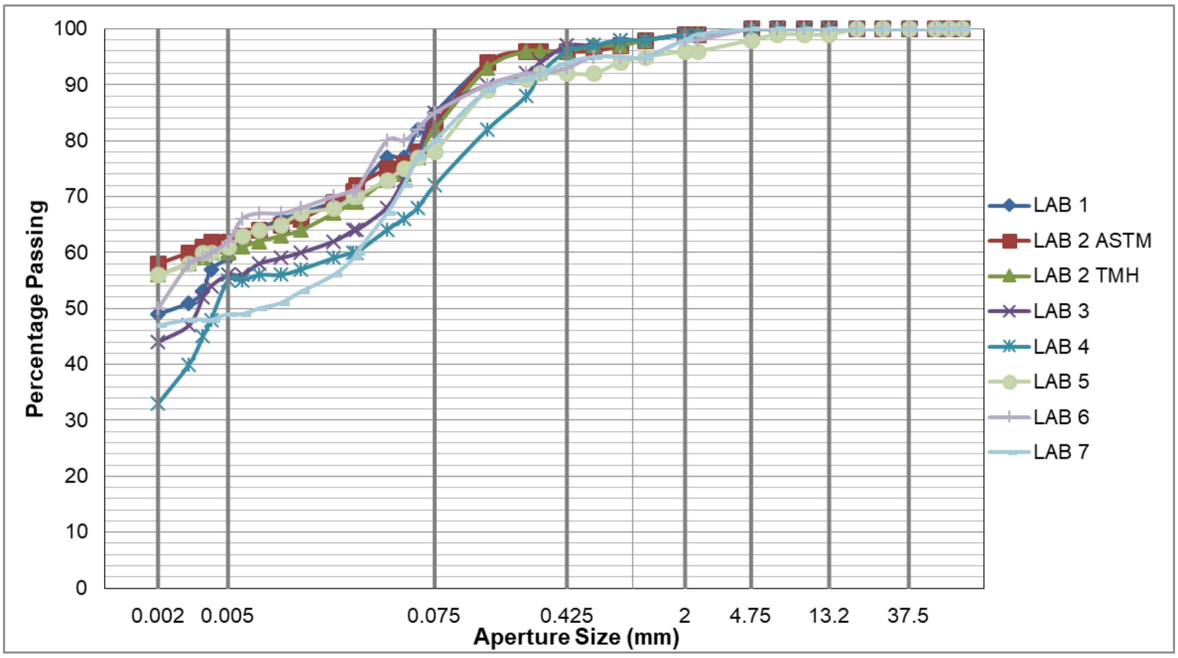


Figure 4.5: Particle size analysis of Brandwag sample

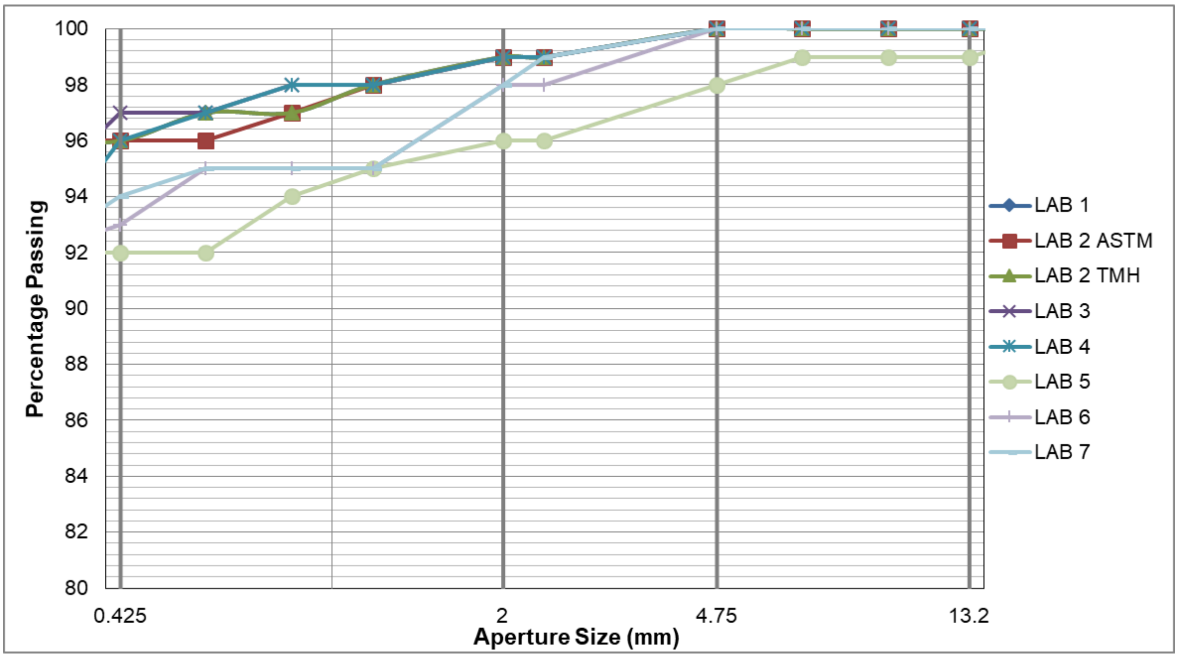


Figure 4.6: Particle size analysis of Brandwag sample with specific focus on apertures 13.2mm through to 0.425mm

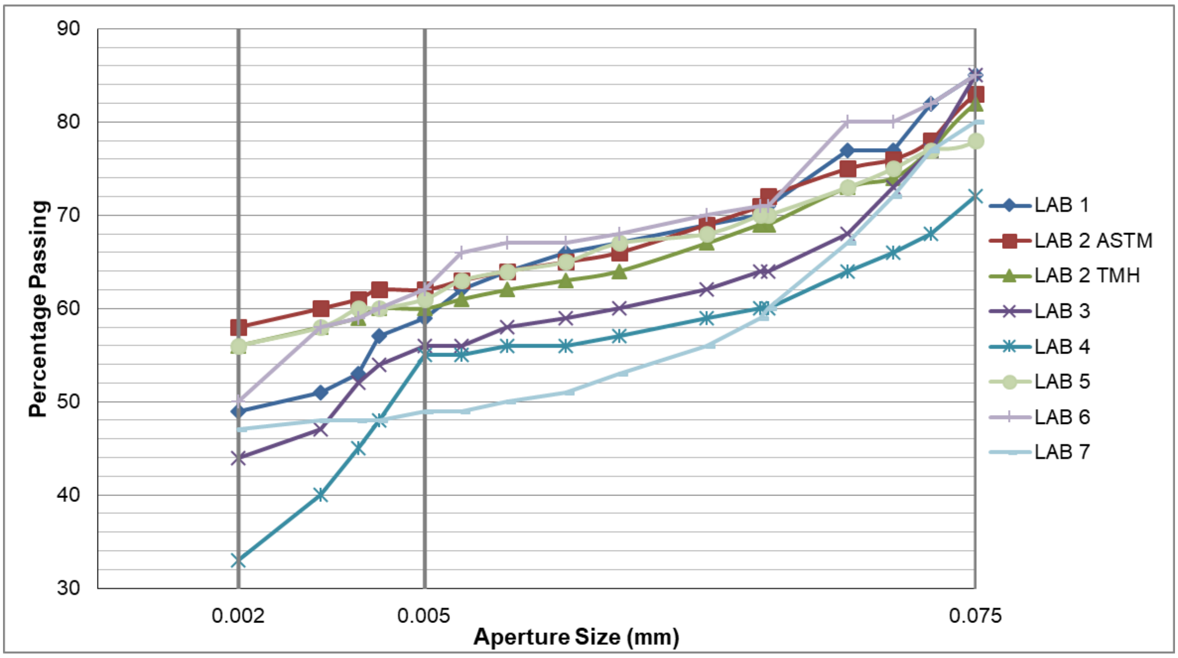


Figure 4.7: Particle size analysis of Brandwag sample with specific focus on fraction determined by hydrometer

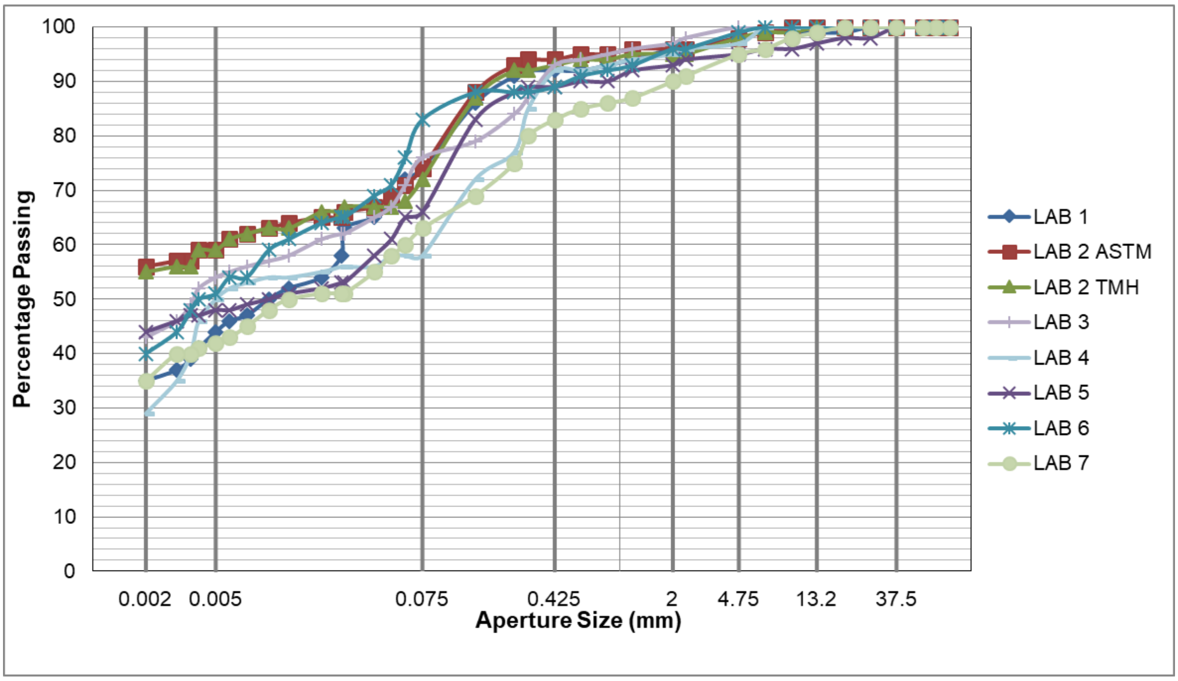


Figure 4.8: Particle size analysis of Botshabelo sample

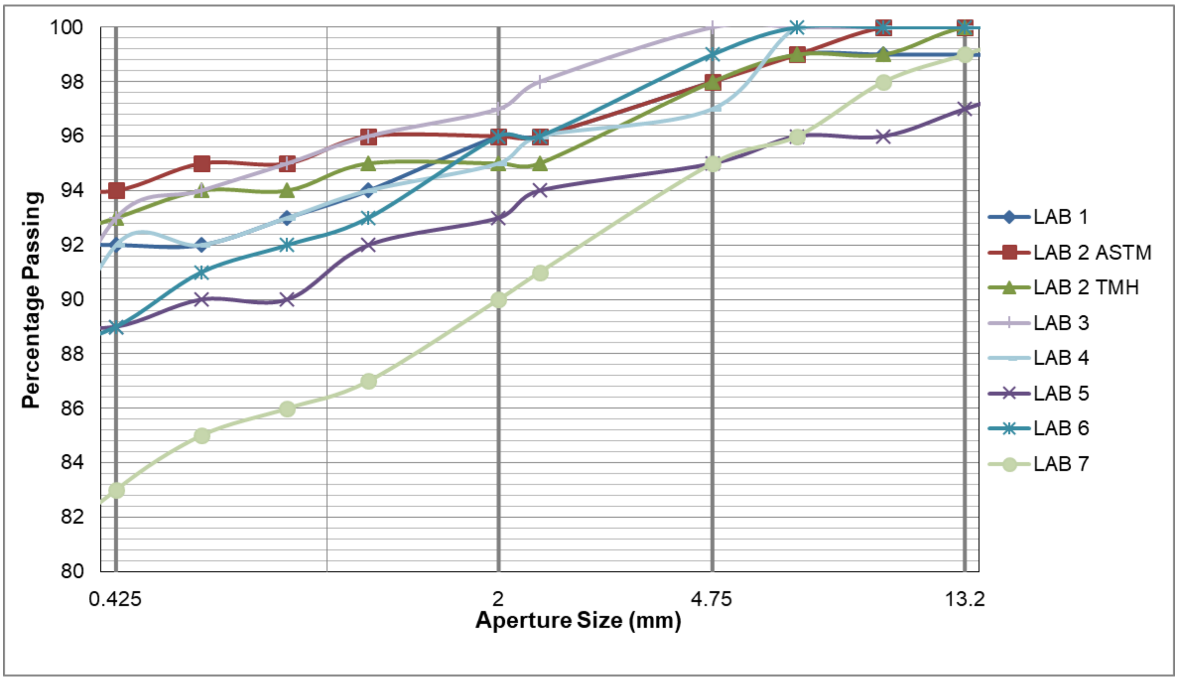


Figure 4.9: Particle size analysis of Botshabelo sample with specific focus on apertures 13.2mm through to 0.425mm

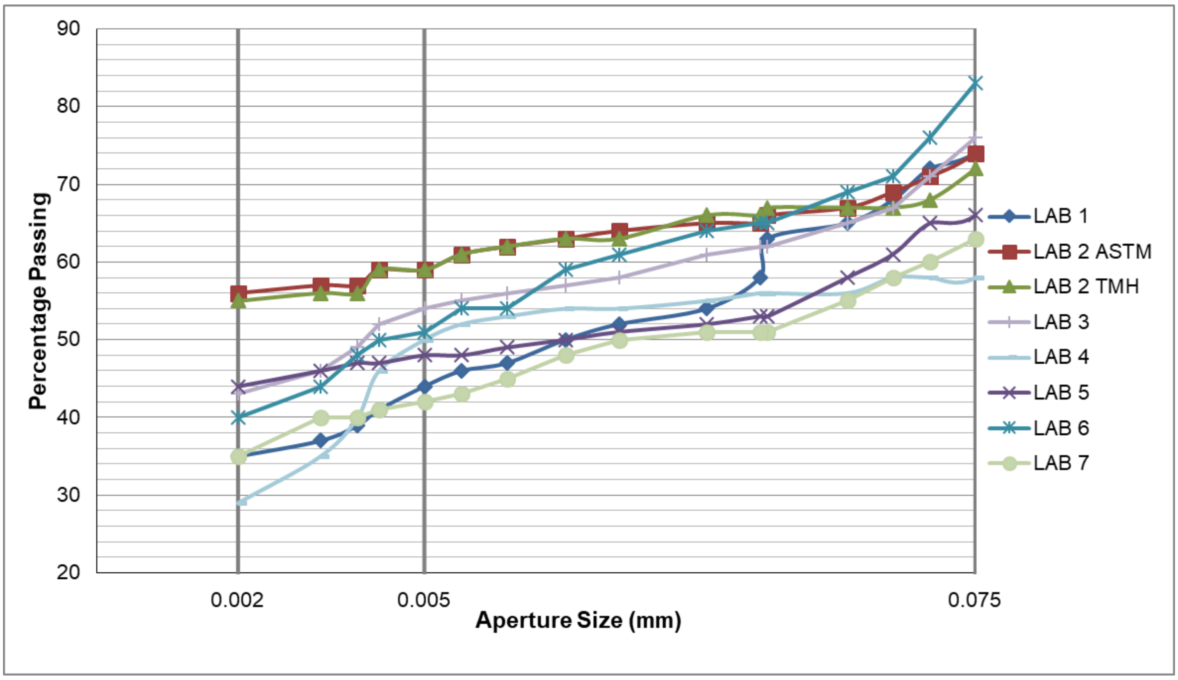


Figure 4.10: Particle size analysis of Botshabelo sample with specific focus on fraction determined by hydrometer

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.

Although variations were expected knowing that various laboratories focus meticulously on sample preparation while others might not, the results varied more than expected.

Due to the limited amount of remaining sample, only one sample was submitted to internal replicate analysis. One large sample was split into seven grading analysis samples and subjected to in-house sieve analysis and hydrometer analysis at a commercial laboratory in Bloemfontein. The samples were analysed using the same preparation, same testers and same equipment. Every effort was made to obtain results that are repetitive. The following tables and graphs indicate the internal analysis of Botshabelo sample. Generally, the standard deviation of percentage passing is acceptable up to the fraction passing 0.425mm. The finer fraction up to 0.002mm varies increasingly as the aperture decreases.

Table 4.4: Particle Size Distribution - Botshabelo Sample - Internal Replicate Testing

Botshabelo Sample	Test 1	Test 2	Test 3	Test 4	Test 5	Test 6	Test 7	Average	Standard Deviation	Minimum	Maximum	% Difference
75 mm	100	100	100	100	100	100	100	100,0	0,00	100	100	0.0%
63 mm	100	100	100	100	100	100	100	100,0	0,00	100	100	0.0%
53 mm	100	100	100	100	100	100	100	100,0	0,00	100	100	0.0%
37,5 mm	100	100	100	100	100	100	100	100,0	0,00	100	100	0.0%
26,5 mm	100	100	100	100	100	100	100	100,0	0,00	100	100	0.0%
19 mm	100	99	100	100	99	100	100	99,7	0,49	99	100	1.0%
13,2 mm	99	98	98	98	98	99	100	98,6	0,79	98	100	2.0%
9,5 mm	96	96	97	96	98	96	96	96,4	0,79	96	98	2.0%
6,7 mm	96	96	96	96	96	96	96	96,0	0,00	96	96	0.0%
4,75 mm	96	96	96	96	96	96	96	96,0	0,00	96	96	0.0%
2,36 mm	93	95	96	93	95	95	94	94,4	1,13	93	96	3.0%
2 mm	93	93	94	93	94	93	94	93,4	0,53	93	94	1.0%
1,18 mm	93	93	93	93	94	93	94	93,3	0,49	93	94	1.0%
0,425 mm	93	89	90	93	93	92	92	91,7	1,60	89	93	4.0%
0,3 mm	89	86	85	89	85	92	85	87,3	2,75	85	92	7.0%
0,25 mm	88	85	83	80	82	88	84	84,3	2,98	80	88	8.0%
0,15 mm	76	82	79	73	77	77	73	76,7	3,20	73	82	9.0%
0,075 mm	61	61	63	63	61	67	64	62,9	2,19	61	67	6.0%
0,005 mm	51	53	54	55	55	62	58	55,4	3,60	51	62	11.0%
0,002 mm	34	39	37	32	37	51	36	38,0	6,16	32	51	19.0%

The standard deviation obtained internally were generally lower compared to the duplicate testing done at various laboratories, but that was expected because the deviations due to different equipment and operators were limited. The deviations remained relatively high, despite every effort to obtain repetitive results. Figures 4.12 and 4.13 indicate the standard deviation increase with a decrease in fraction size for internal testing and duplicate testing respectively.

Generally, the standard deviations obtained on internal replicate testing were acceptable up to 0.075mm; the standard deviations increased significantly with hydrometer analysis despite repeating the same test with the same operator using the same equipment.

The difference between the minimum and maximum values obtained for the 0.002mm fraction by internal replicate testing was 19.0%, which is significant. Figure 4.11 indicates the difference it may make using van der Merwe’s method (1964) to predict heave potential. The misrepresentation of the clay fraction may mean the difference between expecting *zero* heave and expecting significant heave.

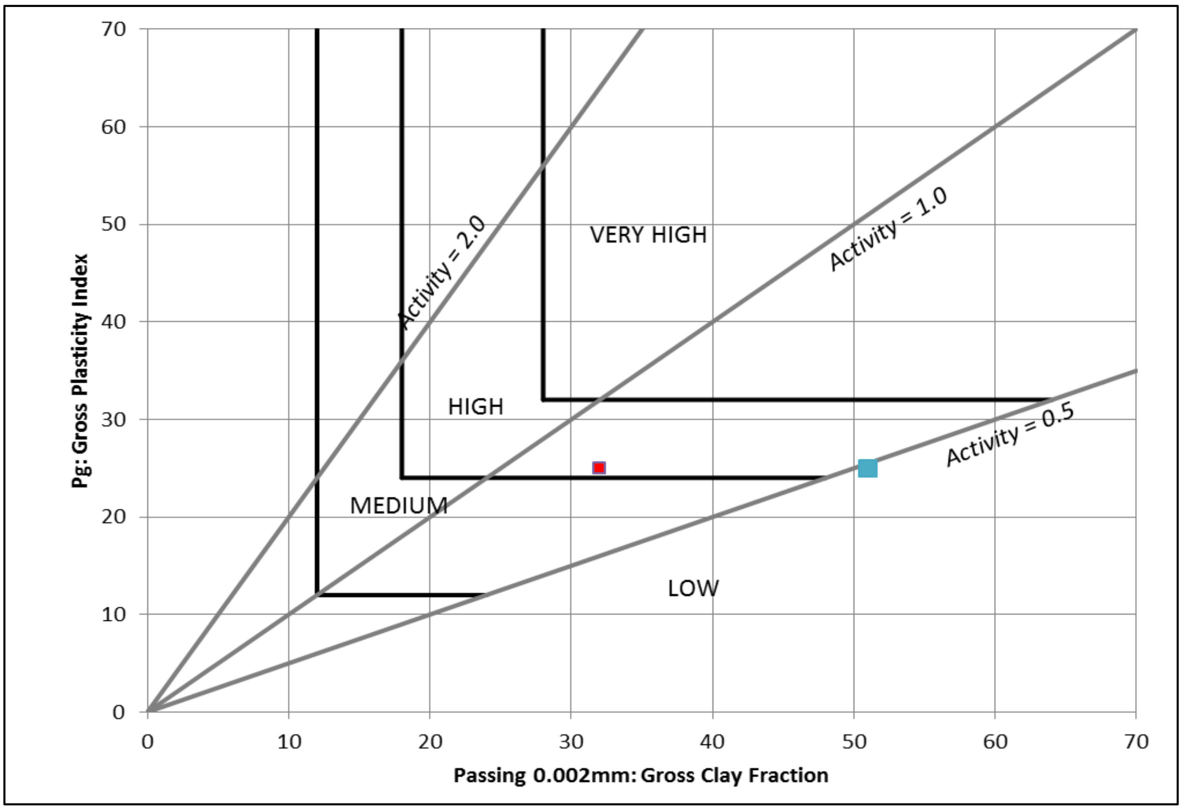


Figure 4.11: The difference 19% can make on heave prediction for a sample with a gross plasticity index of 24%

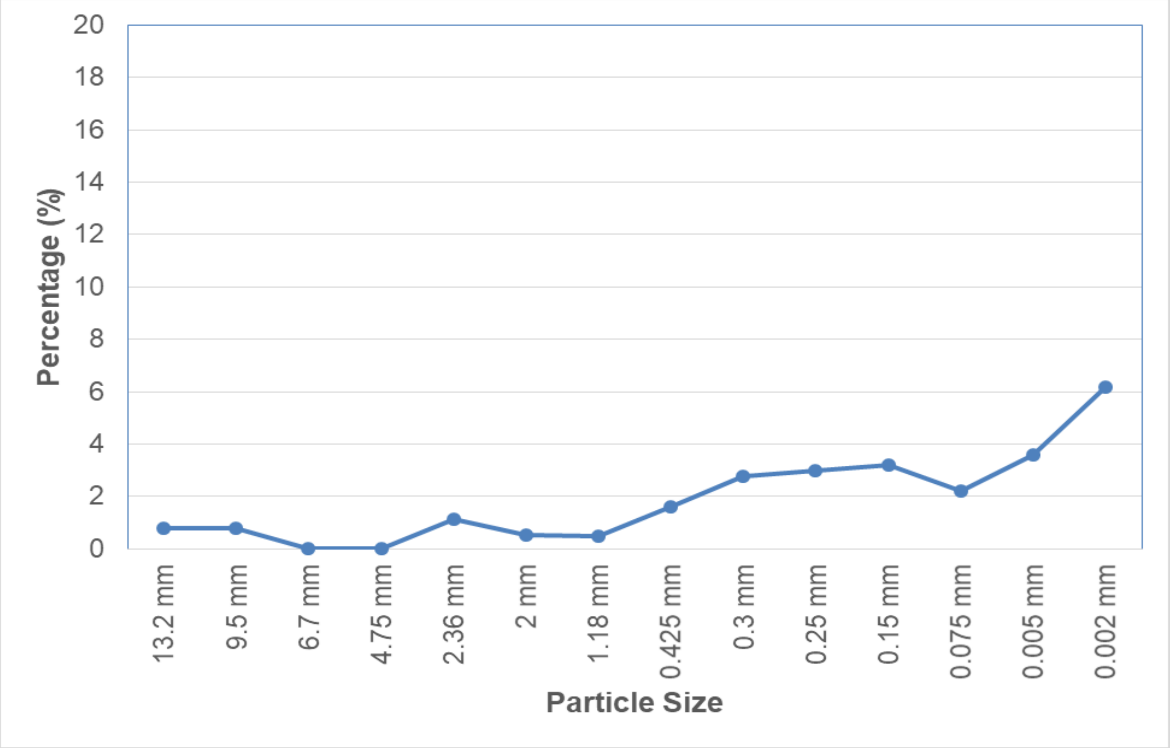


Figure 4.12: Increase of standard deviation with the decrease in fraction size on internal replicate testing done on Botshabelo sample.

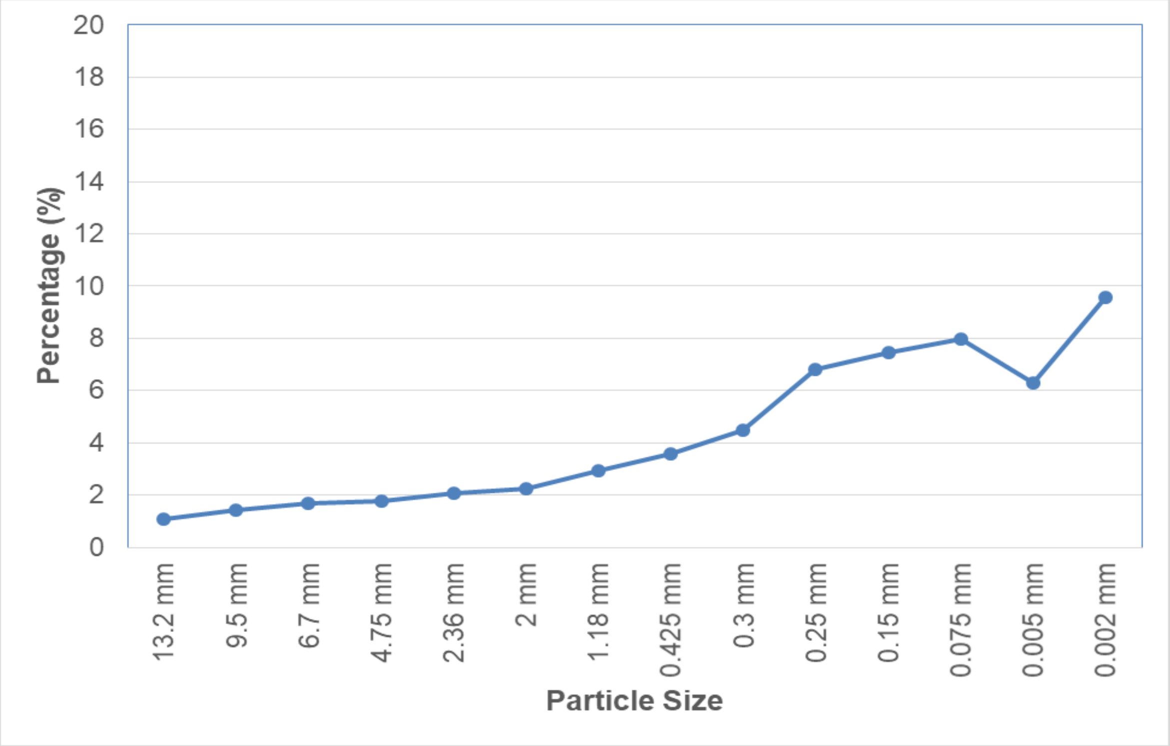


Figure 4.13: Increase of standard deviation with the decrease in fraction size on duplicate testing done on Botshabelo sample.

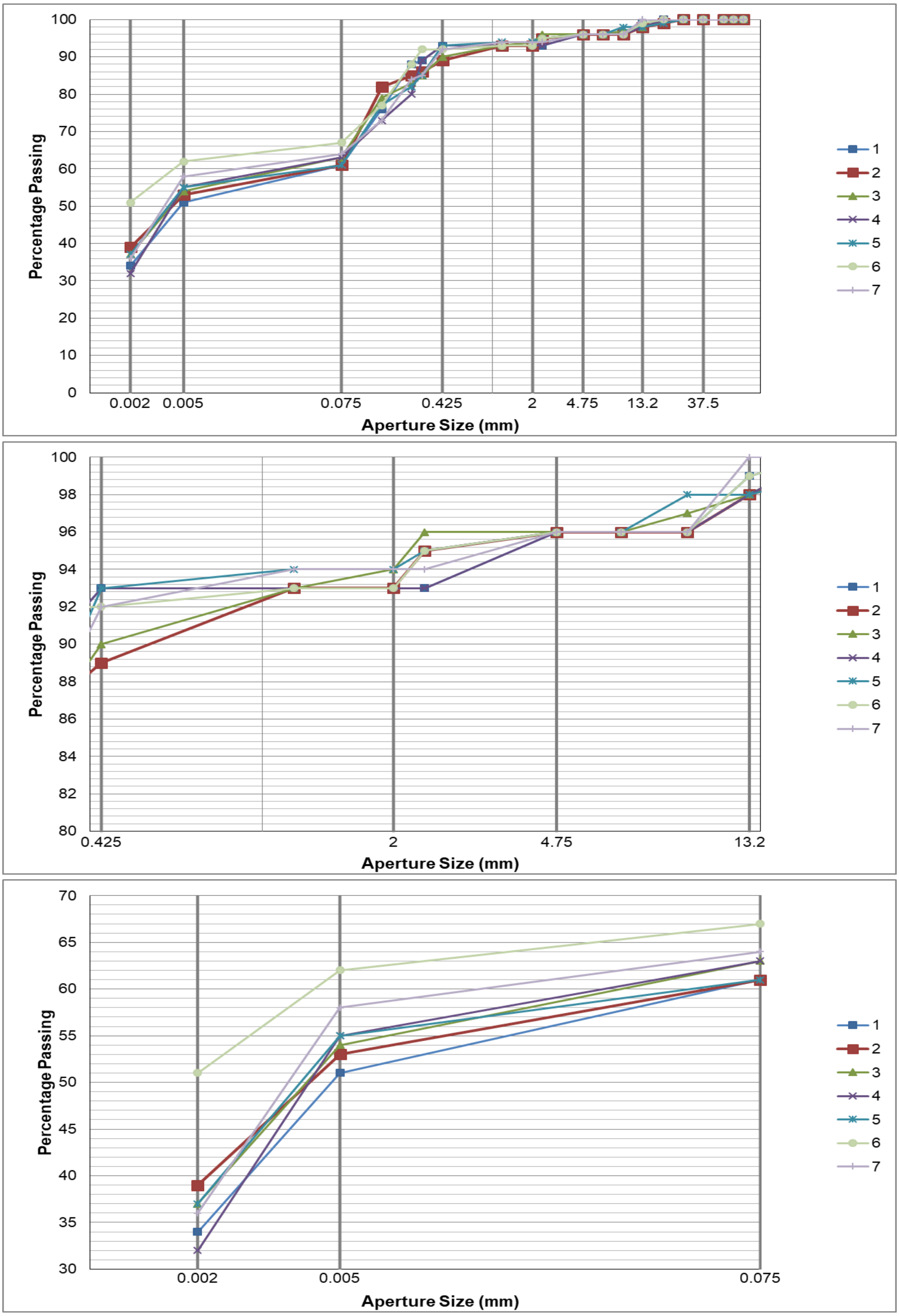


Figure 4.14: Particle size analysis of Botshabelo sample subjected to internal replicate testing

4.2.2 Hydrometer analysis

Hydrometer analysis is typically used to determine the fractions of the fines contained within the soil samples. This method is based on Stoke's law and is flawed due to the assumption under Stoke's law that all particles are spherical. Clay particles are flaky and often have large surface areas (Whitlow 2001). A paper by Savage, 2007, concludes that hydrometer accuracy is doubtful for several reasons: 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 (Savage 2007). Some revisions on the hydrometer tests, notably the SANS 3001 series, has made provision for various relative densities which are now calculated as part of test method, but it still does not account for the spherical particles assumption.

Savage further suggests looking at the Atterberg Limits to determine the clay fraction, more on this in section 3.6 and section 4.2.3.

The hydrometer analysis values and graphs obtained from Steelpoort, Brandwag and Botshabelo samples are included as part of section 4.2.1 in order to have a holistic view of the particle size analysis of these samples.

Typically the variance increased significantly with hydrometer analysis with 25% to 39% difference between the maximum and minimum values recorded on duplicate testing as indicated in Tables 4.1, 4.2 and 4.3. Internal replicate testing on Botshabelo indicated a difference of 19% (Table 4.4). Despite every effort to repeat the test with the same equipment and operator under the same conditions, the variance remained significant.

Figures 4.4, 4.7 and 4.10 indicate the variance obtained from different commercial laboratories on a sample divided and analysed. Note that the values vary between 56% and 17% on the Steelpoort sample (Figure 4.4), which is a 39% gap.

4.2.3 Savage's method to determine the 0.002mm fraction

Based on the short-comings identified (see section 4.2.2), Savage's method (2007) was studied in order to eliminate the need for hydrometer analysis. Savage suggested using Skempton's activity formula derived in 1953 to relate activity to the ratio (R) between the liquid limit and the plastic limit of a soil sample. Taken from Savage's 2007 paper, the formulae are as follows:

Activity as indicated in Equation 2.2, section 2.3.1:

$$\boldsymbol{Activity} = \frac{\boldsymbol{I_P}}{P_{0.002}} \tag{2.2}$$

Where:

$\boldsymbol{I_P}$ denotes plasticity index

$P_{0.002}$ denotes the percentage sample smaller than 0.002mm (2µm)

Plasticity Index as indicated in Equation 2.1, section 2.3.1:

$$\boldsymbol{I_P = W_L - W_P} \tag{2.1}$$

Where:

$\boldsymbol{I_P}$ denotes plasticity index

$\boldsymbol{W_L}$ denotes liquid limit

$\boldsymbol{W_P}$ denotes plastic limit

Savage found an exponential relationship between the Liquid Limit / Plastic Limit ratio (R) used to determine R values from a table published by Cornell University in 1951 and the activity values for Sodium Montmorillonite (7.2), Illite (0.9) and Kaolinite (0.38).

Liquid Limit to Plastic Limit Ratio (R)

$$\boldsymbol{R} = \frac{\boldsymbol{W_L}}{\boldsymbol{W_P}} \tag{4.1}$$

Activity in relation to plasticity ratio

$$\boldsymbol{Activity} = \ \boldsymbol{0.16.R^{2.13}} \tag{4.2}$$

Where:

$\boldsymbol{R}$ denotes plasticity ratio

$\boldsymbol{W_L}$ denotes liquid limit

$\boldsymbol{W_P}$ denotes plastic limit

Using the Equations 2.1, 2.2, 4.1 and 4.2, substituting to determine the percentage fraction passing 0.002mm, the following equation is derived:

Empirical calculation of the 0.002mm fraction (Savage 2007)

$$P_{0.002} = \frac{\boldsymbol{I_P}}{\boldsymbol{Activity}} = \frac{\boldsymbol{I_P}}{\boldsymbol{0.16.R^{2.13}}} = 6.25.\boldsymbol{I_P}.\boldsymbol{R^{-2.13}} \tag{4.3}$$

Typically the plasticity index values are determined from material passing the 0.425mm sieve. In order to determine a value that represents the gross sample, the percentage passing 0.425mm ($P_{0.425}$) needs to be factored in:

Empirical calculation of the 0.002mm fraction (Savage 2007)

$$P_{0.002} = \ 6.25.\boldsymbol{I_P}.P_{0.425}.(\frac{\boldsymbol{W_L}}{\boldsymbol{W_P}})^{\boldsymbol{-2.13}} \tag{4.4}$$

Table 4.5 draws a comparison between the hydrometer values obtained and those derived from Savage’s formula.

Table 4.5: 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

On Table 4.5 it can be seen that some of the results compare favourably, while those in *italics* are unacceptable. The correlation achieved by comparing Savage's method to hydrometer values gives a compelling argument that Savage was correct in not relying on hydrometer values. It might also suggest that the method proposed by Savage does not work. The methods need to be verified by a reliable method, which has apparently not yet been found. Figures 4.17 to 4.19 graphically indicate the comparison between hydrometer values and Savage's values for the $P_{0.002}$ fraction for Steelpoort, Brandwag and Botshabelo samples.

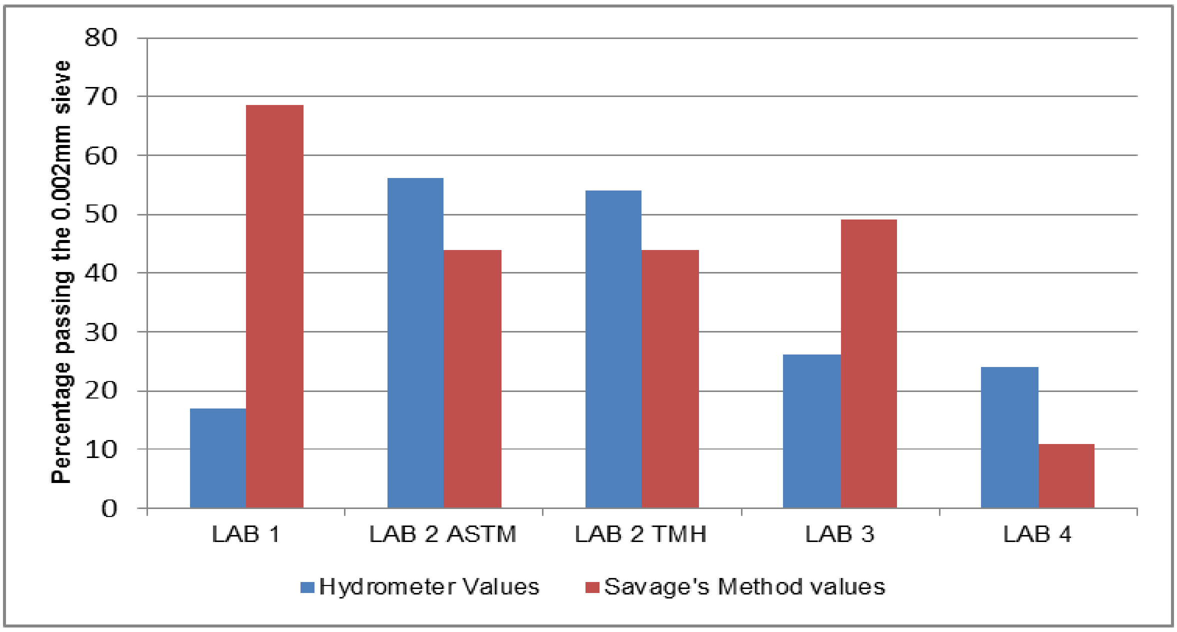


Figure 4.15: Steelpoort sample hydrometers compared to Savage's values

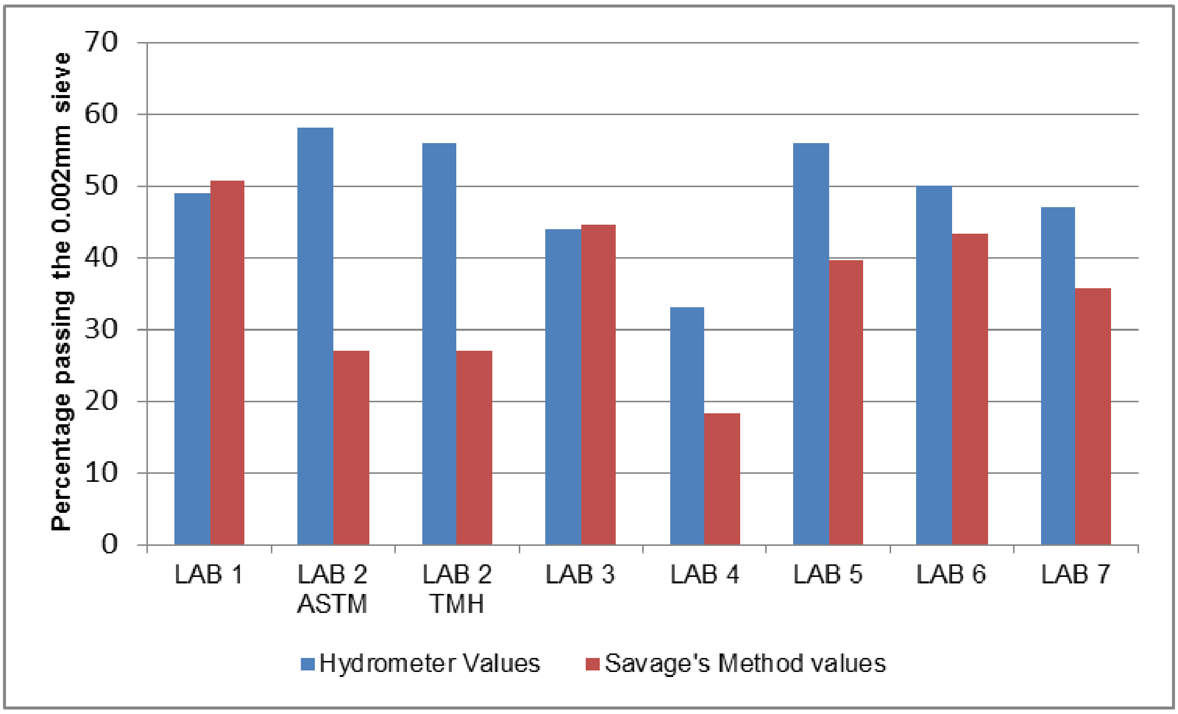


Figure 4.16: Brandwag sample hydrometers compared to Savage's values

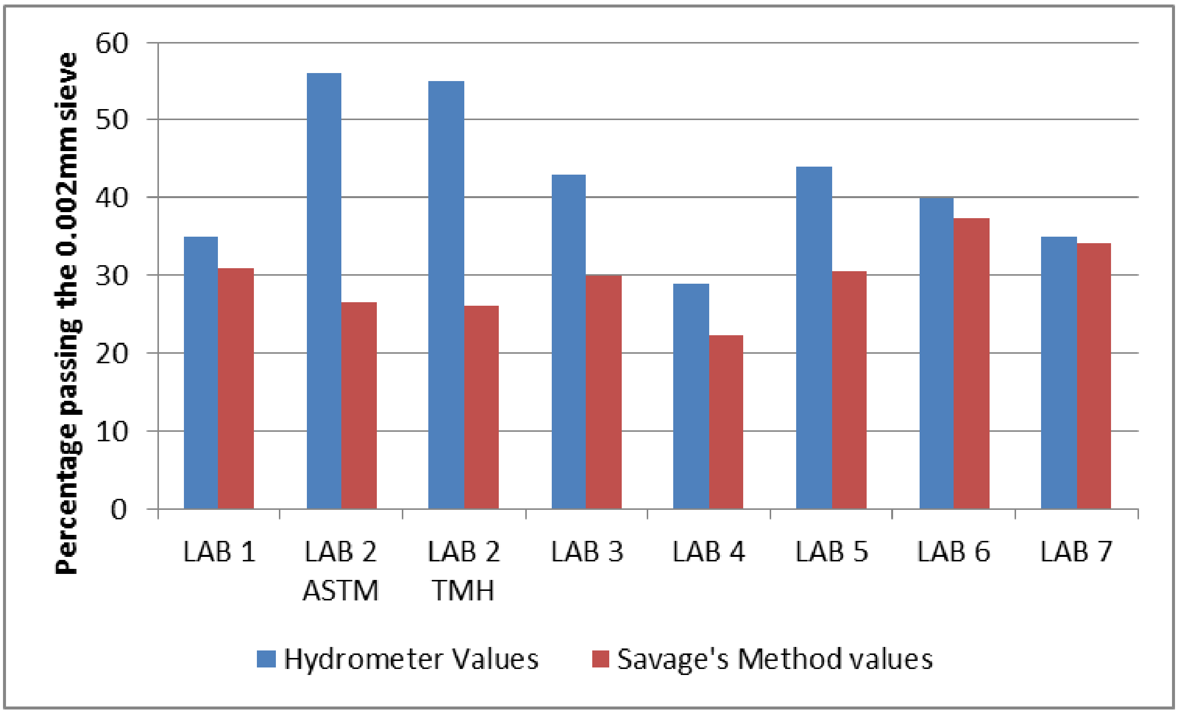


Figure 4.17: Botshabelo sample hydrometers compared to Savage's values

Generally the correlation achieved varies from -0.153 to 0.207, which indicates little to no correlation.

4.2.4 Atterberg limits

Atterberg limits include the Liquid Limit (W_L), Plastic Limit (W_P), Plasticity Index (I_P) and Linear Shrinkage (W_{LS}) of a soil sample. The Shrinkage Limit (W_{SL}) can also be determined using the Atterberg limits.

The commercial laboratories in South Africa make use of Casagrande-cup apparatus and typically perform the Liquid Limit test according to TMH1:1986 Method A2, which uses the same apparatus as the British Standard BS 1377-2. It was found that sample preparation has a significant bearing on the results, which may be responsible for the variance between the different commercial laboratories. The following tables and graphs indicate the analysis of the Atterberg limit states of Steelpoort, Brandwag and Botshabelo samples.

Based on Tables 4.6, 4.7 and 4.8 for Steelpoort, Brandwag and Botshabelo respectively, the differences between the maximum and minimum values were alarming. Several factors may have contributed to the variance obtained, notably sample preparation, operator bias and the mixing time of samples. In the candidate’s experience with the test conducted to determine the liquid limit of soils, it was found that the longer the mixing time, the higher the liquid limit recorded. Variances on the liquid limit and plastic limit have a knock-on effect in that the plasticity index is calculated as the difference between the two.

The Casagrande apparatus is widely considered to be subject to operator bias and has been replaced by the falling cone apparatus in many countries. Sampson and Netterburg (1984) investigated the use of the falling cone apparatus in South Africa, but their conclusions were not attractive. The fall cone test may be worth investigating based on the variance of results obtained in the following tables.

The plastic limit test is highly susceptible to operator bias, as discussed in section 3.4.4 b.

Figures 4.20 to 4.22 indicate the differences obtained between the various commercial laboratories visually.

Table 4.6: Steelpoort Atterberg Limits

Steelpoort Sample	LAB 1	LAB 2 ASTM	LAB 2 TMH	LAB 3	LAB 4	Average	Standard Deviation	Minimum	Maximum	% Difference
Liquid Limit	97	78	78	89	43	77,0	20,63	43	97	54.0%
Plastic Limit	57	35	35	40	12	35,8	16,08	12	57	45.0%
Plasticity Index	40	43	43	49	31	41,2	6,57	31	49	18.0%
Linear Shrinkage	20	18	18	20	15,1	18,2	2,01	15,1	20	4.9%

Table 4.7: Brandwag Atterberg Limits

Brandwag Sample	LAB 1	LAB 2 ASTM	LAB 3	LAB 4	LAB 5	LAB 6	LAB 7	Average	Standard Deviation	Minimum	Maximum	% Difference
Liquid Limit	66	52	60	33	56	71	45	54,4	11,96	33	71	33.0%
Plastic Limit	37	21	32	14	30	33	27	26,9	7,66	14	37	23.0%
Plasticity Index	29	31	28	19	26	38	18	27,5	6,57	18	38	13.0%
Linear Shrinkage	15,5	10	12	10,1	12	13,2	9,2	11,5	2,11	9,2	15,5	6.3%

Table 4.8: Botshabelo Atterberg Limits

Botshabelo Sample	LAB 1	LAB 2 ASTM	LAB 3	LAB 4	LAB 5	LAB 6	LAB 7	Average	Standard Deviation	Minimum	Maximum	% Difference
Liquid Limit	52	43	50	34	46	66	50	48,0	9,24	34	66	18.0%
Plastic Limit	24	20	23	17	24	30	29	23,4	4,47	17	30	12.0%
Plasticity Index	28	23	27	17	22	36	21	24,6	5,73	17	36	11.0%
Linear Shrinkage	13,5	10,5	13	8,6	12	12,2	10,4	11,3	1,62	8,6	13,5	4.9%

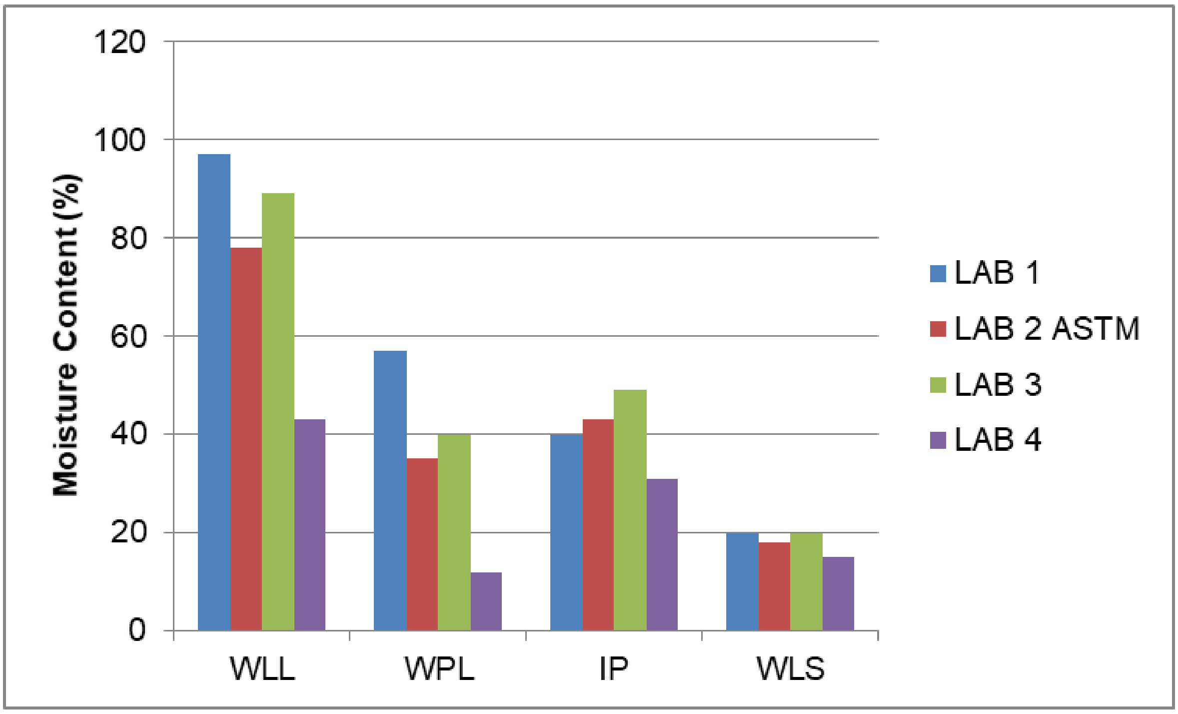


Figure 4.18: Liquid limit, plastic limit, plasticity index and linear shrinkage on Steelpoort sample

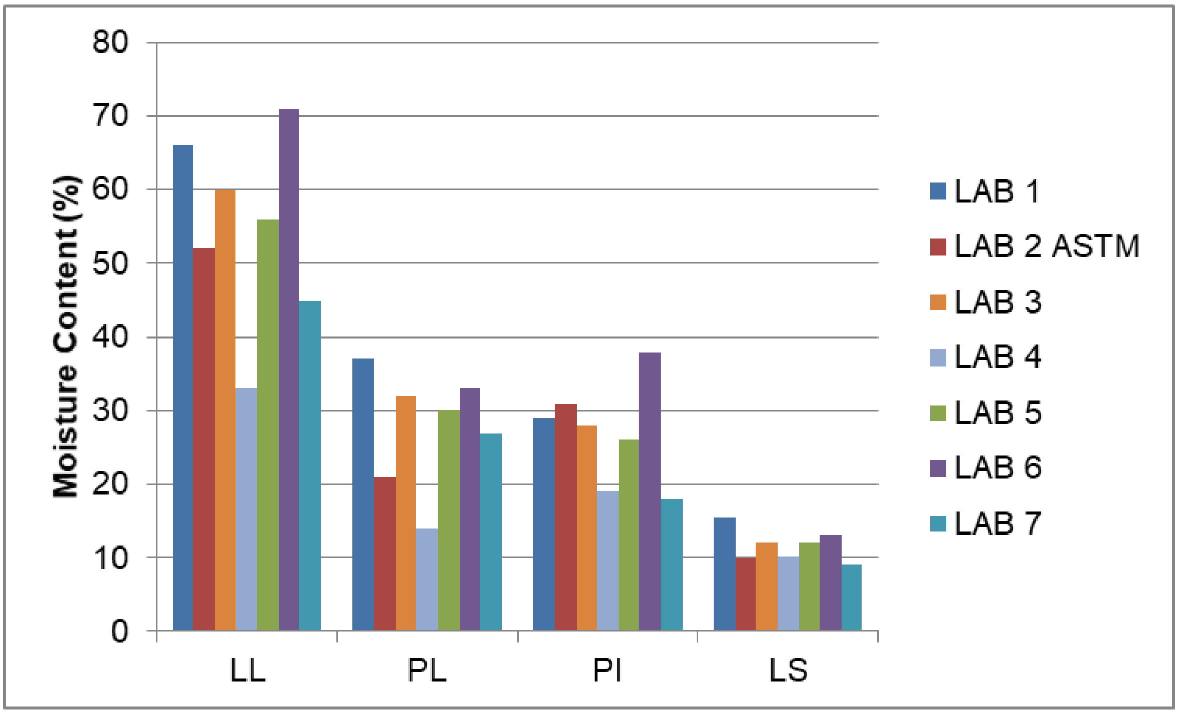


Figure 4.19: Liquid limit, plastic limit, plasticity index and linear shrinkage on Brandwag sample

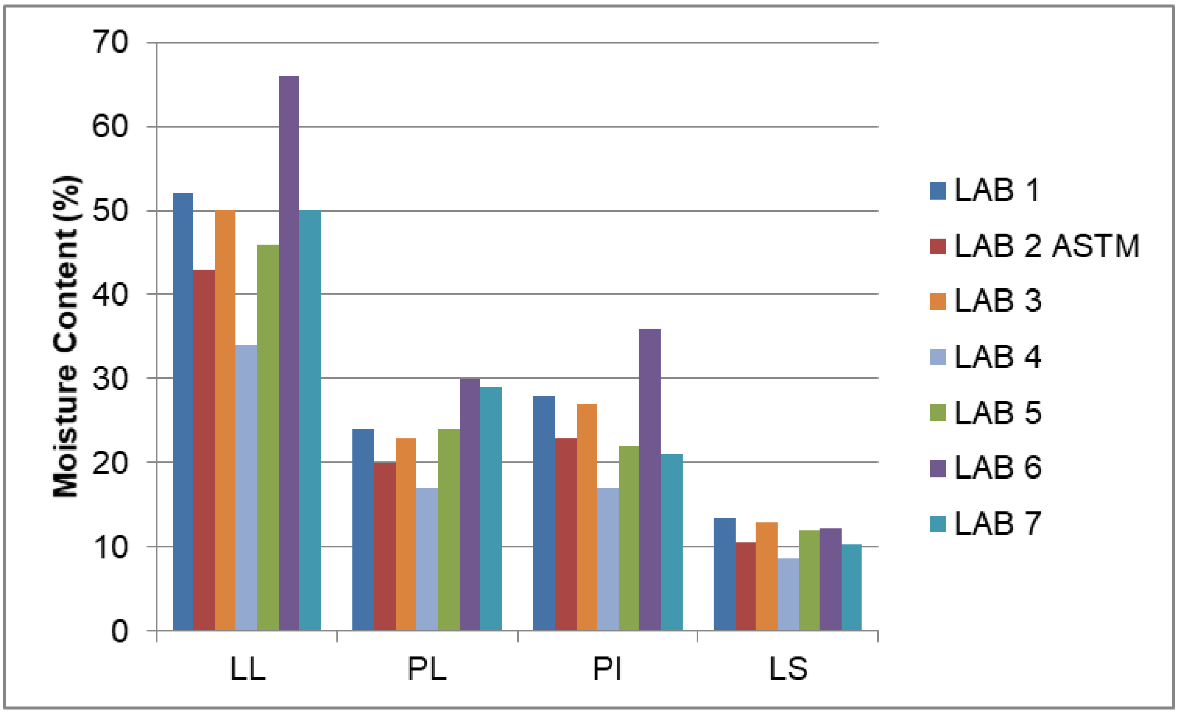


Figure 4.20: Liquid limit, plastic limit, plasticity index and linear shrinkage on Botshabelo sample

Based on Figures 4.20 to 4.22, it is clear that the linear shrinkage values are relatively consistent, with most laboratories within two percent of the average. The liquid limit values show a large variance with standard deviation values ranging from 9.24% (Botshabelo) to 20.63% (Steelpoort). The standard deviation on the plastic limit should be in the order of 50% of the liquid limit standard deviation, but it is significantly higher on Steelpoort and Brandwag. The plastic limit of a soil sample is typically determined by rolling a thread of the material to a diameter of 3mm. It is heavily subjected to operator bias and indicated large variance.

The plasticity index of a soil sample is the difference between the liquid limit and the plastic limit of the soil sample. The standard deviation of the plasticity index was estimated to be similar to those of the plastic limit, but evidently it is significantly lower. Plasticity index values are often considered to be an indication of potential expansiveness. Casagrande (1948) proposed a chart to indicate cohesive soils based on the relationship between the liquid limit and plasticity index of soil. This chart is typically used as part of the Unified Soils Classification System (USCS) to classify soils.

Figures 4.23 to 4.25 indicate plasticity charts for Steelpoort, Brandwag and Botshabelo based on the results obtained from various commercial soil laboratories. Based on the results from the various laboratories, Steelpoort may be classified as CI, CV, MV or ME, while Brandwag may be classified as CL, MI, CH, MV or CV. This gives a good indication of the risk involved with relying on one set of samples for design purposes.

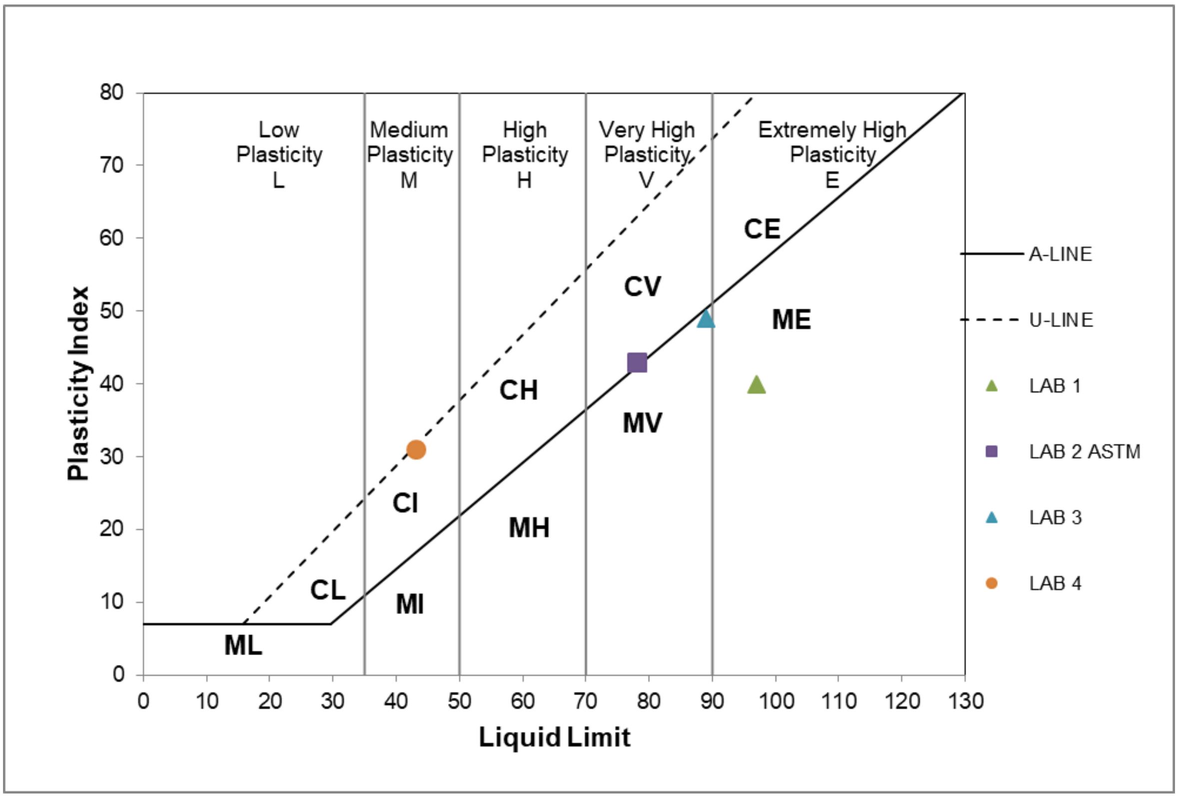


Figure 4.21: Casagrande’s plasticity chart, Steelpoort sample

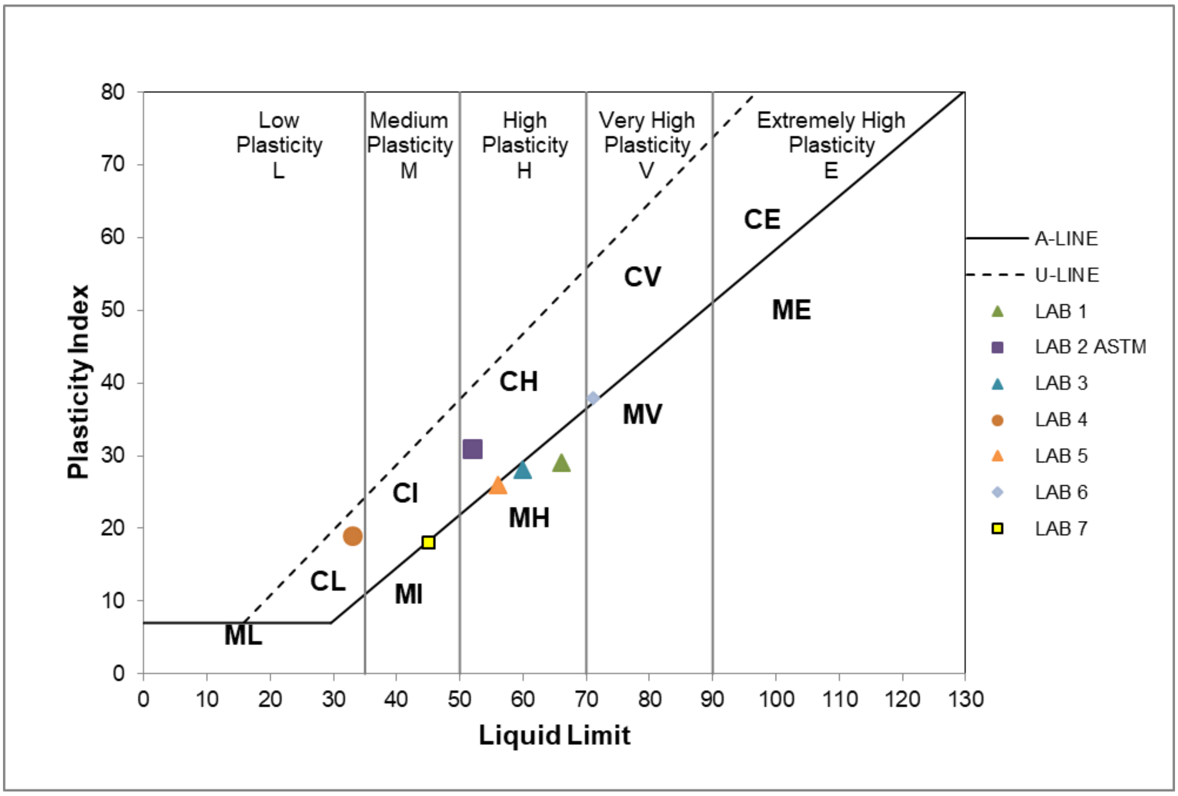


Figure 4.22: Casagrande's plasticity chart, Brandwag sample

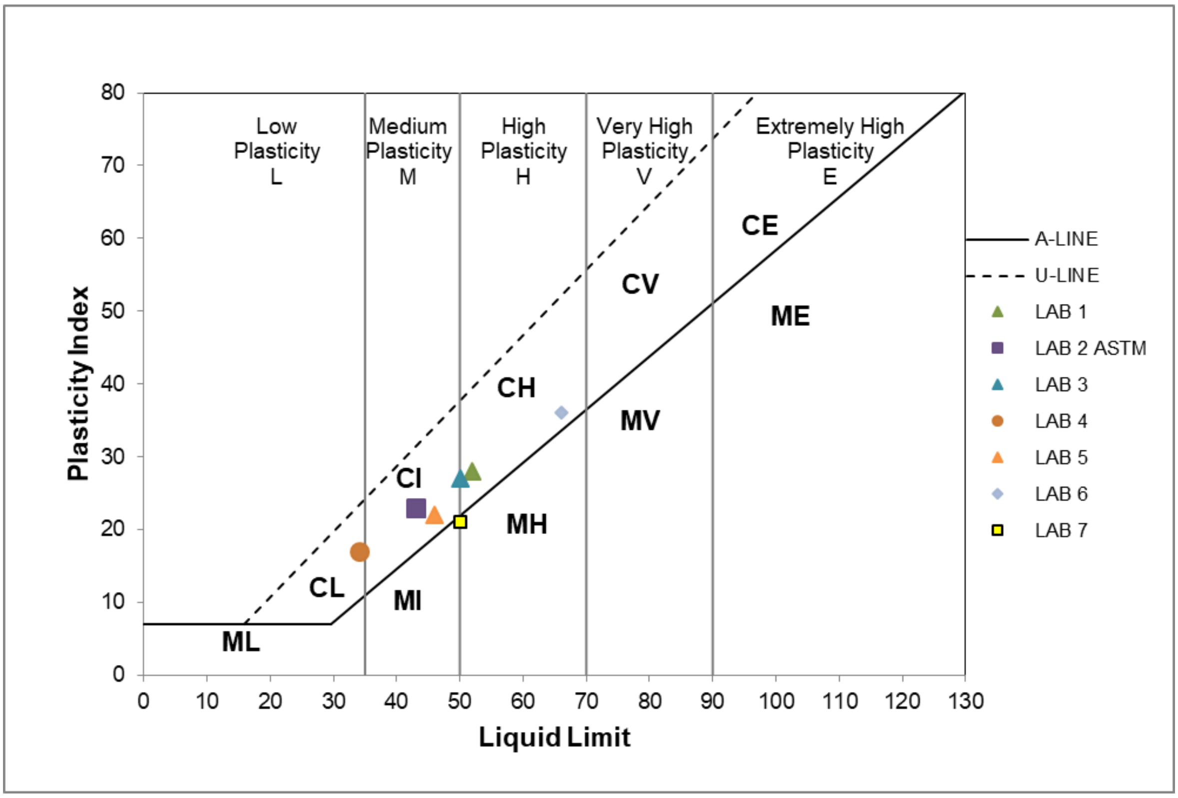


Figure 4.23: Casagrande's plasticity chart, Botshabelo sample

4.3 Van der Merwe's empirical method for the estimation of potential heave

Van der Merwe's method relies on the Plasticity index of the gross sample and the $P_{0.002}$ fines fraction, which according to the British Classification is deemed as the clay fraction of a soil sample. For van der Merwe's method to give valuable output, valuable input is required. This chapter does not focus on the prediction model itself, but rather the input parameters.

For the purpose of better understanding the method, a simplified example on the process follows:

a) Commercial laboratory test results typically yield values for the Plasticity Index of the soil sample; however, depending on the test method used, it typically refers to the material passing the 0.425mm sieve. This value needs to be adjusted to reflect the entire sample. This is done as follows:

Plasticity index of gross sample

$$\boldsymbol{I_{P\,Gross} = I_P . P_{0.425}} \tag{4.5}$$

Where:

$\boldsymbol{I_{P\,Gross}}$ denotes plasticity index of the whole sample

$\boldsymbol{P_{0.425}}$ denotes the percentage fraction passing through an aperture size of 0.425mm

b) The gross plasticity index of the sample is then plotted on the y-axis of a curve compared to the $P_{0.002}$ of the gross sample on the x-axis on "The classification of heave potential" curve, after van der Merwe (1964). Figure 4.32 shows an example of such a plot with values obtained from various laboratories using the Hydrometer analysis and Savage's values to obtain the $P_{0.002}$ fraction. Table 4.9 compares the resultant heave potential to that obtained using Savage's method to determine the $P_{0.002}$ fraction.

Based on Figures 4.26 to 4.28 it is evident that misconceptions may occur when a single set of samples are used to determine the heave potential of soils using foundation indicators and prediction models such as van der Merwe's. Steelpoort sample may be classified as "medium", "high" or "very high". The difference in classification is exponential, meaning that the eventual predicted settlement for a "very high" classification would be four times the eventual predicted settlement of a "medium" classification.

Brandwag sample may be classified as "low", "medium" or "high" with one result bordering "very high" indicating that the eventual heave prediction will vary from 0mm (low classification) to very significant values depending on the depth of the expansive layer and the thickness thereof. Botshabelo sample varied similar to that of Brandwag.

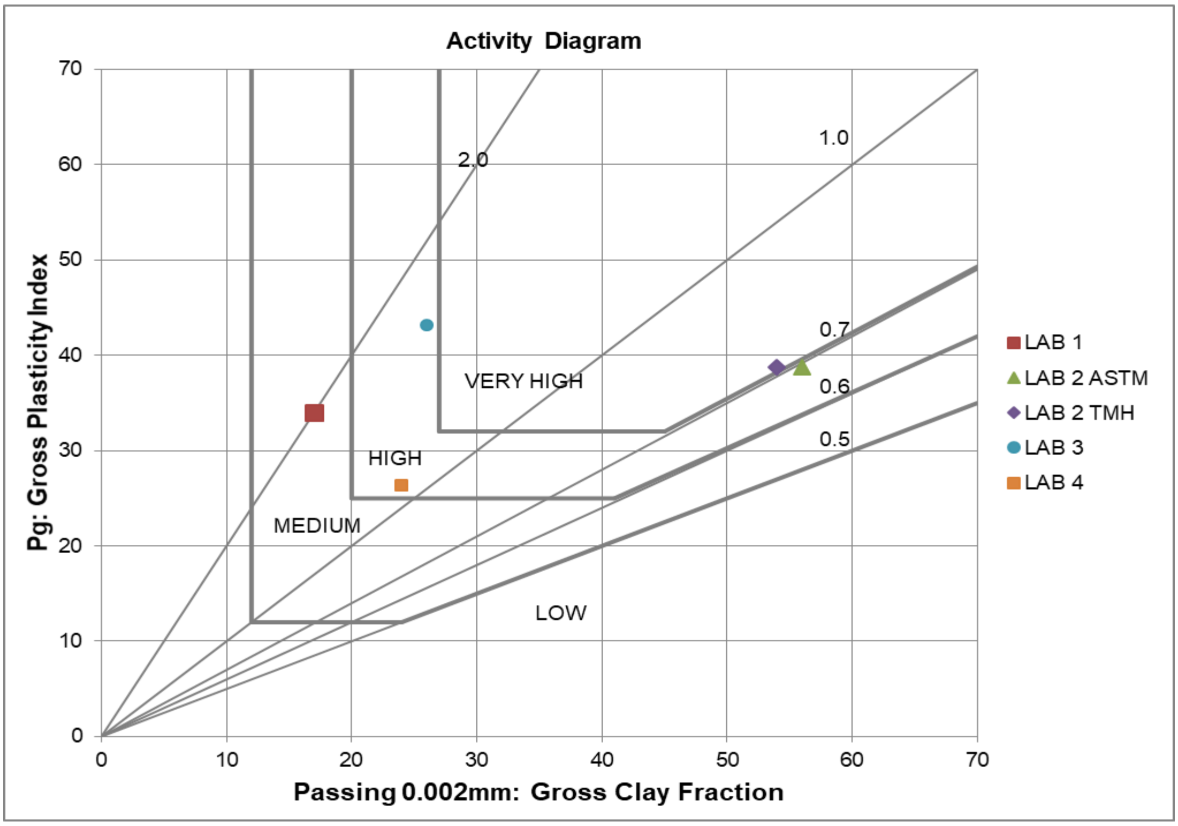


Figure 4.24: The classification of heave potential of Steelpoort Sample after van der Merwe, 1964 with Skempton's (1953) activity lines

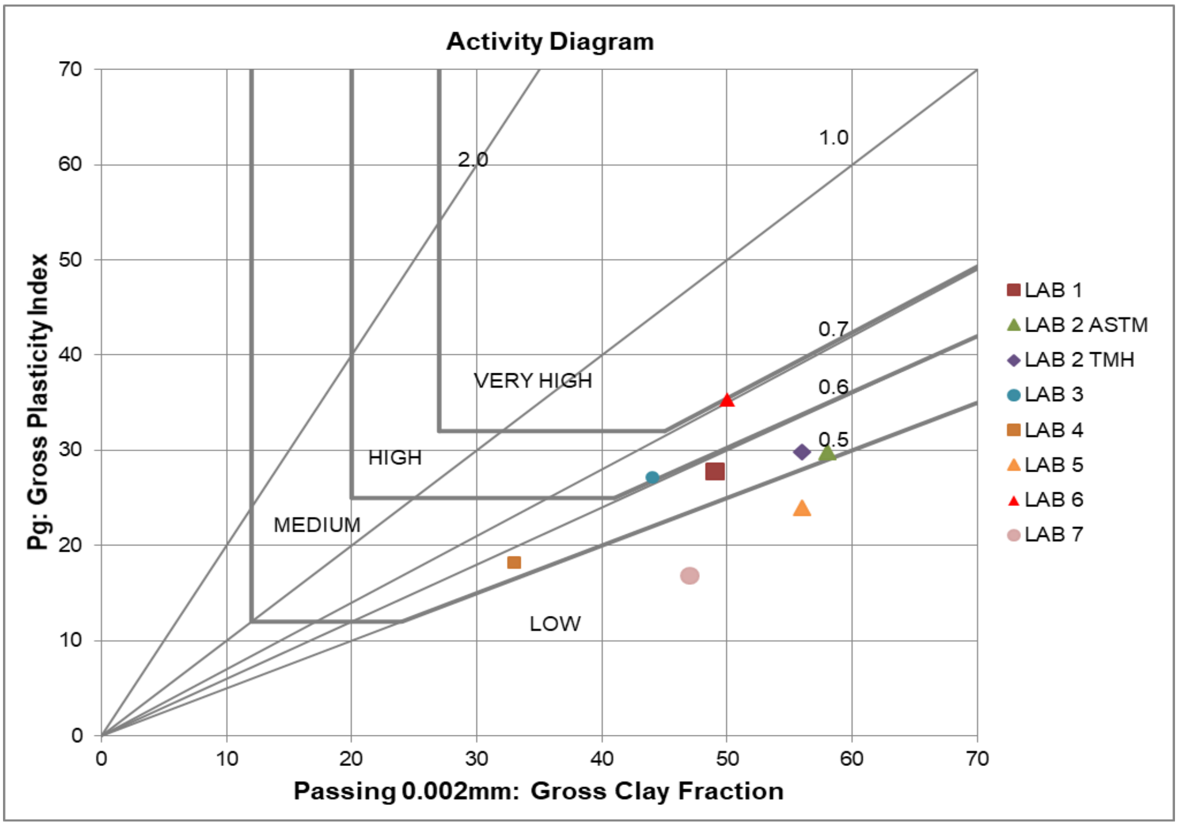


Figure 4.25: The classification of heave potential of Brandwag Sample after van der Merwe, 1964 with Skempton's (1953) activity lines

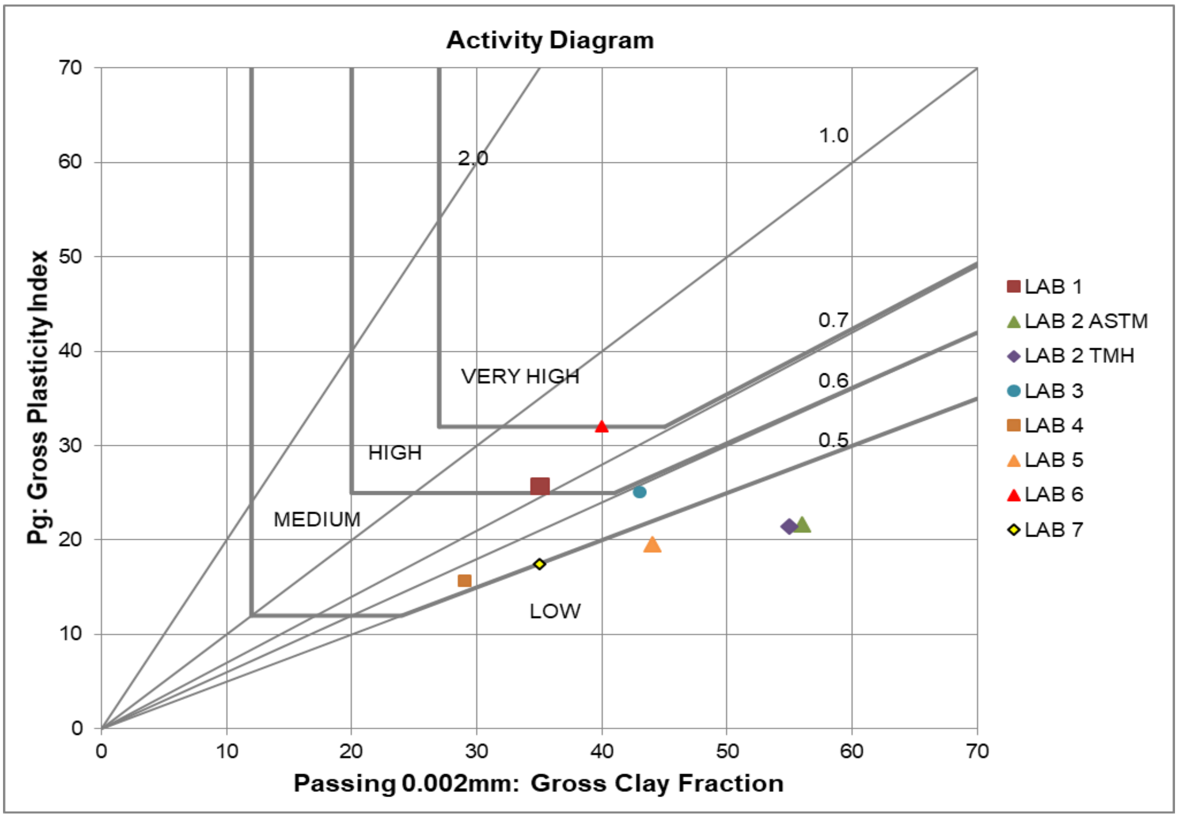


Figure 4.26: The classification of heave potential of Botshabelo Sample after van der Merwe, 1964 with Skempton's (1953) activity lines

Considering Savage's method (2007) to replace the "questionable" hydrometer values, Figures 4.29 to 4.31 were obtained. The classification variance remained and did not seem to improve using Savage's values.

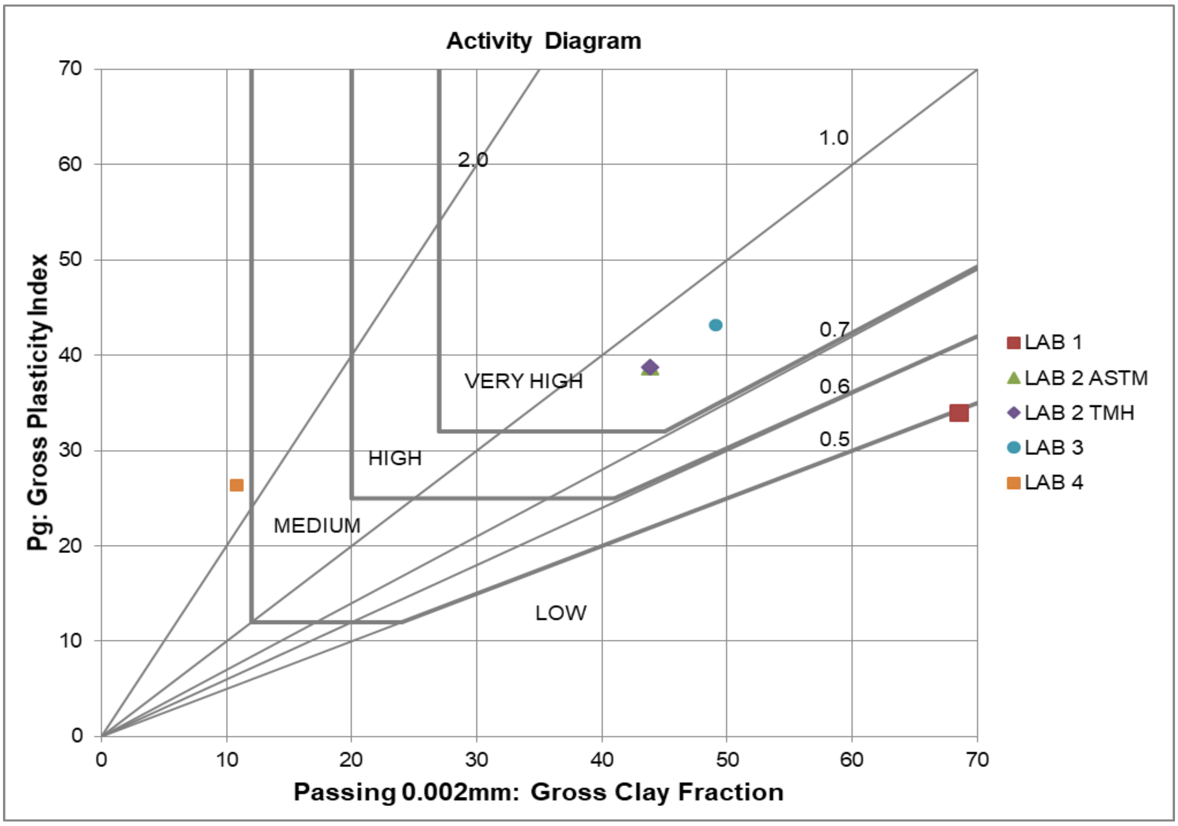


Figure 4.27: The classification of heave potential of Steelpoort Sample using Savage's $P_{0.002}$ values after van der Merwe (1964) with Skempton's (1953) activity lines

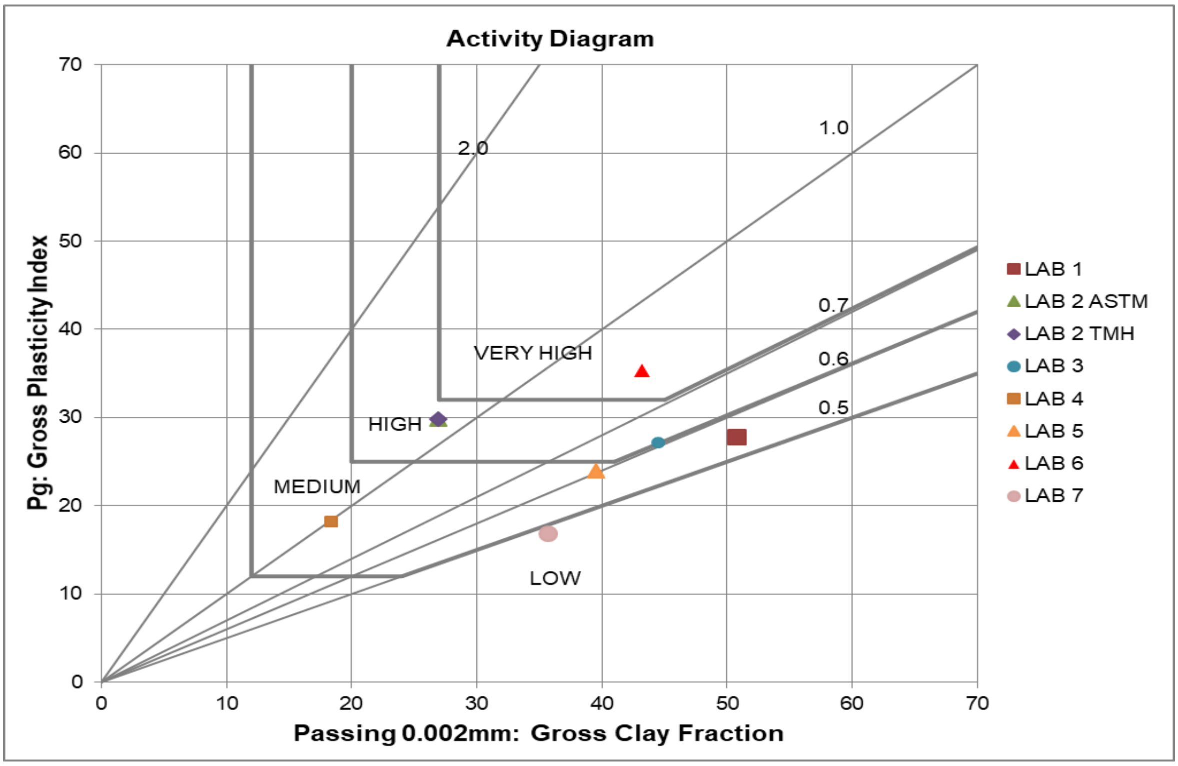


Figure 4.28: The classification of heave potential of Brandwag Sample using Savage's $P_{0.002}$ values after van der Merwe (1964) with Skempton's (1953) activity lines

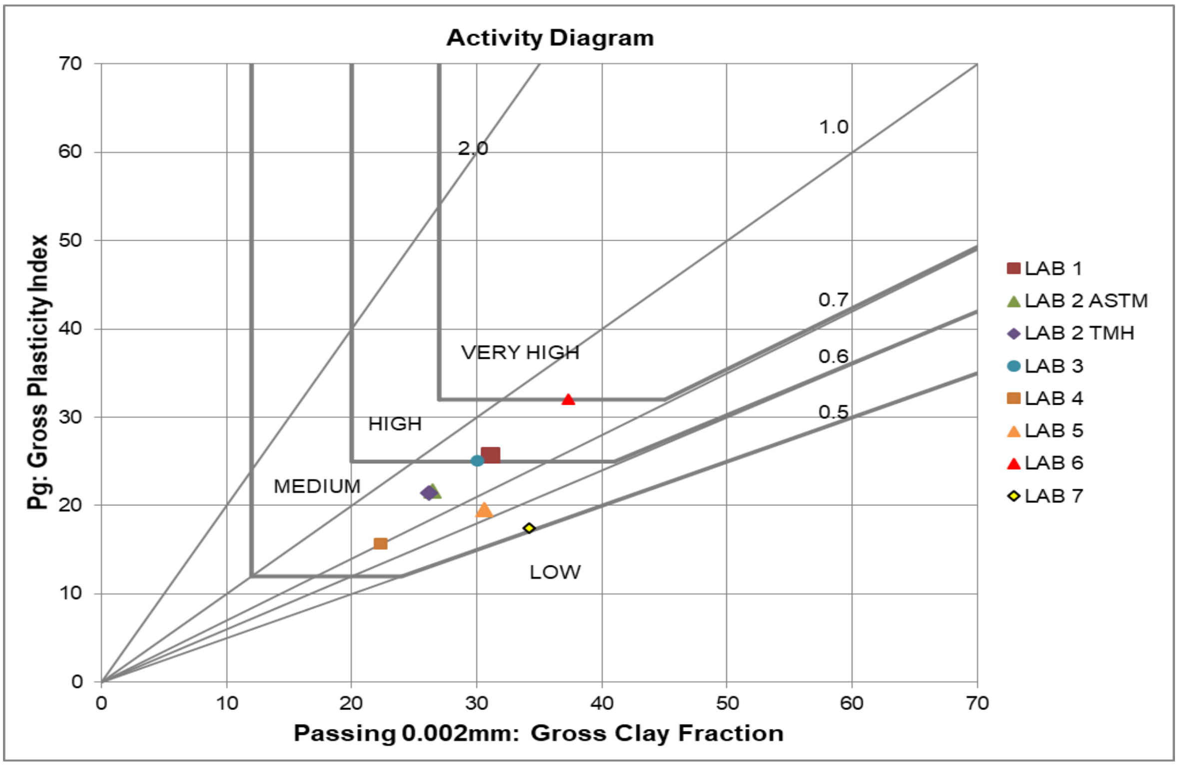


Figure 4.29: The classification of heave potential of Botshabelo Sample using Savage's $P_{0.002}$ values after van der Merwe (1964) with Skempton's (1953) activity lines

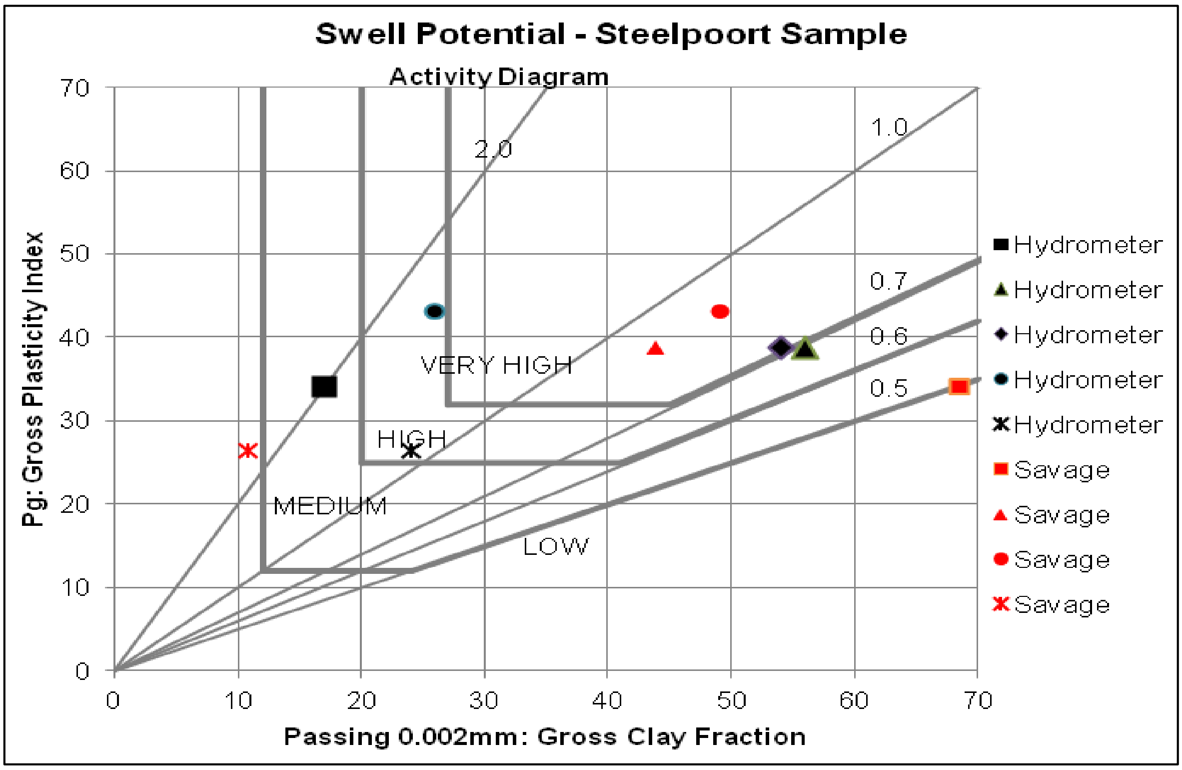


Figure 4.30: The classification of heave potential after van der Merwe (1964) using Hydrometer values and Savage's values for the $P_{0.002}$ fraction with Skempton's (1953) activity lines

Table 4.9: Heave Potential Classification: Hydrometer Analysis compared to Savage's Method on Steelpoort Sample Analysis

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

Skempton in his 1953 paper 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 suggested using slopes of less than 0.75 to refer to inactive clays, slope more than 1.4 would suggest active clays and everything in between would be referred to as normal clays. On Figures 4.26 to 4.32, those slopes were adjusted to reflect slopes of 2.0, 1.0, 0.7, 0.6 and 0.5, with anything less than 0.5 considered inactive.

4.4 Discussion

Seven leading commercial laboratories were tasked with performing Foundation Indicators, which refers to the Atterberg Limits and Particle Size Analysis up to the fraction smaller than 0.002mm ($P_{0.002}$). Although the physical sieving provided comparable results up to the 0.425mm ($P_{0.425}$) sieve, anything finer proved troublesome.

The author concluded that the problem probably lies with the preparation of the samples, as it was found to have a major impact with testing done in-house. Variations within the samples may have had a significant impact on the end results indicating that a reliability approach may be required in order to provide better prediction in terms of heave potential.

The Atterberg Limits were done with fractions passing the 0.425mm sieve, and preparation might have played a role there as well. A grouping of laboratories obtained results that compared well, with two laboratories getting substantially different results.

The finer fraction, those passing the 0.075mm sieve, proved problematic as not all of the laboratories used the same method, although theoretically it should yield similar results. The results varied substantially and the range between the highest and lowest $P_{0.002}$ (passing 0.002mm) is alarming.

It can be concluded that using foundation indicators as input parameters for empirical methods is a very risky approach and that other approaches need to be identified.

Stott and Theron (2016) described the variance of the Steelpoort sample and other clays using small-scale suction tests and noted that due to the variance obtained, it is dangerous to base design on one set of test results. The co-efficient of variance obtained from soil-suction tests on Steelpoort was recorded as 39 which may explain the major difference in the assessment by the leading commercial laboratories. The variations in laboratory results on Steelpoort sample was

presented at the 16th African Regional Geotechnical Conference and led to the Soil Mechanics Research Group focussing attention on the variability of soils resulting in various papers, including one in the Geotechnical Engineering Journal.
The author believes that the variance within clays are significant to such an extent that major differences may be recorded in the determination of finer fractions and the Atterberg limits of soils. The factors identified as contributors to variance in-house are considered to be limited to operator skill in testing, splitting the sample into several representative samples, minor deviations in sample preparation in that the effort applied to break down lumps are not always the same. Operator bias may have contributed to minor deviation, but the majority of the deviations encountered are ascribed to sample variance and questionable hydrometer analysis.

Savage's method to determine the $P_{0.002}$ may hold merit and may be worth investigating further. It is dependent on Atterberg limits as an input parameter and the model itself may need some refining. Comparing the values obtained to those obtained using hydrometer analysis did not yield favourable results, but studies by Stott and Theron, notably a paper named "Shortcomings in the estimation of clay fraction by Hydrometer" published in the Journal of Civil Engineering Volume 58 No 2 (June 2016), indicates that hydrometer analysis to determine the clay fraction is not reliable.

Foundation indicators, a grouping of tests offered by most commercial soils laboratories in order to use as input to van der Merwe's method, may not offer adequate information to reliably predict the heave potential of soil samples. With the current available information and variations observed, it is possible to predict that for clay with a high coefficient of variability the method may give very unreliable results.

The current procedures used to determine the heave potential of clayey South African soils do not yield reliable inputs to properly assess and design for heaving clays. On this basis, an investigation into methods used internationally followed to lay a possible foundation for future research (Chapter 5).

CHAPTER 5: OTHER METHODS USED INTERNATIONALLY

5.1 Introduction

Chapters 3 and 4 focussed primarily on van der Merwe’s empirical method and the input parameters used in order to predict heave potential. Questionable input parameters and the variability within soils noted has prompted the author to consider alternative procedures used internationally.

Various heave prediction models were studied and relating tests were done and analysed with specific focus given to two methods:

a) Free Swell Ratio tests: A paper by Prakash and Sridharan (2004) measured the ratio of volume change of soil passing the 0.425mm sieve in distilled water (V_d) to the volume change of the same soil in kerosene (V_k) to determine the Free Swell Ratio. Kerosene is a non-polar fluid and as such is not expected to be absorbed into the clay structure of the sample.

b) Methylene Blue test: A paper by Türköz and Tosun (2010) determined the heave potential of soils by measuring to amount of methylene blue solution absorbed by the soil samples. The amount of methylene blue being adsorbed is a factor of cation exchange capacity, the amount and type of clay minerals and the specific surface area.

5.2 Free Swell Ratio Tests

Free swell ratio tests were conducted on eighteen (18) samples in accordance to the method described by Prakash and Sridharan in a paper by them published in 2004. The test compares the volume of the soil sample in kerosene to the final volume of a sample inundated with distilled water. The swell is measured in relation to the unchanged volume in kerosene. The ratios obtained are then plotted on a graph as indicated in Figure 5.1. The expansiveness Table 5.1 indicates the results obtained.

Table 5.1: Free Swell Ratio Test Results after Prakash and Sridharan (2004)

Sample	Free swell ratio	Soil Expansiveness	Clay type	Dominant Clay	Classification
Soil 1 Iliso Consulting, 1 Kgotsong, TP1, KG 01, 0-1820	1,22	Low	Mixture of swelling	Kaolinitic and Montmorillonitic	Class II : Kaolinitic Montmorillonitic Soils
Soil 2 Iliso Consulting, 2 Kgotsong, TP 2, KG 02, 0-670	0,85	Negligible	Non-swelling	Kaolinitic	Class I : Kaolinitic Soils
Soil 3 Iliso Consulting, 3 Kgotsong, TP5, KG05, 0-740	1,63	Moderate	Swelling	Montmorillonitic	Class IIIA : Montmorillonitic Soils - Moderately Swelling
Soil 4 Iliso Consulting, 4 Kgotsong , TP 5, KG 06, 740-1240	1,50	Moderate	Swelling	Montmorillonitic	Class IIIA : Montmorillonitic Soils - Moderately Swelling
Soil 5 Iliso Consulting, 5 Kgotsong, TP 6, 0-400	0,89	Negligible	Non-swelling	Kaolinitic	Class I : Kaolinitic Soils
Soil 6 Iliso Consulting, 6 kgotsong, TP7, KG 08, 0-1020	0,90	Negligible	Non-swelling	Kaolinitic	Class I : Kaolinitic Soils
Soil 7 Iliso Consulting, 7 Lauries Park, TP 1, LP01, 0-630	1,31	Low	Mixture of swelling	Kaolinitic and Montmorillonitic	Class II : Kaolinitic Montmorillonitic Soils
Soil 8 Iliso Consulting, 8 Lauries park, TP3-LP11, 1420-1840	0,95	Negligible	Non-swelling	Kaolinitic	Class I : Kaolinitic Soils
Soil 9 Iliso Consulting, 9 Lauries park, TP4, LP09, 1870-3000	1,13	Low	Mixture of swelling	Kaolinitic and Montmorillonitic	Class II : Kaolinitic Montmorillonitic Soils

Sample	**Free swell ratio**	**Soil Expansiveness**	**Clay type**	**Dominant Clay**	**Classification**
Soil 10 Iliso Consulting, 10 Lauries park, TP5, LP 11, 1420-1840	1,43	Low	Mixture of swelling	Kaolinitic and Montmorillonitic	Class II : Kaolinitic Montmorillonitic Soils
Soil 11 Iliso Consulting, 11 Lauries park, TP5, LP 12, 1840-2870	1,67	Moderate	Swelling	Montmorillonitic	Class IIIA : Montmorillonitic Soils - Moderately Swelling
Soil 12 Iliso Consulting, 12 Lauries park, TP 6, LP13, 0-1890	1,21	Low	Mixture of swelling	Kaolinitic and Montmorillonitic	Class II : Kaolinitic Montmorillonitic Soils
Soil 13 Iliso Consulting, 13 Lauries Park, TP7, LP15, 1460-2810	1,50	Moderate	Swelling	Montmorillonitic	Class IIIA : Montmorillonitic Soils - Moderately Swelling
Soil 14 Iliso Consulting, 14 MK Square, TP 1, MK 01, 0-710	0,83	Negligible	Non-swelling	Kaolinitic	Class I : Kaolinitic Soils
Soil 15 Iliso Consulting, 15 MK Square, TP1, MK 02, 710-1220	0,94	Negligible	Non-swelling	Kaolinitic	Class I : Kaolinitic Soils
Soil 16 Iliso Consulting, 16 MK Square, TP2, MK04, 940-2810	1,13	Low	Mixture of swelling	Kaolinitic and Montmorillonitic	Class II : Kaolinitic Montmorillonitic Soils
Soil 17 Iliso Consulting, 17 MK Square, TP 3, MK 05, 0-800	1,43	Low	Mixture of swelling	Kaolinitic and Montmorillonitic	Class II : Kaolinitic Montmorillonitic Soils
Soil 18 Iliso Consulting, 18 MK Square, TP4, MK07, 0-690	1,06	Low	Mixture of swelling	Kaolinitic and Montmorillonitic	Class II : Kaolinitic Montmorillonitic Soils

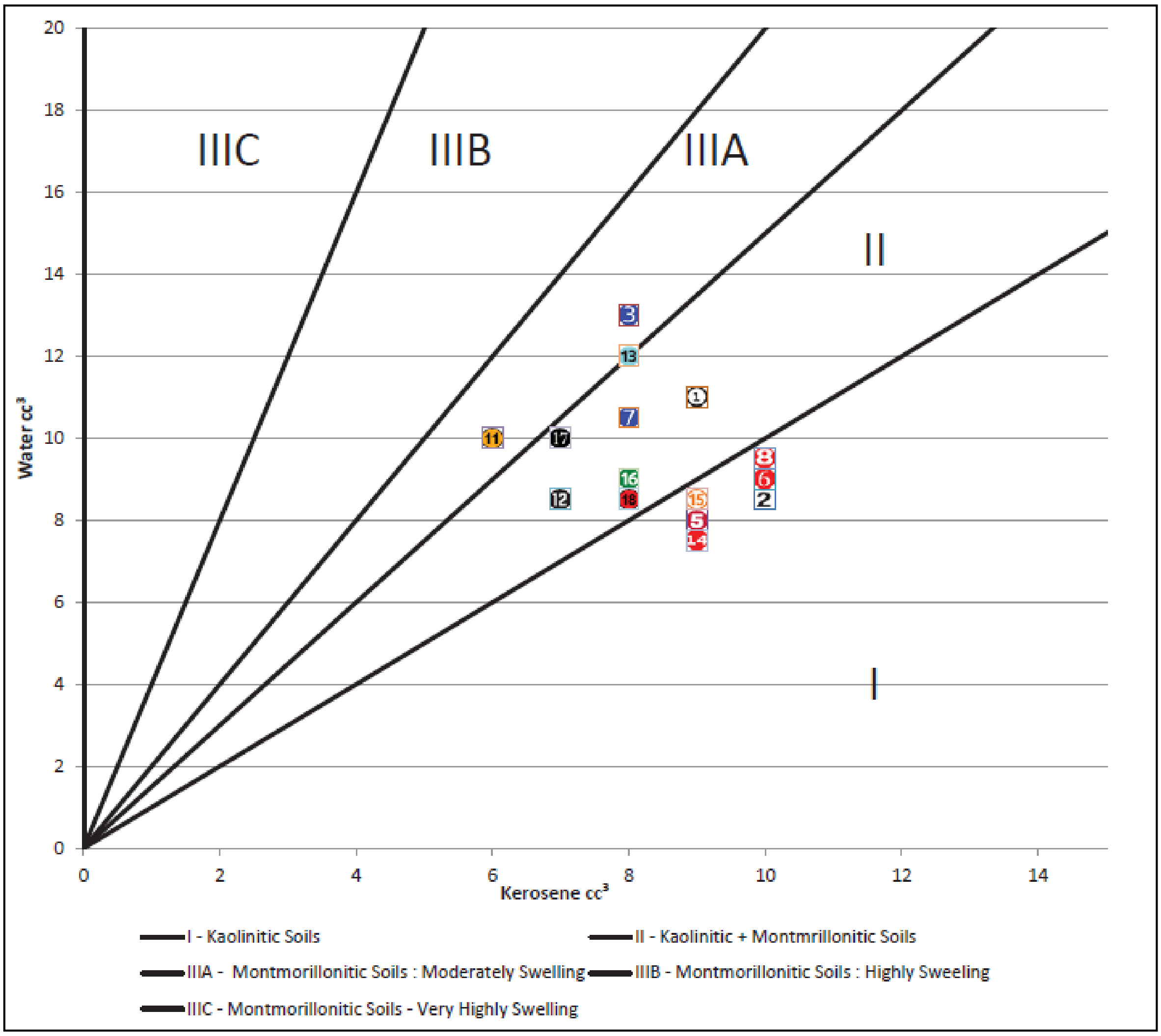


Figure 5.1: Classification of soils as swelling (montmorillonitic) and non-swelling (kaolinitic) types after Prakash and Sridharan (2004)

The test method and the theory behind it initially seemed very promising, but it does not account for the effect of over burden pressure and some of the readings obtained were questionable. All the values were expected to exceed 1.0, but several values lower than one were recorded. The test may not have been performed correctly or the values lower than one may indicate some other problem. Further investigation may be required. Comparison test results including other testing methods and models follow under the subsequent subtitles as part of this chapter.

Some difficulties encountered with this test include:

a) Difficulty in measuring 10mm of materials – 10 grams was used as specified in the paper

b) Difficulty in measuring out 100mm of water or kerosene – volume varies as the voids in the soil are filled

c) Difficulty in mixing the water or kerosene properly with soil sticking to the sides of the glass containers

d) The taking of manual readings every hour proved troublesome.

It is possible that the Free Swell Index may prove to give a good indication of potential expansiveness. Based on tests conducted as part of this study, this method seems to underestimate the heave potential of South African clays, generally finding the estimated heave of the analysed samples to be negligible, low or moderate.

5.3 Methylene Blue Tests

A paper by Türköz and Tosun (2010) indicated a relatively easy test method focused on cation exchange capacity, which is generally believed to be a good indicator of heave potential. Sixteen samples were analysed accordingly, graphed to a chart proposed by Çokça (1991). The results are as follows: (Table 5.2)

Table 5.2: Methylene Blue Value Heave Prediction Values using the method stated by Türköz and Tosun (2010)

Sample nr	**Clay Content (% smaller than 0.002mm)**	**Methylene Blue Value (g / 100g)**	**Estimated Heave Potential**
Sample 1	16	3,1	Low
Sample 2	56	8,3	High
Sample 3	25	6,7	Low
Sample 4	51	9,3	High
Sample 5	51	8,3	High
Sample 6	49	7,3	High
Sample 7	48	5,8	High
Sample 8	27	9	Low
Sample 9	36	7,6	Medium
Sample 10	48	7,1	High
Sample 11	37	10	High
Sample 12	20	3,2	Low
Sample 13	37	4	Medium
Sample 14	36	10,2	High
Sample 15	36	9,1	High
Sample 16	35	6,4	Medium

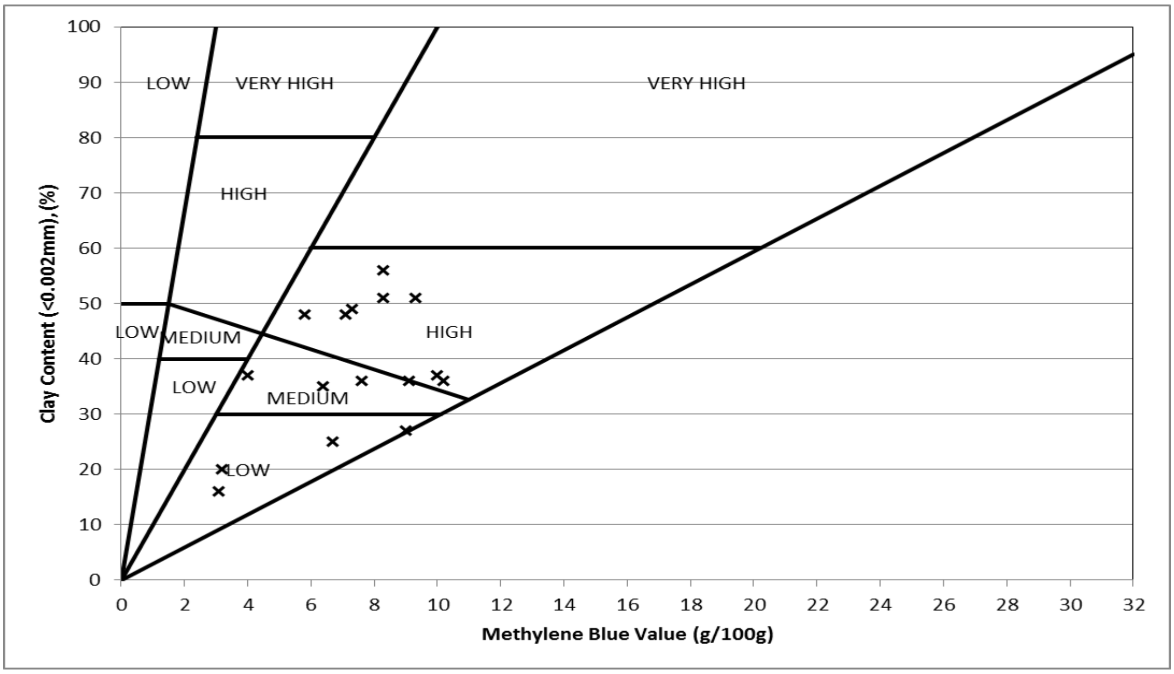


Figure 5.2: Estimating Heave Potential based on Methylene Blue Values after Çokça (1991)

With an increase in methylene blue adsorbed from kaolinite to montmorillonite, it is possible that potential expansiveness may by estimated by the amount of solution adsorbed. Various samples perceived to range from non-expansive to highly expansive were subjected to analysis using this method as part of this study. A spectrum of potential heave classifications was achieved ranging from “low” to “high” with the “very high” classification seeming somewhat illusive. The results obtained to date were viable, further research is warranted.

5.4 Gross Methylene Blue Value

Assuming that it is only clay minerals that absorb the methylene blue, the methylene blue value of the gross sample could also be used to determine heave potential. Gross Methylene Blue Value (GMBV) has been calculated for each sample. The boundaries differentiating between various classifications (low, medium, high and very high) should be investigated. For the purpose of this study the boundary values were assumed.

Table 5.3: Determination of Gross Methylene Blue Values

Sample	% Passing 0.425mm	MBV	GMBV	Sample	% Passing 0.425mm	MBV	GMBV
1	70	3.1	2.2	9	92	7.6	7.0
2	88	8.3	7.3	10	92	7.1	6.5
3	96	6.7	6.4	11	92	10	9.2
4	85	9.3	7.9	12	89	3.2	2.8
5	93	8.3	7.8	13	92	4	3.7
6	88	7.3	6.4	14	98	10.2	9.9
7	87	5.8	5.1	15	81	9.1	7.4
8	80	9	7.2	16	65	6.4	4.1

5.5 Comparing the Results of Various Heave Prediction Models in order to Evaluate the Methods Used

Upon analysing samples using van der Merwe’s method, the Free Swell Ratio method and the Methylene Blue Value method, it was still unclear which method produced more favourable results. It was then decided to compare the results with each other. The following pages indicate the findings made and contain more detailed analysis on the Free Swell Tests and Methylene Blue Value Tests.

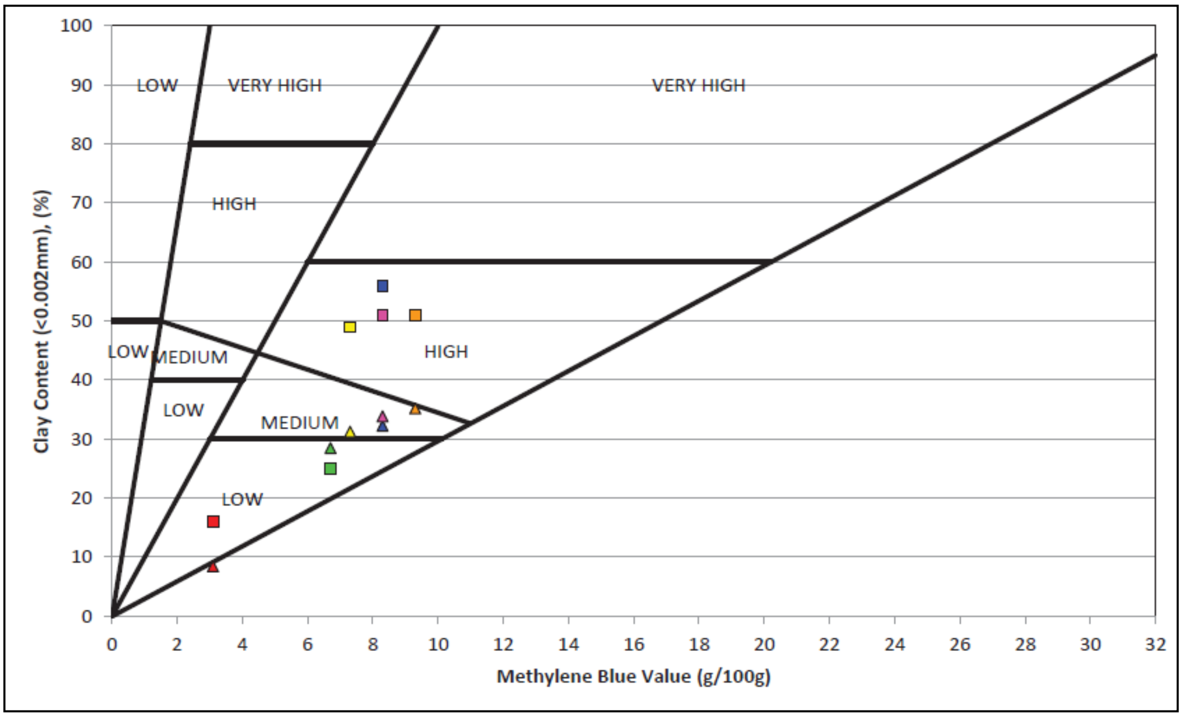


Figure 5.3: Heave potential classification chart (samples 1-6) after Çokça (1991)

Table 5.4: Input parameters to determine heave potential on samples 1-6

Values depicted as squares □		Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage	Percentage passing 0.002mm	Methylene Blue Value	Savage P0.002mm	MBV Classification
Sample 1	🟥	29	9,9	19,1	6,7	16	3,1	8	LOW
Sample 2	🟦	58,1	26,3	31,8	7,6	56	8,3	32	HIGH
Sample 3	🟩	38,6	20,8	17,8	10,1	25	6,7	28	LOW
Sample 4	🟧	65,3	29,5	35,8	15	51	9,3	35	HIGH
Sample 5	🟪	57,8	26	31,8	14,5	51	8,3	34	HIGH
Sample 6	🟨	56,4	25,5	30,9	14,3	49	7,3	31	HIGH
Using the corresponding Savage (2007) P0.002mm values on the Y-Axis: (Depicted as Triangles Δ)									
Sample 1	🟥	29	9,9	19,1	6,7	16	3,1	8	NOT CLASSIFIED
Sample 2	🟦	58,1	26,3	31,8	7,6	56	8,3	32	MEDIUM
Sample 3	🟩	38,6	20,8	17,8	10,1	25	6,7	28	LOW
Sample 4	🟧	65,3	29,5	35,8	15	51	9,3	35	MEDIUM
Sample 5	🟪	57,8	26	31,8	14,5	51	8,3	34	MEDIUM
Sample 6	🟨	56,4	25,5	30,9	14,3	49	7,3	31	MEDIUM

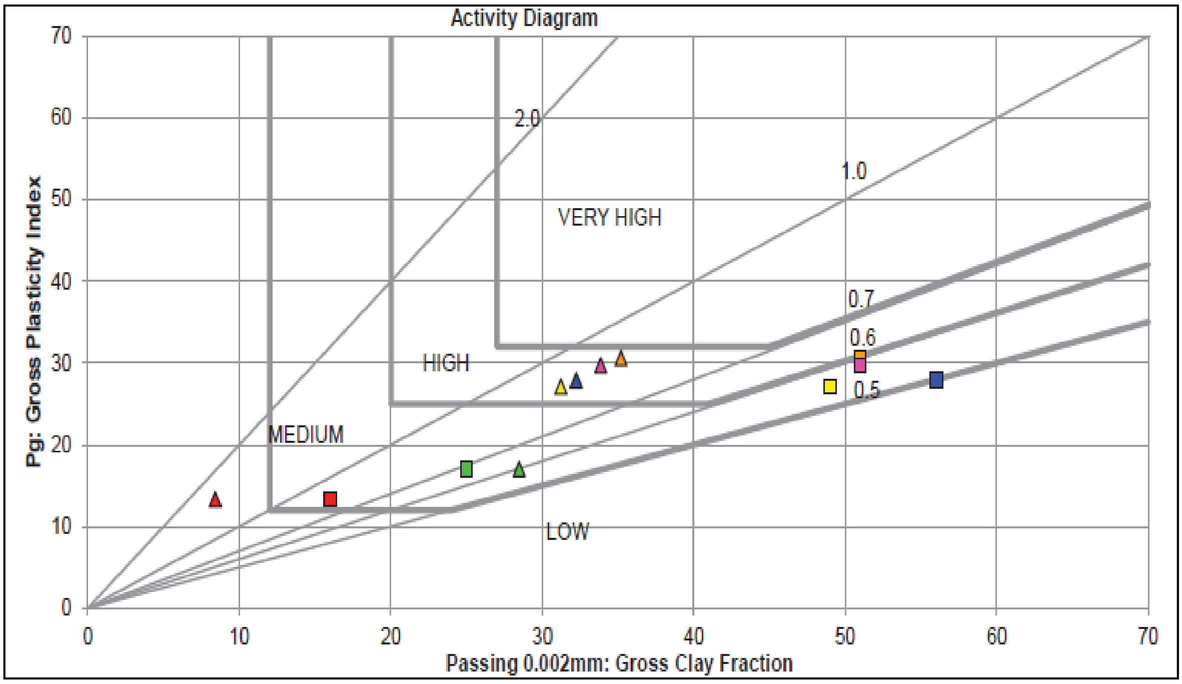
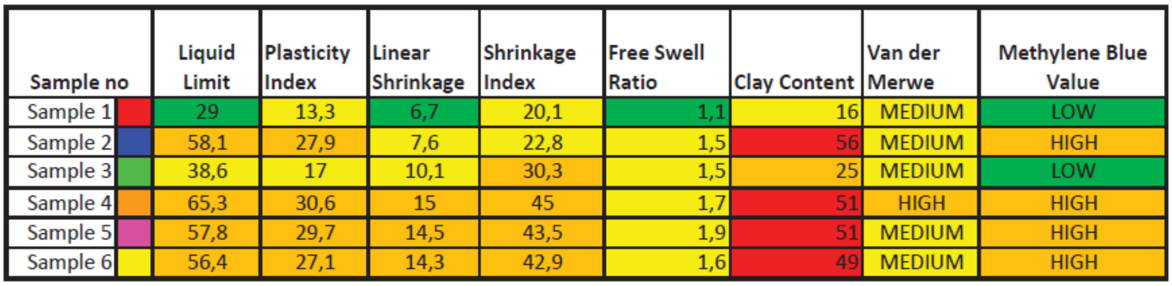


Figure 5.4: Estimated Heave Potential (samples 1-6) after van der Merwe (1964) as modified by Williams and Donaldson (1980)

Table 5.5: Output parameters indicating heave potential based on various methods used for analysis (samples 1-6)

Values depicted as squares □		Liquid Limit	Gross Plasticity Index	Linear Shrinkage	Percentage passing 0.002mm	Potential Heave Classification (hydrometer)	Savage P0.002mm	Potential Heave Classification (Savage)
Sample 1	🟥	29	13,3	6,7	16	MEDIUM	8	LOW
Sample 2	🟦	58,1	27,9	7,6	56	MEDIUM	32	MEDIUM
Sample 3	🟩	38,6	17	10,1	25	MEDIUM	28	HIGH
Sample 4	🟧	65,3	30,6	15	51	HIGH	35	HIGH
Sample 5	🟪	57,8	29,7	14,5	51	MEDIUM	34	HIGH
Sample 6	🟨	56,4	27,1	14,3	49	MEDIUM	31	HIGH



Sample no		Liquid Limit	Plasticity Index	Linear Shrinkage	Shrinkage Index	Free Swell Ratio	Clay Content	Van der Merwe	Methylene Blue Value
Sample 1	🟥	29	13,3	6,7	20,1	1,1	16	MEDIUM	LOW
Sample 2	🟦	58,1	27,9	7,6	22,8	1,5	56	MEDIUM	HIGH
Sample 3	🟩	38,6	17	10,1	30,3	1,5	25	MEDIUM	LOW
Sample 4	🟧	65,3	30,6	15	45	1,7	51	HIGH	HIGH
Sample 5	🟪	57,8	29,7	14,5	43,5	1,9	51	MEDIUM	HIGH
Sample 6	🟨	56,4	27,1	14,3	42,9	1,6	49	MEDIUM	HIGH

POTENTIAL EXPANSIVENESS	Liquid Limit	Gross Plasticity Index	Shrinkage Index	Free Swell Ratio	Clay Content	Linear Shrinkage
Low	20-35	<12	<15	1.0 - 1.5	<12	<7
Med	35-50	12-23	15-30	1.5 - 2.0	12-20	7-14
High	50-70	23-32	30-60	2.0 - 4.0	20-40	14-20
Very High	70-90	>32	>60	> 4.0	>40	>20

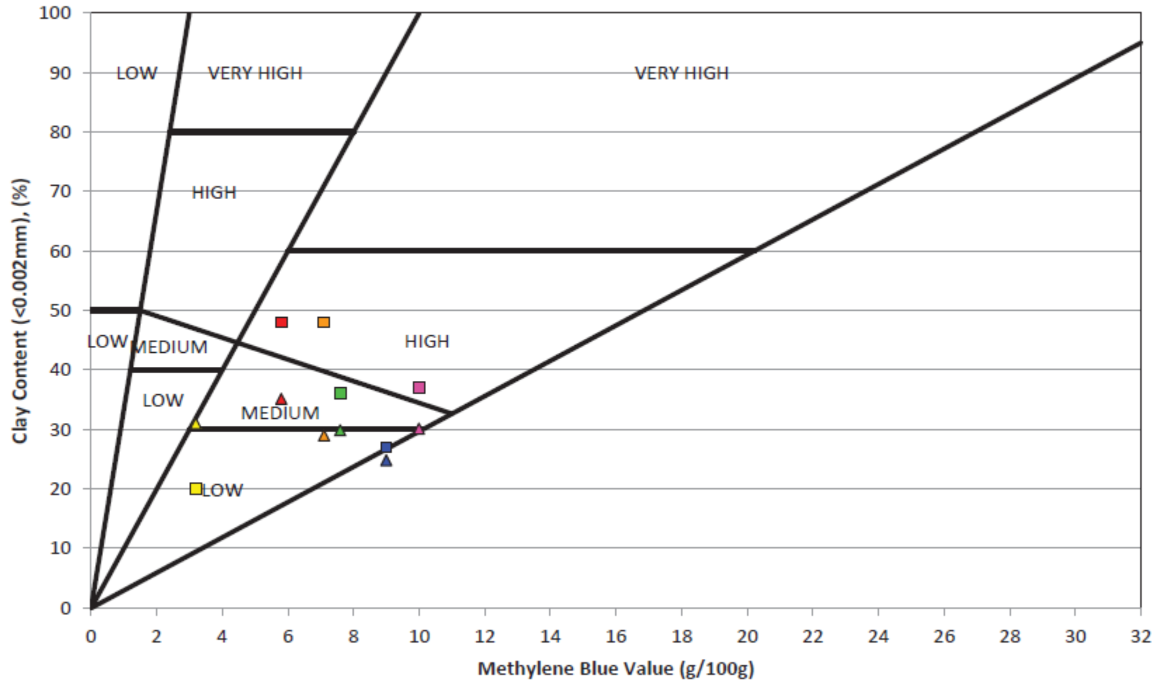


Figure 5.5: Heave potential classification chart (samples 7-12) after Çokça (1991)

Table 5.6: Input parameters to determine heave potential on samples 7-12

Values depicted as squares □		Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage	Percentage passing 0.002mm	Methylene Blue Value	Savage P0.002mm	MBV Classification
Sample 7	🟥	56,5	28,2	28,3	13,6	48	5,8	35	HIGH
Sample 8	🟦	41,3	21,6	19,7	11,5	27	9	25	LOW
Sample 9	🟩	50,9	23,1	27,8	14,5	36	7,6	30	MEDIUM
Sample 10	🟧	46,8	22,3	24,5	11,1	48	7,1	29	HIGH
Sample 11	🟪	55,1	23,8	31,3	13,9	37	10	30	HIGH
Sample 12	🟨	47,7	24,4	23,3	13	20	3,2	31	LOW
Using the corresponding Savage (2007) P0.002mm values on the Y-Axis: (Depicted as Triangles Δ)									
Sample 7	🟥	56,5	28,2	28,3	13,6	48	5,8	35	MEDIUM
Sample 8	🟦	41,3	21,6	19,7	11,5	27	9	25	NOT CLASSIFIED
Sample 9	🟩	50,9	23,1	27,8	14,5	36	7,6	30	LOW
Sample 10	🟧	46,8	22,3	24,5	11,1	48	7,1	29	LOW
Sample 11	🟪	55,1	23,8	31,3	13,9	37	10	30	MEDIUM
Sample 12	🟨	47,7	24,4	23,3	13	20	3,2	31	MEDIUM

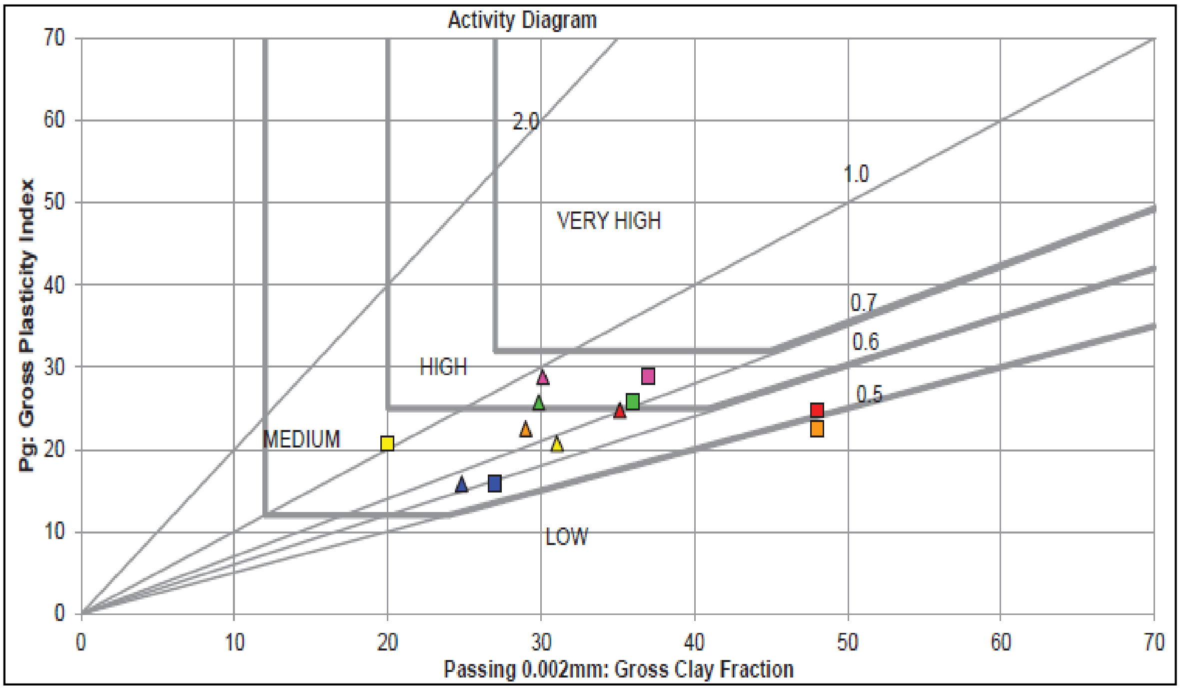


Figure 5.6: Estimated Heave Potential (samples 7-12) after van der Merwe (1964) as modified by Williams and Donaldson (1980)

Table 5.7: Output parameters indicating heave potential based on various methods used for analysis (samples 7-12)

Values depicted as squares □		Liquid Limit	Gross Plasticity Index	Linear Shrinkage	Percentage passing 0.002mm	Potential Heave Classification (hydrometer)	Savage P0.002mm	Potential Heave Classification (Savage)
Sample 7	🟥	56,5	24,7	13,6	48	MEDIUM	35	HIGH
Sample 8	🟦	41,3	15,8	11,5	27	MEDIUM	25	MEDIUM
Sample 9	🟩	50,9	25,7	14,5	36	HIGH	30	HIGH
Sample 10	🟧	46,8	22,5	11,1	48	LOW	29	MEDIUM
Sample 11	🟪	55,1	28,8	13,9	37	HIGH	30	HIGH
Sample 12	🟨	47,7	20,7	13	20	MEDIUM	31	MEDIUM

Sample no		Liquid Limit	Plasticity Index	Linear Shrinkage	Shrinkage Index	Free Swell Ratio	Clay Content	Van der Merwe	Methylene Blue Value
Sample 7	🟥	56,5	24,7	13,6	40,8	1,7	48	MEDIUM	HIGH
Sample 8	🟦	41,3	15,8	11,5	34,5	1,5	27	MEDIUM	LOW
Sample 9	🟩	50,9	25,7	14,5	43,5	1,6	36	HIGH	MEDIUM
Sample 10	🟧	46,8	22,5	11,1	33,3	1,3	48	LOW	HIGH
Sample 11	🟪	55,1	28,8	13,9	41,7	1,5	37	HIGH	HIGH
Sample 12	🟨	47,7	20,7	13	39	1,3	20	MEDIUM	LOW

POTENTIAL EXPANSIVENESS	Liquid Limit	Gross Plasticity Index	Shrinkage Index	Free Swell Ratio	Clay Content	Linear Shrinkage
Low	20-35	<12	<15	1.0 - 1.5	<12	<7
Med	35-50	12-23	15-30	1.5 - 2.0	12-20	7-14
High	50-70	23-32	30-60	2.0 - 4.0	20-40	14-20
Very High	70-90	>32	>60	> 4.0	>40	>20

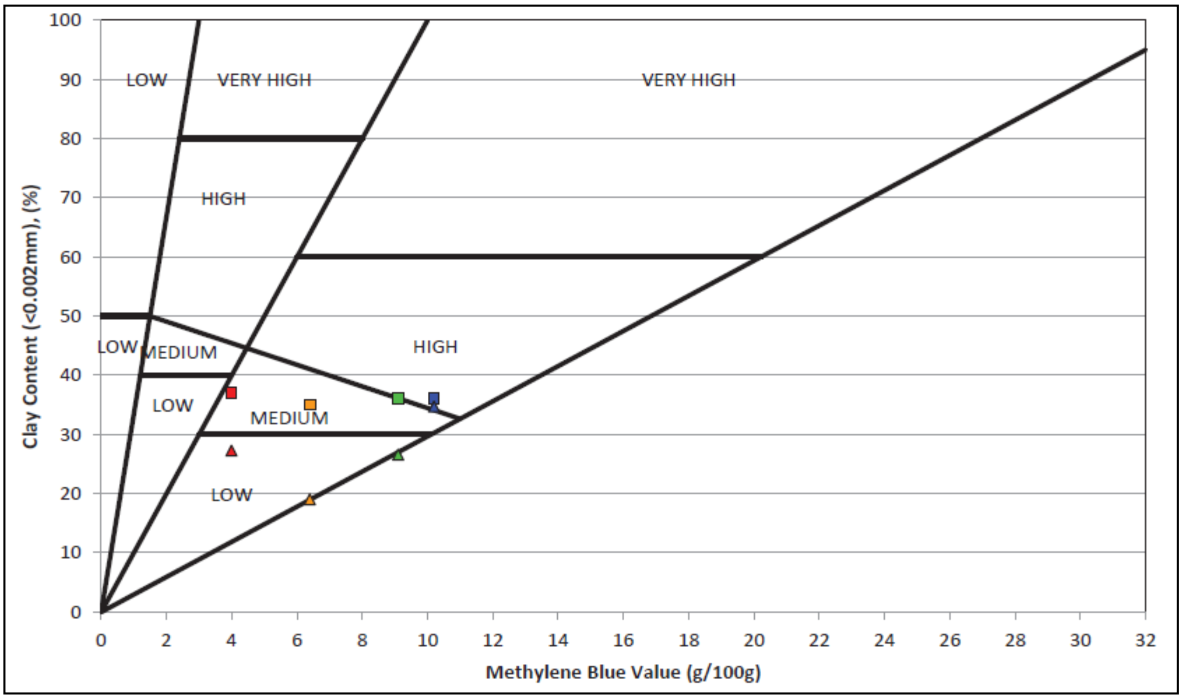


Figure 5.7: Heave potential classification chart (samples 13-16) after Çokça (1991)

Table 5.8: Input parameters to determine heave potential on samples 13-16

Values depicted as squares □		Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage	Percentage passing 0.002mm	Methylene Blue Value	Savage P0.002mm	MBV Classification
Sample 13	🟥	38,5	20,8	17,7	9,9	37	4	27	MEDIUM
Sample 14	🟦	53,5	25,2	28,3	13,9	36	10,2	35	HIGH
Sample 15	🟩	53,8	23,6	30,2	12	36	9,1	27	HIGH
Sample 16	🟧	47,9	21,2	26,7	12	35	6,4	19	MEDIUM
	🟪								
	🟨								
Using the corresponding Savage (2007) P0.002mm values on the Y-Axis: (Depicted as Triangles Δ)									
Sample 13	🟥	38,5	20,8	17,7	9,9	37	4	27	LOW
Sample 14	🟦	53,5	25,2	28,3	13,9	36	10,2	35	HIGH
Sample 15	🟩	53,8	23,6	30,2	12	36	9,1	27	LOW
Sample 16	🟧	47,9	21,2	26,7	12	35	6,4	19	LOW
	🟪								
	🟨								

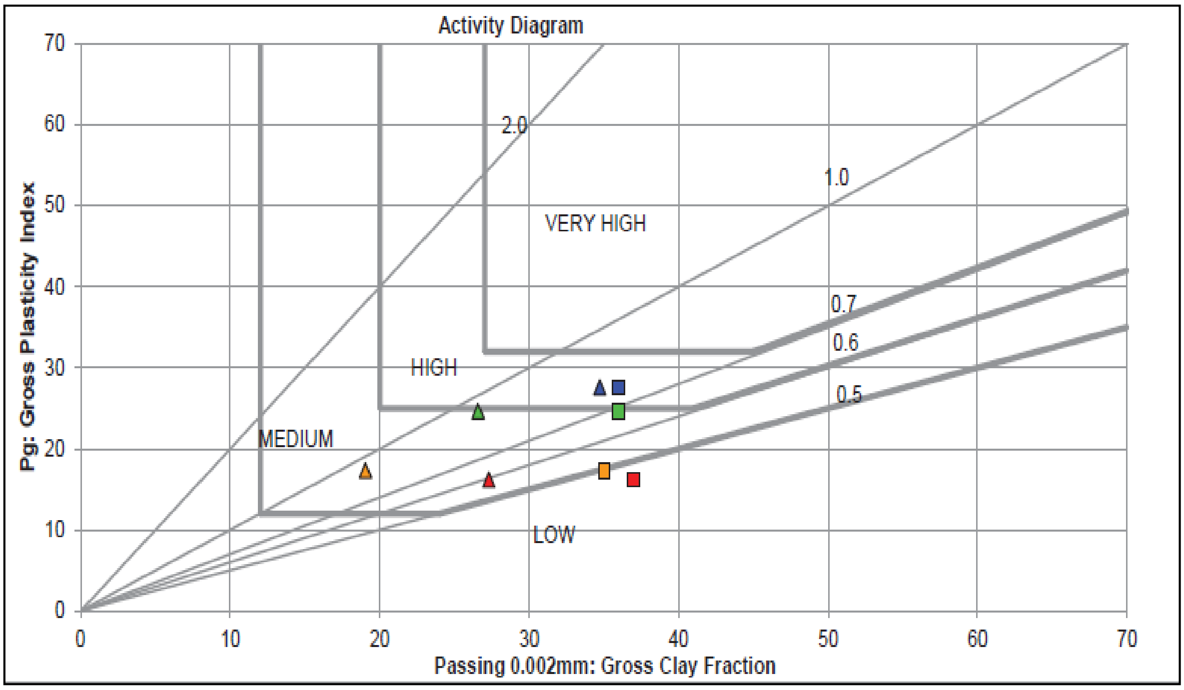


Figure 5.8: Estimated Heave Potential (samples 13-16) after van der Merwe (1964) as modified by Williams and Donaldson (1980)

Table 5.9: Output parameters indicating heave potential based on various methods used for analysis (samples 13-16)

Values depicted as squares □		Liquid Limit	Gross Plasticity Index	Linear Shrinkage	Percentage passing 0.002mm	Potential Heave Classification (hydrometer)	Savage P0.002mm	Potential Heave Classification (Savage)
Sample 13	🟥	38,5	16,2	9,9	37	LOW	27	MEDIUM
Sample 14	🟦	53,5	27,6	13,9	36	HIGH	35	HIGH
Sample 15	🟩	53,8	24,6	12	36	HIGH	27	HIGH
Sample 16	🟧	47,9	17,3	12	35	LOW	19	MEDIUM
	🟪							
	🟨							

Sample no		Liquid Limit	Plasticity Index	Linear Shrinkage	Shrinkage Index	Free Swell Ratio	Clay Content	Van der Merwe	Methylene Blue Value
Sample 13	🟥	38,5	16,2	9,9	29,7	1,4	37	LOW	MEDIUM
Sample 14	🟦	53,5	27,6	13,9	41,7	1,8	36	HIGH	HIGH
Sample 15	🟩	53,8	24,6	12	36	1,5	36	HIGH	HIGH
Sample 16	🟧	47,9	17,3	12	36	1,5	35	LOW	MEDIUM
	🟪								
	🟨								

POTENTIAL EXPANSIVENESS	Liquid Limit	Gross Plasticity Index	Shrinkage Index	Free Swell Ratio	Clay Content	Linear Shrinkage
Low	20-35	<12	<15	1.0 - 1.5	<12	<7
Med	35-50	12-23	15-30	1.5 - 2.0	12-20	7-14
High	50-70	23-32	30-60	2.0 - 4.0	20-40	14-20
Very High	70-90	>32	>60	> 4.0	>40	>20

Table 5.10: The prediction of heave classification based on an average weighted score

Sample no	Liquid Limit	Plasticity Index	Linear Shrinkage	Shrinkage Index	Free Swell Ratio	Clay Content	Van der Merwe	Methylene Blue Value	Gross Methylene Blue Value	Average Weighed Score
Sample 1	29	13.3	6.7	20.1	1.1	16	MEDIUM	LOW	2.2	**2.3**
Sample 2	58.1	27.9	7.6	22.8	1.5	56	MEDIUM	HIGH	7.3	**7.1**
Sample 3	38.6	17	10.1	30.3	1.5	25	MEDIUM	LOW	6.4	**4.6**
Sample 4	65.3	30.6	15	45	1.7	51	HIGH	HIGH	7.9	**8.4**
Sample 5	57.8	29.7	14.5	43.5	1.9	51	MEDIUM	HIGH	7.8	**8.0**
Sample 6	56.4	27.1	14.3	42.9	1.6	49	MEDIUM	HIGH	6.4	**7.6**
Sample 7	56.5	24.7	13.6	40.8	1.7	48	MEDIUM	HIGH	5.1	**7.1**
Sample 8	41.3	15.8	11.5	34.5	1.5	27	MEDIUM	LOW	7.2	**5.0**
Sample 9	50.9	25.7	14.5	43.5	1.6	36	HIGH	MEDIUM	7.0	**7.1**
Sample 10	46.8	22.5	11.1	33.3	1.3	48	LOW	HIGH	6.5	**5.6**
Sample 11	55.1	28.8	13.9	41.7	1.5	37	HIGH	HIGH	9.2	**7.1**
Sample 12	47.7	20.7	13	39	1.3	20	MEDIUM	LOW	2.8	**3.9**
Sample 13	38.5	16.2	9.9	29.7	1.4	37	LOW	MEDIUM	3.7	**3.4**
Sample 14	53.5	27.6	13.9	41.7	1.8	36	HIGH	HIGH	9.9	**7.1**
Sample 15	53.8	24.6	12	36	1.5	36	HIGH	HIGH	7.4	**7.1**
Sample 16	47.9	17.3	12	36	1.5	35	LOW	MEDIUM	4.1	**4.6**

POTENTIAL EXPANSIVENESS	Numerical Value	Liquid Limit	Gross Plasticity Index	Shrinkage Index	Free Swell Ratio	Clay Content	Gross Methylene Blue Value	Linear Shrinkage	Classification based on weighted scoring
Low	1	20-35	<12	<15	1.0 - 1.5	<12	<4	<7	**<3**
Med	4	35-50	12-23	15-30	1.5 - 2.0	12-20	4-7	7-14	**3-6**
High	8	50-70	23-32	30-60	2.0 - 4.0	20-40	7-10	14-20	**6-12**
Very High	16	>70	>32	>60	> 4.0	>40	>10	>20	**>12**

5.6 Results and Discussion

It remains unclear which method produced more favourable and reflective results. Perhaps this will become clear with future research. In order to attach any value to the methods discussed in this chapter, it is highly recommended to compare the analysis of the results with oedometer tests and suction tests performed on the same samples. A reliability based approach is recommended to counter the effects of variability within the soil samples, as encountered with the Steelpoort sample in Chapter 4.

As an interim measure and a future research proposal, it is suggested to calculate a weighted or factored heave prediction using various factors, including the liquid limit, gross plasticity index, linear shrinkage, shrinkage index, free swell ratio, clay content, van der Merwe's method and methylene blue value test. This will not be feasible for use in industry, but should provide good indicators for research purposes.

Some of the above criteria are based on findings made in this study while others have been taken from referenced studies such as those by van der Merwe (1964), Casagrande (1948) and industry standards. It is not meant to serve as definite indications, but as a starting point for future research.

CHAPTER 6: SUMMARY, CONCLUSION AND FUTURE RESEARCH

6.1 Summary

One dimensional heave of expansive soils are typically estimated based on the findings of oedometer tests or suction tests, but these procedures are typically time consuming and expensive. The aim of this study was to investigate the current procedures used to estimate heave potential in clays with specific focus on empirical methods used in practise.

Upon finding the current procedures questionable, it was decided to look at other cost effective methods used internationally. The Free Swell Ratio and Methylene Blue Value tests were investigated and compared to the current procedures hoping that favourable correlation would be achieved.

The correlations achieved were generally scattered and not consistent enough to be considered reliable.

6.2 Conclusion

The hypothesis to this study was that the current empirical procedures used to determine heave potential of clays were not adequate. This was found to be true (see Appendix F, Figure 3.3, section 3.4 and chapter 4); in order to design for clays, a more reliable approach would be required.

It was found that sample preparation played a significant role in the correlations achieved with in-house testing. The author believes it had a significant impact on the poor correlation achieved among the various commercial laboratories in Chapter 4. A parallel study indicated that variance within soils may lead to misinterpretation of the soil type. Sample preparation and variance within the soil proved to substantially influence the Atterberg limits of soils, resulting in questionable input parameters to empirical methods.

The duplicate testing on samples covered in chapter 4 was originally intended to identify problems with the current testing procedures in order to gain a significant starting point to this research. It became a pivotal chapter that changed the direction of the study completely. The variance in results obtained from the leading commercial laboratories was alarming and it was originally thought that the laboratories were at fault. Although it may be true to an extent, variance within the soils sampled was noted in a parallel study by Dr Philip Stott.

Hydrometer analysis is used to determine the clay fraction of soils which generally serves as a vital input parameter in almost all empirical methods. Hydrometer analysis has been proven to be questionable which puts all the involved empirical methods in doubt too.

Atterberg limits and particle size distribution form critical input parameters in a majority of the empirical methods studied
Based on the findings made in this study, it is evident that empirical methods do not produce a reliable prediction on the behaviour of clays. It may be worth investigating a weighted approach where several empirical methods are compared and factored as described in chapter 5, but it is recommended that future studies venture towards direct measurement tests such as soil suction or sedimentation tests.

6.3 Future Studies

Future study in the use of methylene blue value tests to determine heave potential may have a positive contribution to the industry, as well as Free Swell Ratio tests. This study briefly touched on both, but the focus was primarily on procedures currently used in South Africa.

Several studies have been done on the fall cone test as a replacement to the Casagrande Liquid Limit test internationally and it is widely considered to be a superior test. This needs to be researched in a South African context. With research on the fall cone method, future research on Savage's method may also be beneficial. Savage relied on information provided by Cornell University; perhaps the mineralogical properties of kaolinite, illite and montmorillonite may be further studied to improve Savage's method.

Generally the author believes that studies into direct sedimentation and direct measurement of the clay fraction using electron microscopy holds value as well as further study into the variance within soil types.

CHAPTER 7: REFERENCES

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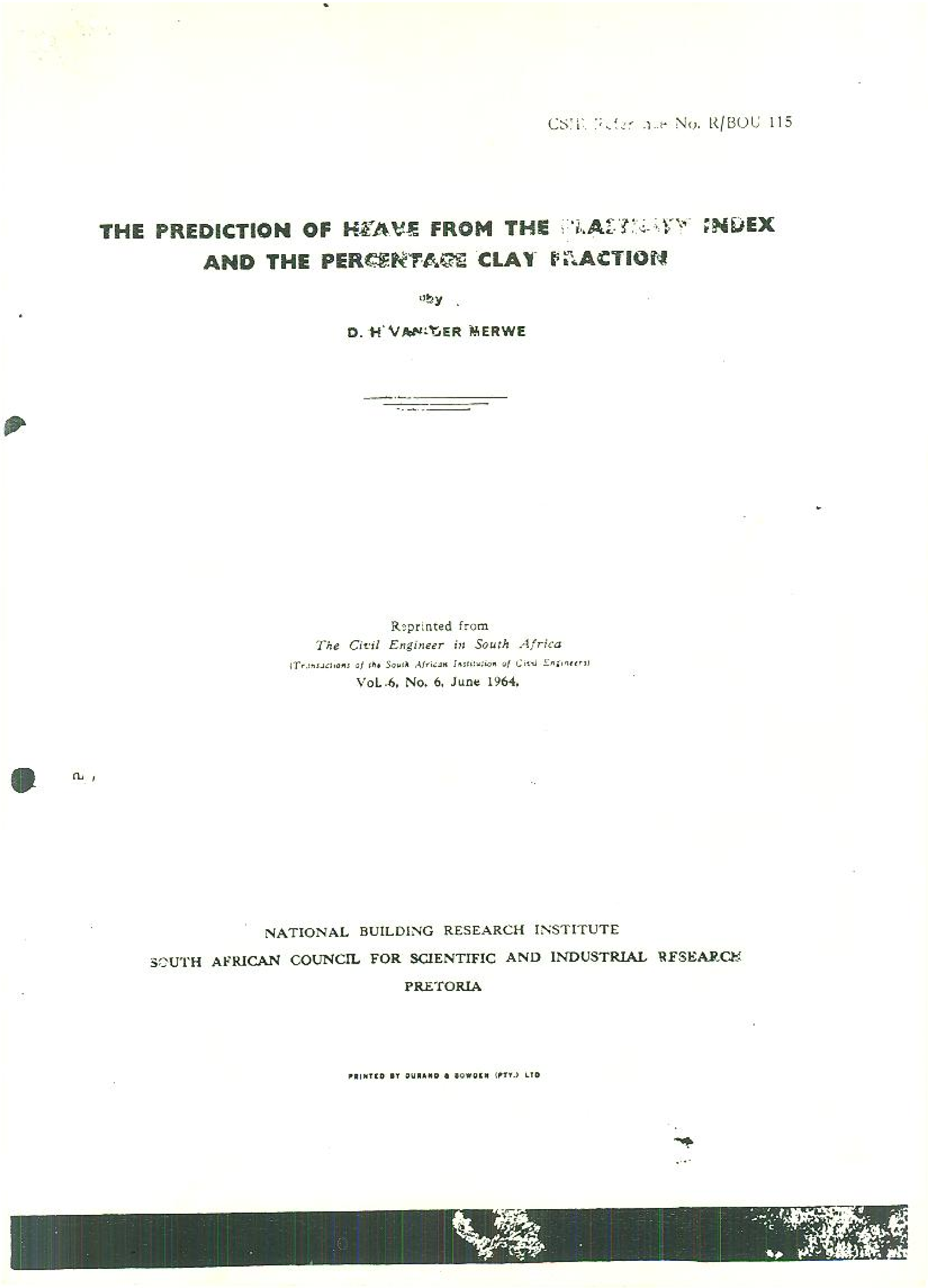
APPENDIX A: VAN DER MERWE’S 1964 ARTICLE



CSIR Reference No. R/BOU 115

THE PREDICTION OF HEAVE FROM THE PLASTICITY INDEX AND THE PERCENTAGE CLAY FRACTION

by

D. H. VAN DER MERWE

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NATIONAL BUILDING RESEARCH INSTITUTE
SOUTH AFRICAN COUNCIL FOR SCIENTIFIC AND INDUSTRIAL RESEARCH
PRETORIA

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A Paper by D. H. VAN DER MERWE*, M.Sc.(Geology)
(Participant, Division of Soil Mechanics and Foundation Engineering)

THE PREDICTION OF HEAVE FROM THE PLASTICITY INDEX AND PERCENTAGE CLAY FRACTION OF SOILS

SYNOPSIS

THE heave to be expected under a building may be estimated by using a simple formula based on the results of Atterberg limit and particle size determinations. The results of these tests are used to classify the soil into four degrees of potential expansiveness to each of which a figure for potential heave under normal conditions is assigned. Because the amount of heave will decrease with depth below the soil surface, reduction factors are introduced. In this way the potential heave of any layer can be calculated and by the summation of the results for the whole profile the total heave for the site can be estimated accurately enough for the purposes of foundation and structural design.

The formula referred to is:

$$\text{total heave} = \sum_{D=1}^{D=n} F_D \cdot P.E._D$$

where $P.E.$ is potential expansiveness, determined from the plasticity index and percentage clay fraction, and F is the factor by which the heave decreases with depth. F is obtained from the formula:

$$D = 20 \log_{10} F$$

where D is negative depth in feet.

It is assumed that soil at ground surface will heave 1 in. ft, ½ in. ft, ¼ in. ft and 0 in. ft depth of expansive soil when its potential expansiveness is *very high, high, medium* and *low*, respectively.

Introduction

A KNOWLEDGE of the total amount of heave that may be expected under a building will be a great advantage for designers of structures on expansive soils. The two major factors that have an influence on the amount of heave are *potential expansiveness* of the soil, and the amount of *moisture change* which will occur under a structure. At the present time the latter factor is difficult to assess. The double oedometer method[2], which is widely used in South Africa, attempts to assess both of these factors simultaneously by measuring the amount of heave which will occur if the soil moisture is increased from the natural moisture content to the fully saturated condition. While this test is frequently applied for large projects it has not been used for smaller undertakings because of the expense involved in obtaining undisturbed samples and carrying out the tests.

A simple classification[4] based on the relationship between the plasticity index of the whole sample and the percentage clay fraction has been used successfully to classify soil into *very high, high, medium* and *low* degrees of potential expansiveness. (See Fig. 1.)

An attempt has now been made to correlate these degrees of expansiveness with a prediction of movement. It must be realized however that the accuracy of this prediction will be governed by the actual moisture

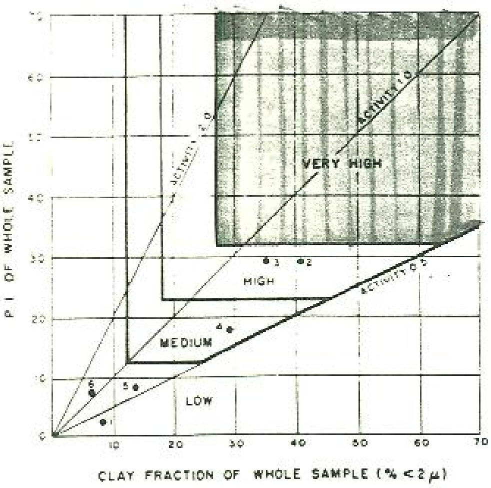

Fig. 1
Determination of potential expansiveness of soils

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*Senior Research Officer, Material Building Research Institute, C.S.I.R., Pretoria.

changes that occur in the soil. Thus, although the test may show highly expansive soil, little movement will take place if the natural moisture content is high or if a high water-table is present. Similarly, considerable movement will occur when the soil is in a very dry condition at the time of construction. This factor can, however, be taken into consideration when the profile is examined.

Potential total heave

Unit heave and variation with depth

From double oedometer tests on unloaded soil and field observations, Jennings and Kerrich[3] have arrived at figures for *unit heave* (inches heave per inch depth of expansive soil). Unit heave, which is dependent on the thickness of non-expansive overburden, is taken as being a maximum at the upper boundary of the expansive soil layer and decreasing with depth along a parabolic curve, which has to be determined for every site by carrying out a number of double oedometer tests.

In this paper the unit heave at ground surface is assumed to be 1 in. for each foot depth of very highly expansive soil, $\frac{1}{2}$ in. per foot for highly expansive, $\frac{1}{4}$ in. per foot for medium expansive, and 0 in. per foot for low or non-expansive soil. The potential expansiveness varies, however, from layer to layer in a single profile, and this variation should be taken into account.

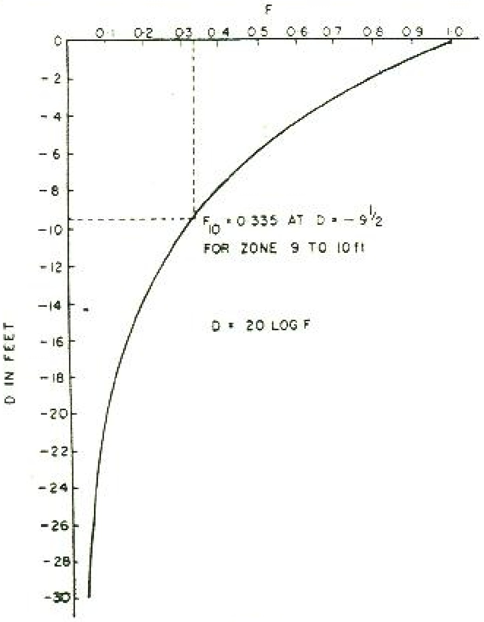


Fig. 2

Curve showing relative change in potential heave with depth

Determination of potential heave

In a single profile consisting of potentially expansive soil the potential heave decreases with depth due to increases in the weight of overlying soil, and due to decreases in changes in moisture content and other factors. By studying the *heave : depth* plots for depth points at Vereeniging and Odendaalsrust a curve was derived that gave approximately the change in potential heave with depth. The equation of the curve, which is shown in Fig. 2, is:

$$D = k \log F$$

where D is negative depth of soil in feet, F a factor indicating the relative decrease in heave at depth D compared with that at the surface, and k is a constant. It was found that when $k = 20$, the calculated heave at the sites mentioned was about the same as the mean maximum heave measured on level pegs on houses.

The values of F with depth are given in Table I.

TABLE I

Value of factor F with depth D from the relation $D = 20 \log F$

Depth in ft	Mean value of F		Depth in ft	Mean value of F	
0 — 1	F_1	0·943	15 — 16	F_{16}	0·168
1 — 2	F_2	0·842	16 — 17	F_{17}	0·150
2 — 3	F_3	0·750	17 — 18	F_{18}	0·133
3 — 4	F_4	0·668	18 — 19	F_{19}	0·119
4 — 5	F_5	0·596	19 — 20	F_{20}	0·106
5 — 6	F_6	0·531	20 — 21	F_{21}	0·094
6 — 7	F_7	0·473	21 — 22	F_{22}	0·084
7 — 8	F_8	0·422	22 — 23	F_{23}	0·075
8 — 9	F_9	0·376	23 — 24	F_{24}	0·067
9 — 10	F_{10}	0·335	24 — 25	F_{25}	0·060
10 — 11	F_{11}	0·298	25 — 26	F_{26}	0·053
11 — 12	F_{12}	0·266	26 — 27	F_{27}	0·047
12 — 13	F_{13}	0·237	27 — 28	F_{28}	0·042
13 — 14	F_{14}	0·211	28 — 29	F_{29}	0·038
14 — 15	F_{15}	0·188	29 — 30	F_{30}	0·034

If the potential expansiveness ($P.E.$) of every soil layer is known, the total heave of a soil profile extending to a depth of n feet may be determined by:

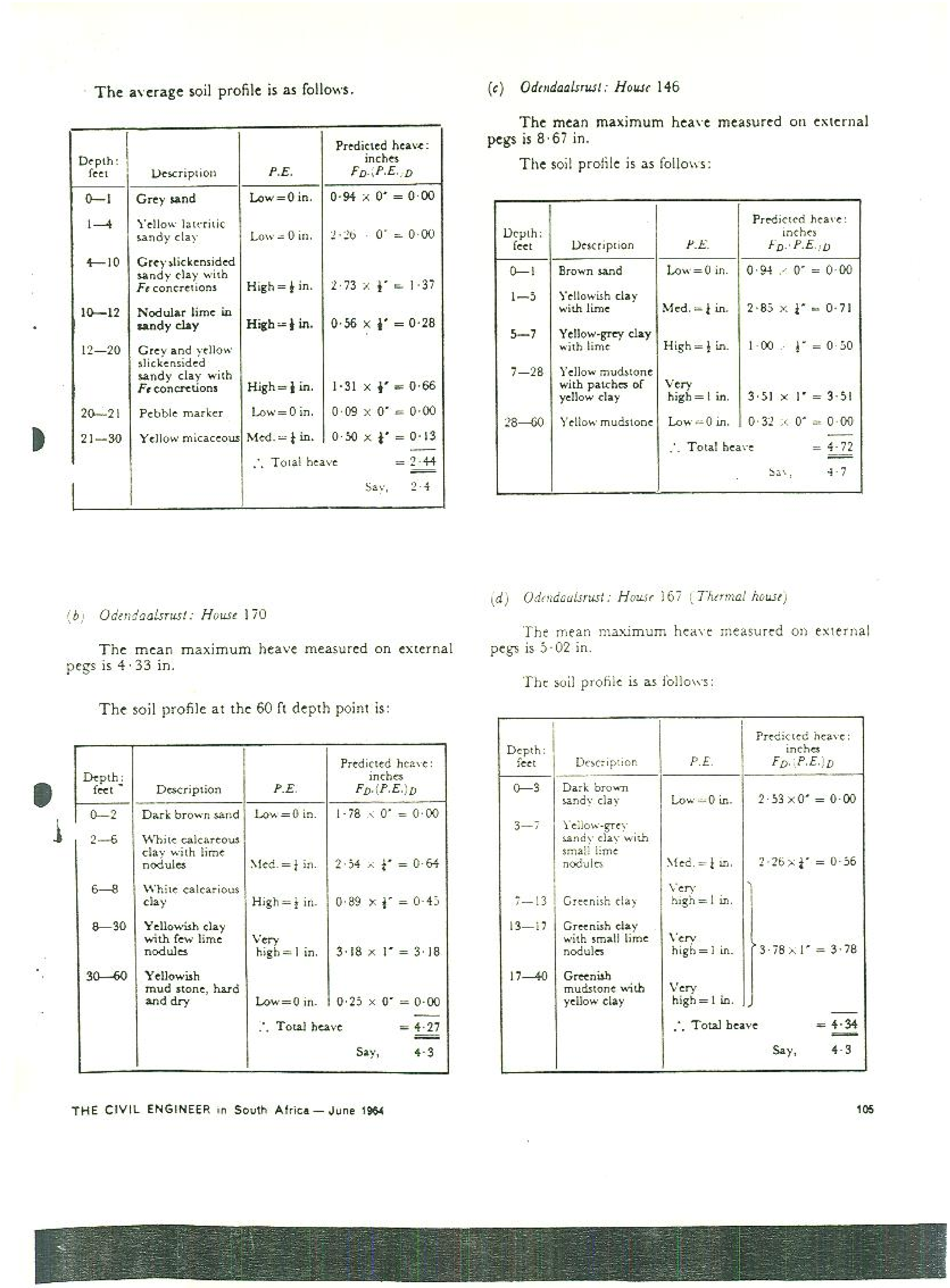
$$\text{total heave} = \sum_{D=1}^{D=n} F_D \cdot (P.E.)_D$$

Examples of estimation of heave

The method of total heave prediction described above has been applied to data on various soil profiles in the Transvaal and Orange Free State. The data and calculated heave values are given in the tabulations which follow. In some of the cases the total heave has actually been measured and these values are also given for comparison. In other cases the predicted heaves obtained are compared with the published predictions given by other observers using other methods.

(*a*) *Leeuhof: Thermal house*

The mean maximum heave measured on external pegs is 2·66 in.



The average soil profile is as follows.

Depth: feet	Description	*P.E.*	Predicted heave: inches $F_D.(P.E.)_D$
0—1	Grey sand	Low = 0 in.	0·94 × 0″ = 0·00
1—4	Yellow lateritic sandy clay	Low = 0 in.	2·26 × 0″ = 0·00
4—10	Grey slickensided sandy clay with *Fe* concretions	High = ½ in.	2·73 × ½″ = 1·37
10—12	Nodular lime in sandy clay	High = ½ in.	0·56 × ½″ = 0·28
12—20	Grey and yellow slickensided sandy clay with *Fe* concretions	High = ½ in.	1·31 × ½″ = 0·66
20—21	Pebble marker	Low = 0 in.	0·09 × 0″ = 0·00
21—30	Yellow micaceous	Med. = ¼ in.	0·50 × ¼″ = 0·13
		∴ Total heave	= 2·44
		Say,	2·4

(*b*) *Odendaalsrust: House* 170

The mean maximum heave measured on external pegs is 4·33 in.

The soil profile at the 60 ft depth point is:

Depth: feet	Description	*P.E.*	Predicted heave: inches $F_D.(P.E.)_D$
0—2	Dark brown sand	Low = 0 in.	1·78 × 0″ = 0·00
2—6	White calcareous clay with lime nodules	Med. = ¼ in.	2·54 × ¼″ = 0·64
6—8	White calcarious clay	High = ½ in.	0·89 × ½″ = 0·45
8—30	Yellowish clay with few lime nodules	Very high = 1 in.	3·18 × 1″ = 3·18
30—60	Yellowish mud stone, hard and dry	Low = 0 in.	0·25 × 0″ = 0·00
		∴ Total heave	= 4·27
		Say,	4·3

(*c*) *Odendaalsrust: House* 146

The mean maximum heave measured on external pegs is 8·67 in.

The soil profile is as follows:

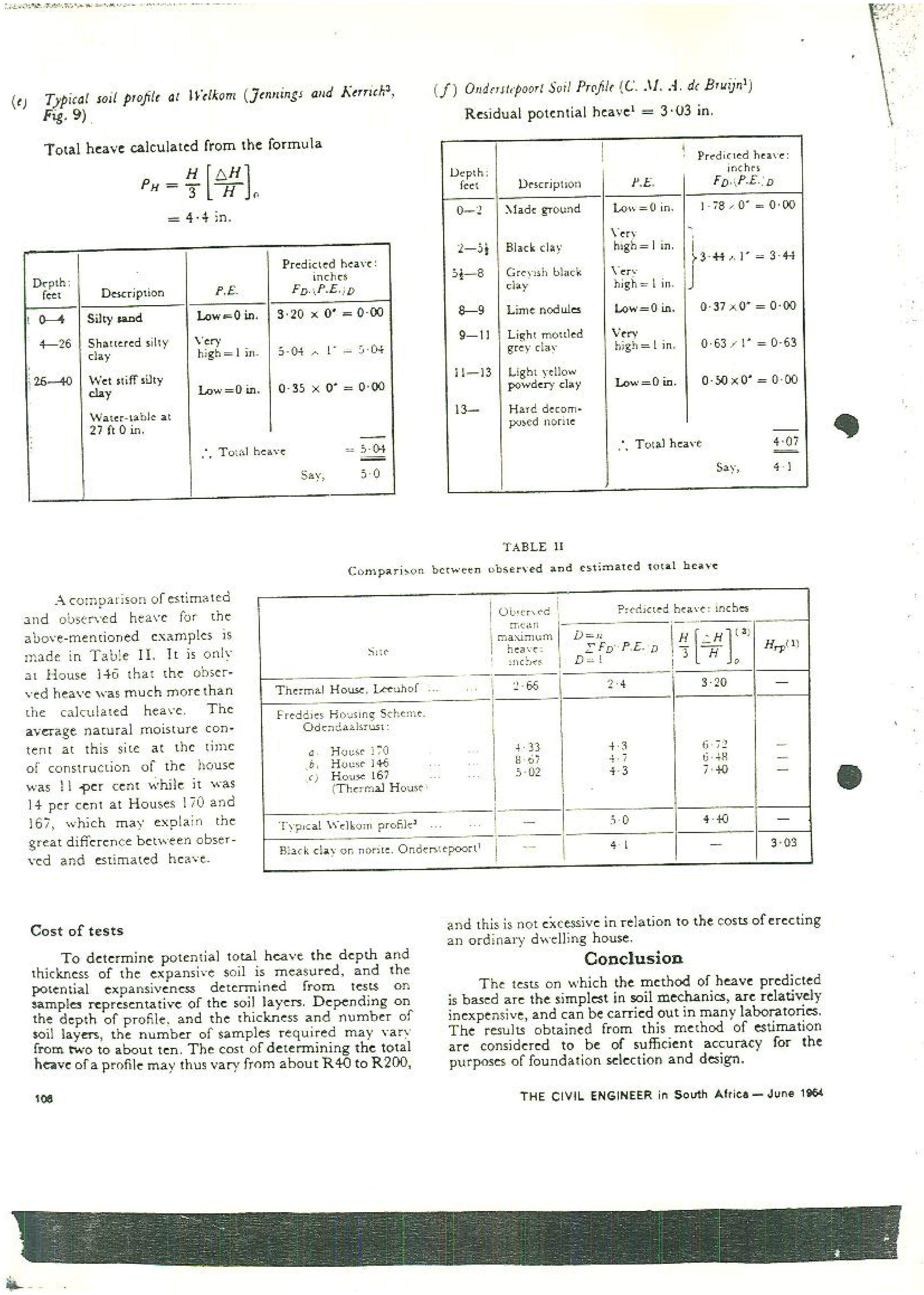
Depth: feet	Description	*P.E.*	Predicted heave: inches $F_D.(P.E.)_D$
0—1	Brown sand	Low = 0 in.	0·94 × 0″ = 0·00
1—5	Yellowish clay with lime	Med. = ¼ in.	2·85 × ¼″ = 0·71
5—7	Yellow-grey clay with lime	High = ½ in.	1·00 × ½″ = 0·50
7—28	Yellow mudstone with patches of yellow clay	Very high = 1 in.	3·51 × 1″ = 3·51
28—60	Yellow mudstone	Low = 0 in.	0·32 × 0″ = 0·00
		∴ Total heave	= 4·72
		Say,	4·7

(*d*) *Odendaalsrust: House* 167 (*Thermal house*)

The mean maximum heave measured on external pegs is 5·02 in.

The soil profile is as follows:

Depth: feet	Description	*P.E.*	Predicted heave: inches $F_D.(P.E.)_D$
0—3	Dark brown sandy clay	Low = 0 in.	2·53 × 0″ = 0·00
3—7	Yellow-grey sandy clay with small lime nodules	Med. = ¼ in.	2·26 × ¼″ = 0·56
7—13	Greenish clay	Very high = 1 in.	3·78 × 1″ = 3·78
13—17	Greenish clay with small lime nodules	Very high = 1 in.	
17—40	Greenish mudstone with yellow clay	Very high = 1 in.	
		∴ Total heave	= 4·34
		Say,	4·3



(e) *Typical soil profile at Welkom (Jennings and Kerrich*[2], *Fig. 9)*

Total heave calculated from the formula

$$P_H = \frac{H}{3}\left[\frac{\Delta H}{H}\right]_o$$

$$= 4{\cdot}4 \text{ in.}$$

Depth: feet	Description	P.E.	Predicted heave: inches $F_D\cdot(P.E.)_D$
0—4	Silty sand	Low = 0 in.	3·20 × 0″ = 0·00
4—26	Shattered silty clay	Very high = 1 in.	5·04 × 1″ = 5·04
26—40	Wet stiff silty clay	Low = 0 in.	0·35 × 0″ = 0·00
	Water-table at 27 ft 0 in.		
		∴ Total heave	= 5·04
		Say,	5·0

(*f*) *Onderstepoort Soil Profile (C. M. A. de Bruijn*[1]*)*

Residual potential heave[1] = 3·03 in.

Depth: feet	Description	P.E.	Predicted heave: inches $F_D\cdot(P.E.)_D$
0—2	Made ground	Low = 0 in.	1·78 × 0″ = 0·00
2—5½	Black clay	Very high = 1 in.	3·44 × 1″ = 3·44
5½—8	Greyish black clay	Very high = 1 in.	
8—9	Lime nodules	Low = 0 in.	0·37 × 0″ = 0·00
9—11	Light mottled grey clay	Very high = 1 in.	0·63 × 1″ = 0·63
11—13	Light yellow powdery clay	Low = 0 in.	0·50 × 0″ = 0·00
13—	Hard decomposed norite		
		∴ Total heave	4·07
		Say,	4·1

A comparison of estimated and observed heave for the above-mentioned examples is made in Table II. It is only at House 146 that the observed heave was much more than the calculated heave. The average natural moisture content at this site at the time of construction of the house was 11 per cent while it was 14 per cent at Houses 170 and 167, which may explain the great difference between observed and estimated heave.

TABLE II

Comparison between observed and estimated total heave

Site	Observed mean maximum heave: inches	Predicted heave: inches		
		$\sum_{D=1}^{D=n} F_D\cdot(P.E.)_D$	$\frac{H}{3}\left[\frac{\Delta H}{H}\right]_o$ (3)	H_{rp}(1)
Thermal House, Leeuhof	2·66	2·4	3·20	—
Freddies Housing Scheme, Odendaalsrust:				
(a) House 170 ... ... (b) House 146 ... ... (c) House 167 ... ... (Thermal House)	4·33 8·67 5·02	4·3 4·7 4·3	6·72 6·48 7·40	— — —
Typical Welkom profile[2]	—	5·0	4·40	—
Black clay on norite, Onderstepoort[1]	—	4·1	—	3·03

Cost of tests

To determine potential total heave the depth and thickness of the expansive soil is measured, and the potential expansiveness determined from tests on samples representative of the soil layers. Depending on the depth of profile, and the thickness and number of soil layers, the number of samples required may vary from two to about ten. The cost of determining the total heave of a profile may thus vary from about R40 to R200, and this is not excessive in relation to the costs of erecting an ordinary dwelling house.

Conclusion

The tests on which the method of heave predicted is based are the simplest in soil mechanics, are relatively inexpensive, and can be carried out in many laboratories. The results obtained from this method of estimation are considered to be of sufficient accuracy for the purposes of foundation selection and design.

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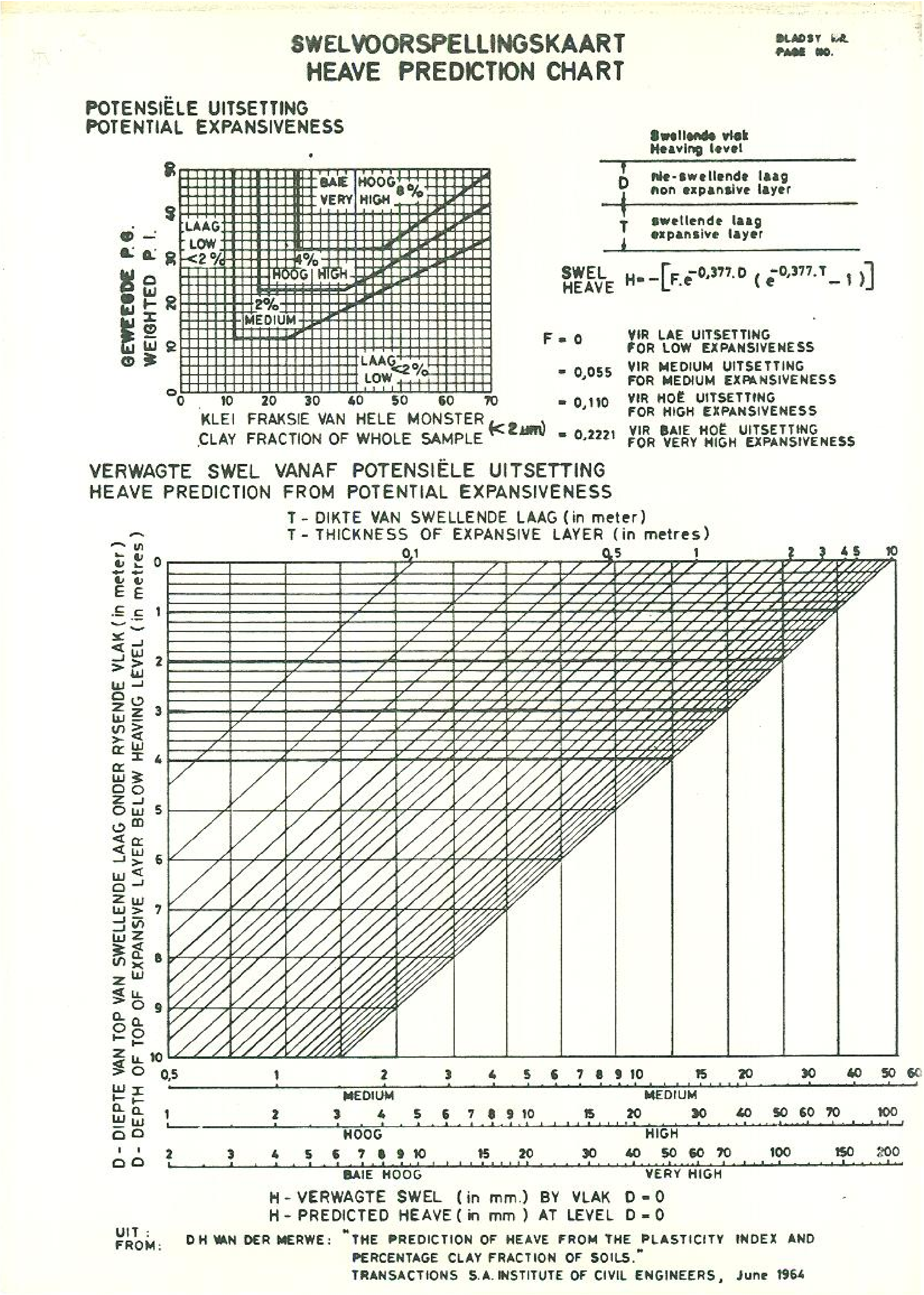


SWELVOORSPELLINGSKAART
HEAVE PREDICTION CHART

BLADSY NR.
PAGE NO.

POTENSIËLE UITSETTING
POTENTIAL EXPANSIVENESS

SWEL
HEAVE $H = -\left[F.e^{-0,377.D}\left(e^{-0,377.T} - 1\right)\right]$

F = 0 VIR LAE UITSETTING
FOR LOW EXPANSIVENESS

= 0,055 VIR MEDIUM UITSETTING
FOR MEDIUM EXPANSIVENESS

= 0,110 VIR HOË UITSETTING
FOR HIGH EXPANSIVENESS

= 0,2221 VIR BAIE HOË UITSETTING
FOR VERY HIGH EXPANSIVENESS

VERWAGTE SWEL VANAF POTENSIËLE UITSETTING
HEAVE PREDICTION FROM POTENTIAL EXPANSIVENESS

UIT :
FROM: D H VAN DER MERWE: "THE PREDICTION OF HEAVE FROM THE PLASTICITY INDEX AND PERCENTAGE CLAY FRACTION OF SOILS."
TRANSACTIONS S.A. INSTITUTE OF CIVIL ENGINEERS, June 1964

APPENDIX B: FREE SWELL RATIO TEST RESULTS

SOILS MECHANICS RESEARCH GROUP
Central University of Technology, Free State

FREE SWELL RATIO

WS BADENHORST

Sample : Soil 1
Iliso Consulting, 1 Kgotsong, TP1, KG 01, 0-1820

Date :
2015-05-27

Time:
24 hours

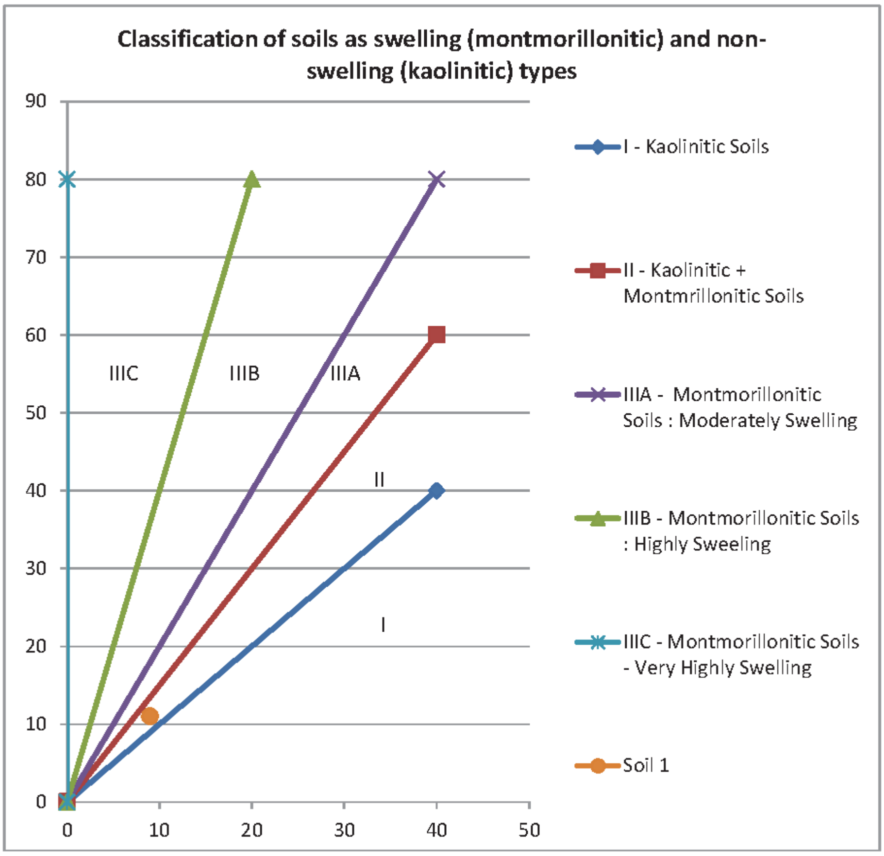
Photo:

Reading (cc^3): W1 : 11 W2 : 11 K1 : 9

Classification of soils based on free swell ratio

Water (cc^3)		Kerosene (cc^3)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	11	K1	9	1,22	Low	Mixture of swelling	Kaolinitic and Montmorillonitic
W2	11						
Ave	11						

The classification of the soil tested : The expansivity was Low, the clay was Mixture of swelling and the dominant was Kaolinitic and Montmorillonitic


Classification of soils as swelling (montmorillonitic) and non-swelling (kaolinitic) types

Classification
Class II : Kaolinitic Montmorillonitic Soils

The classification of the soil tested was Class II : Kaolinitic Montmorillonitic Soils

SOILS MECHANICS RESEARCH GROUP

FREE SWELL RATIO

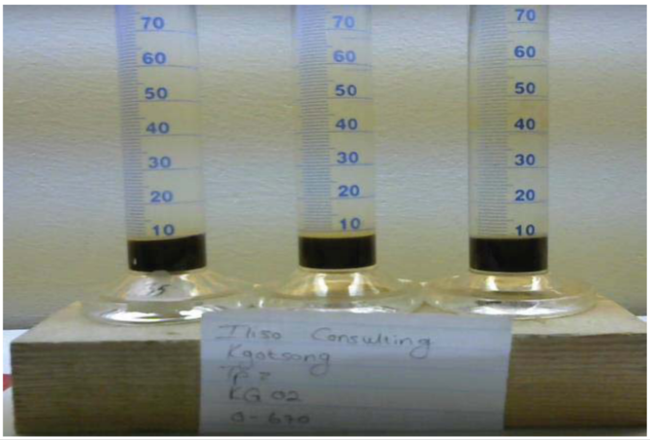
WS BADENHORST

Sample : Soil 2
Iliso Consulting, 2 Kgotsong, TP 2, KG 02, 0-670

Date :
2015-06-01

Time:
24 hours

Photo:

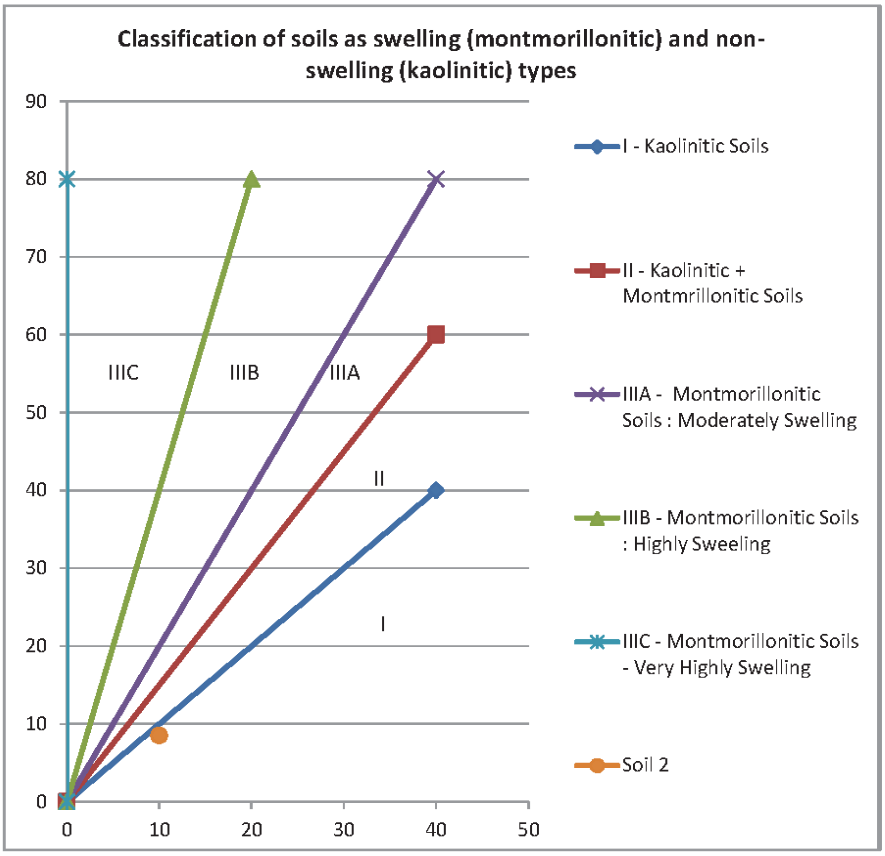


Reading (cc³): W1 : 9 W2 : 8 K1 : 10

Classification of soils based on free swell ratio

Water (cc³)		Kerosene (cc³)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	9	K1	10	0,85	Negligible	Nonswelling	Kaolinitic
W2	8						
Ave	8,5						

The classification of the soil tested : The expansivity was Negligible, the clay was Nonswelling and the dominant was Kaolinitic



Classification
Class I : Kaolinitic Soils

The classification of the soil tested was Class I : Kaolinitic Soils

SOILS MECHANICS RESEARCH GROUP
Central University of Technology, Free State

FREE SWELL RATIO

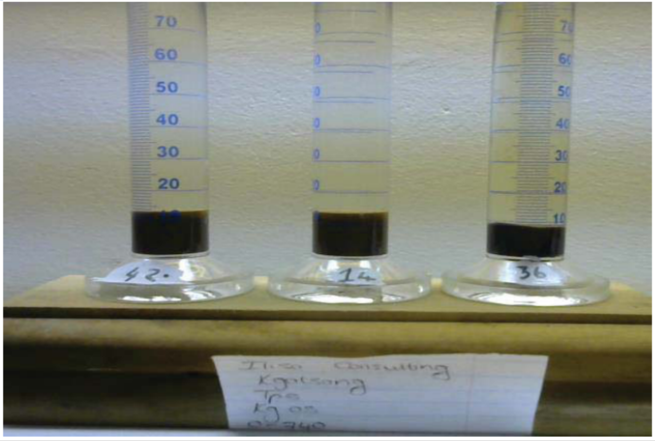
WS BADENHORST

Sample : Soil 3
Iliso Consulting, 3
Kgotsong, TP5,
KG05, 0-740

Date :
2015-05-25

Time:
24 hours

Photo:

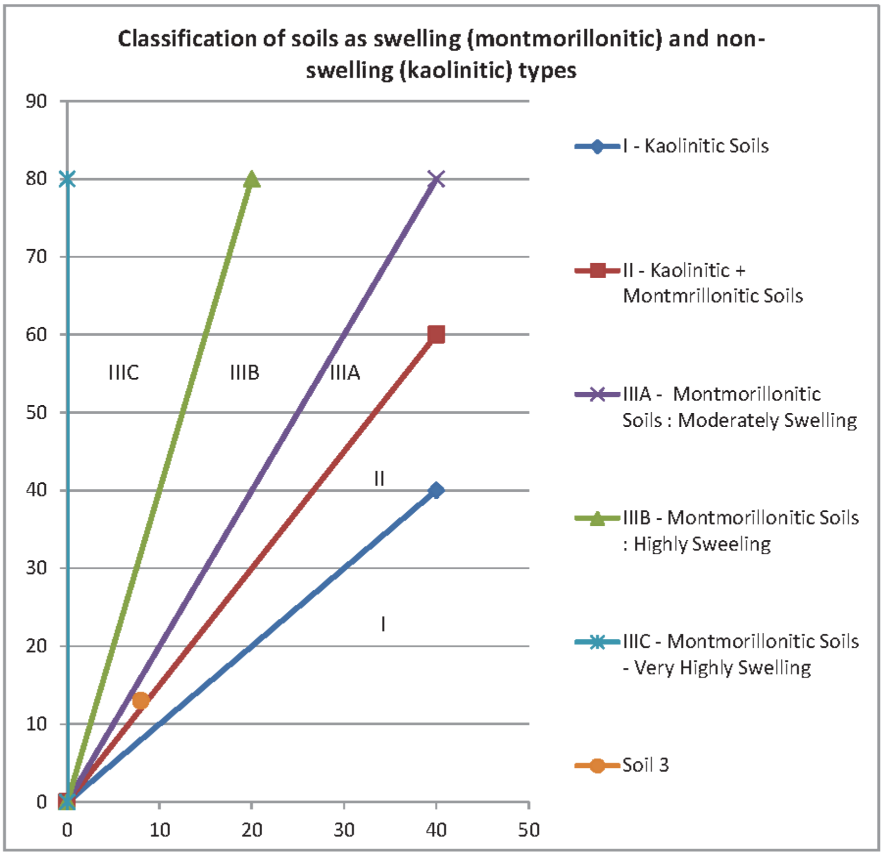


Reading (cc^3): W1 : 13 W2 : 13 K1 : 8

Classification of soils based on free swell ratio

Water (cc^3)		Kerosene (cc^3)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	13	K1	8	1,63	Moderate	Swelling	Montmorillonitic
W2	13						
Ave	13						

The classification of the soil tested : The expansivity was Moderate, the clay was Swelling and the dominant was Montmorillonitic



Classification
Class IIIA : Montmorillonitic Soils - Moderately

The classification of the soil tested was Class IIIA : Montmorillonitic Soils - Moderately Swelling

SOILS MECHANICS RESEARCH GROUP

Central University of Technology, Free State

FREE SWELL RATIO

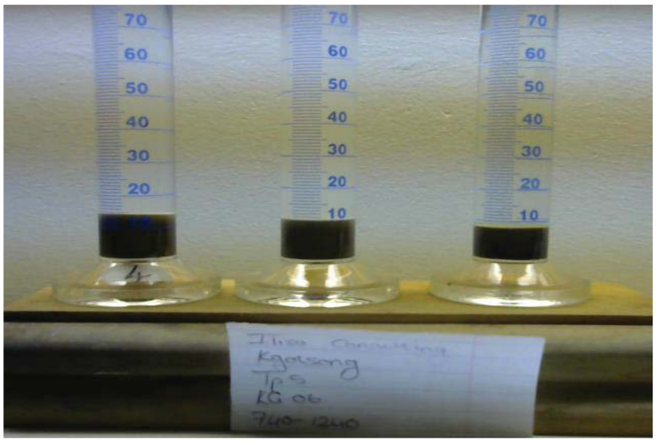
WS BADENHORST

Sample : Soil 4
Iliso Consulting, 4
Kgotsong , TP 5, KG
06, 740-1240

Date :
2015-05-22

Time:
24 hours

Photo:

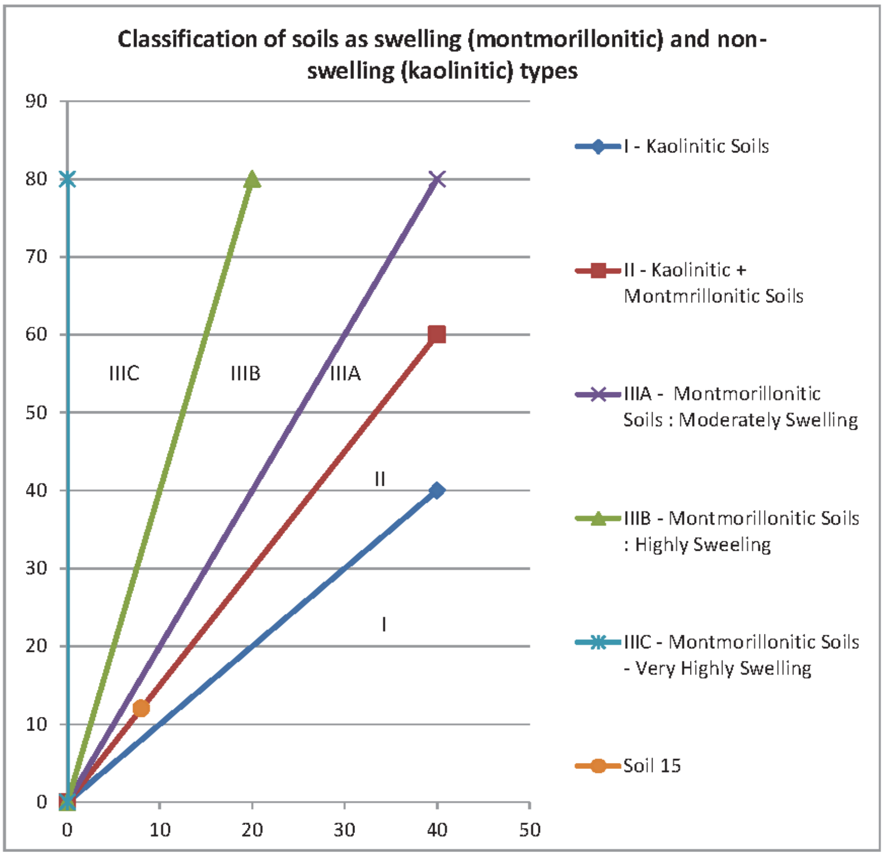


Reading (cc^3): W1 : 14 W2 : 10 K1 : 8

Classification of soils based on free swell ratio

Water (cc^3)		Kerosene (cc^3)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	14	K1	8	1,50	Moderate	Swelling	Montmorillonitic
W2	10						
Ave	12						

The classification of the soil tested : The expansivity was Moderate, the clay was Swelling and the dominant was Montmorillonitic



Classification
Class IIIA : Montmorillonitic Soils - Moderately

The classification of the soil tested was Class IIIA : Montmorillonitic Soils - Moderately Swelling

SOILS MECHANICS RESEARCH GROUP
Central University of Technology, Free State

FREE SWELL RATIO

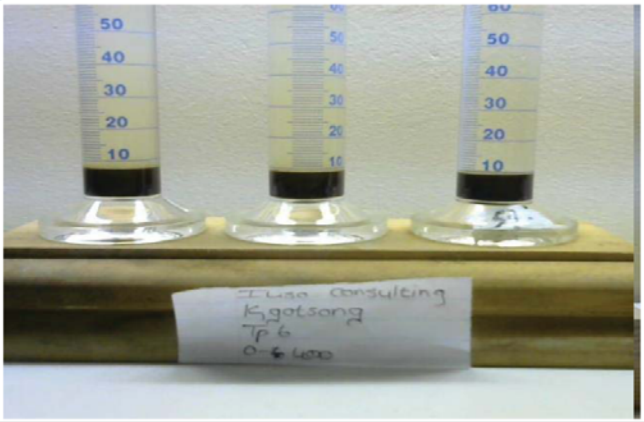
WS BADENHORST

Sample : Soil 5 Iliso Consulting, 5 Kgotsong, TP 6, 0-400

Date : 2015-05-28

Time: 24 hours

Photo:

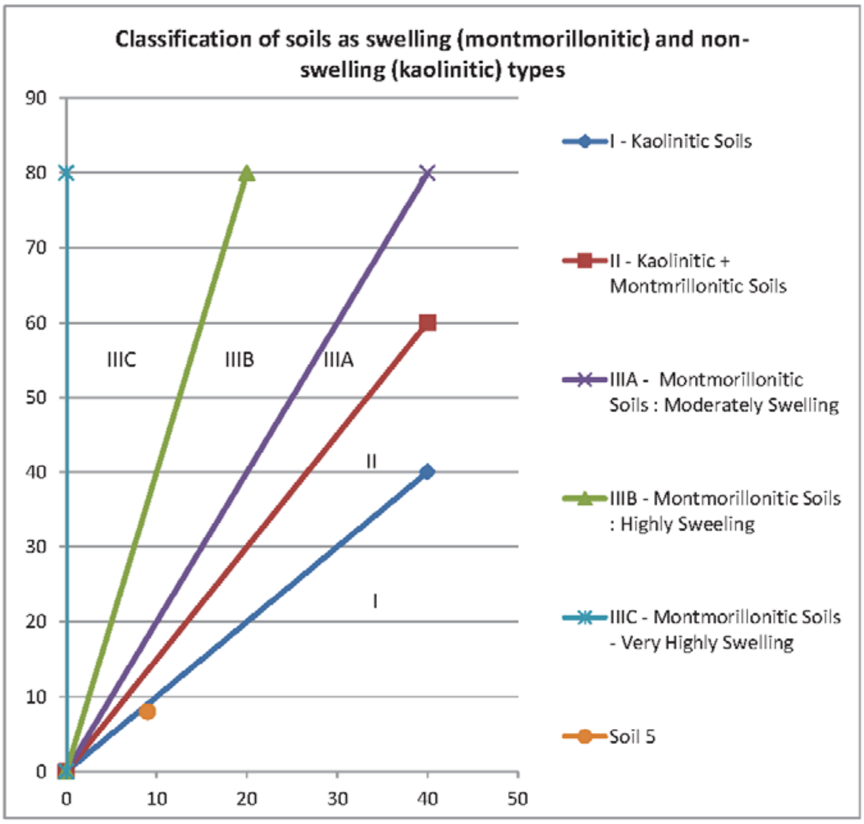


Reading (cc^3): W1 : 7 W2 : 9 K1 : 9

Classification of soils based on free swell ratio

Water (cc^3)		Kerosene (cc^3)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	7	K1	9	0,89	Negligible	Nonswelling	Kaolinitic
W2	9						
Ave	8						

The classification of the soil tested : The expansivity was Negligible, the clay was Nonswelling and the dominant was Kaolinitic



Classification
Class I : Kaolinitic Soils

The classification of the soil tested was Class I : Kaolinitic Soils

SOILS MECHANICS
RESEARCH GROUP

FREE SWELL RATIO

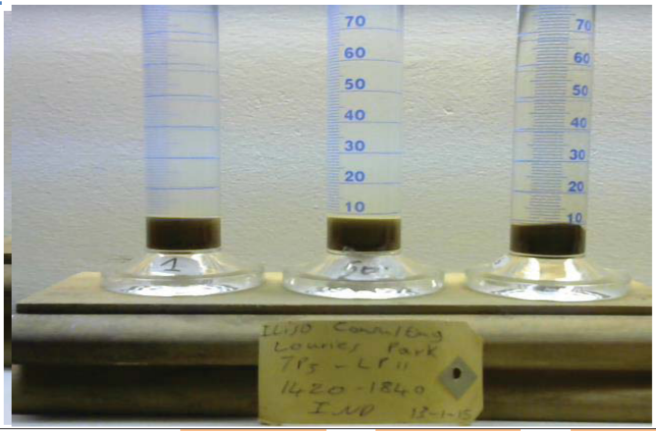
WS BADENHORST

Sample : Soil 6
Iliso Consulting, 6 kgotsong, TP7, KG 08, 0-1020

Date :
2015-05-21

Time:
24 hours

Photo:



Reading (cc³): W1 : 10 W2 : 8 K1 : 10

Classification of soils based on free swell ratio

Water (cc³)		Kerosene (cc³)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	10	K1	10	0,90	Negligible	Nonswelling	Kaolinitic
W2	8						
Ave	9						

The classification of the soil tested : The expansivity was Negligible, the clay was Nonswelling and the dominant was Kaolinitic

Classification
Class I : Kaolinitic Soils

The classification of the soil tested was Class I : Kaolinitic Soils

SOILS MECHANICS
RESEARCH GROUP

FREE SWELL RATIO

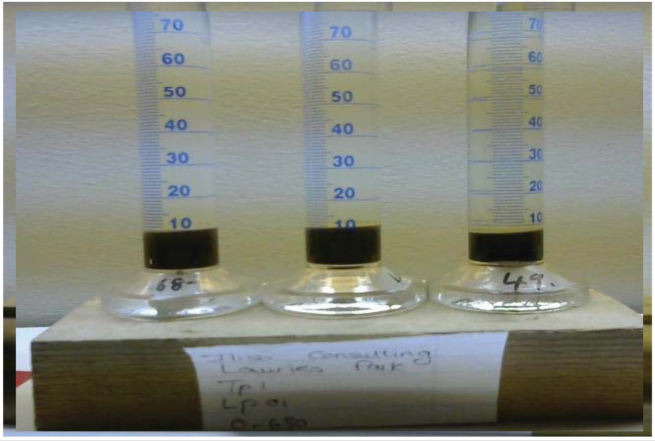
WS BADENHORST

Sample : Soil 7
Iliso Consulting, 7 Lauries Park, TP 1, LP01, 0-630

Date :
2015-05-29

Time:
24 hours

Photo:

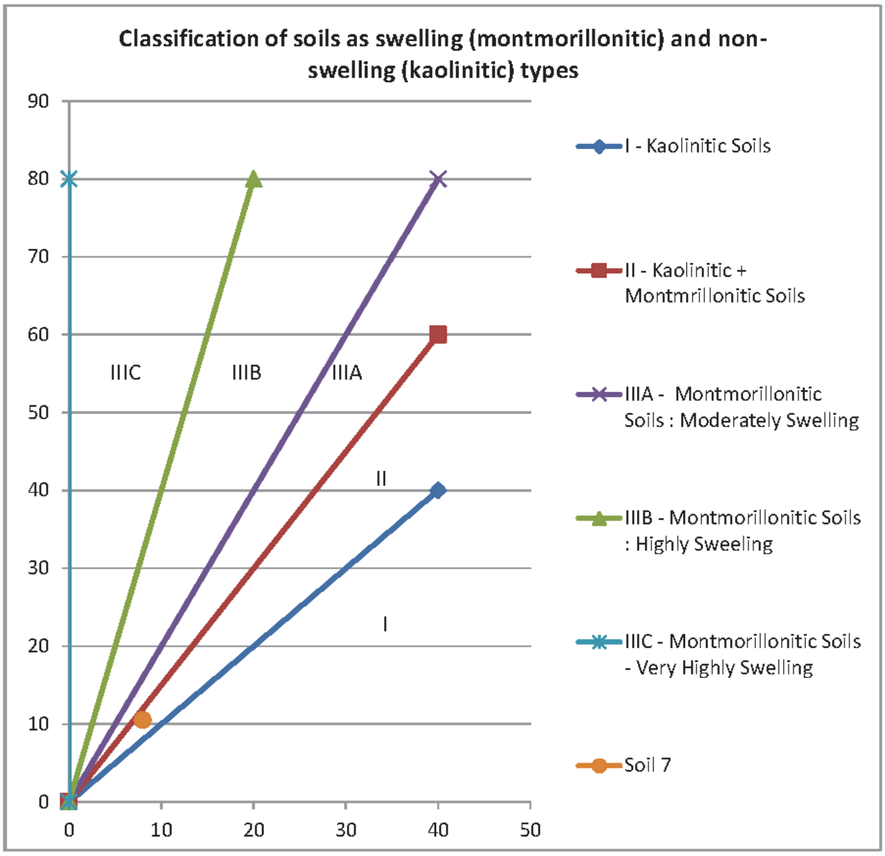


Reading (cc^3): W1 : 10 W2 : 11 K1 : 8

Classification of soils based on free swell ratio

Water (cc^3)		Kerosene (cc^3)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	10	K1	8	1,31	Low	Mixture of swelling	Kaolinitic and Montmorillonitic
W2	11						
Ave	10,5						

The classification of the soil tested : The expansivity was Low, the clay was Mixture of swelling and the dominant was Kaolinitic and Montmorillonitic



Classification
Class II : Kaolinitic Montmorillonitic Soils

The classification of the soil tested was Class II : Kaolinitic Montmorillonitic Soils

SOILS MECHANICS RESEARCH GROUP

FREE SWELL RATIO

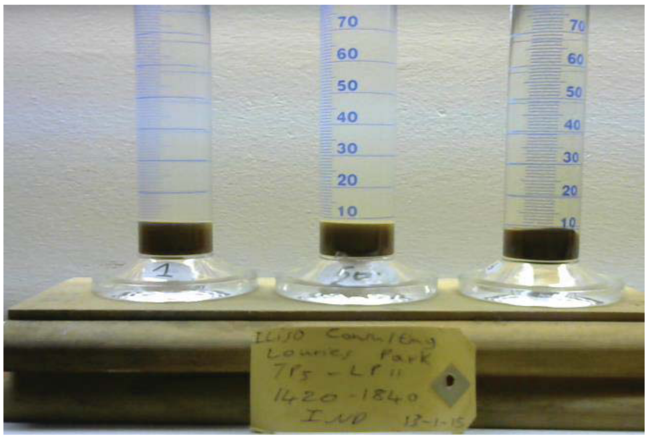
WS BADENHORST

Sample : Soil 8 Iliso Consulting, 8 Lauries park, TP3-LP11, 1420-1840

Date :
2015-05-21

Time:
24 hours

Photo:

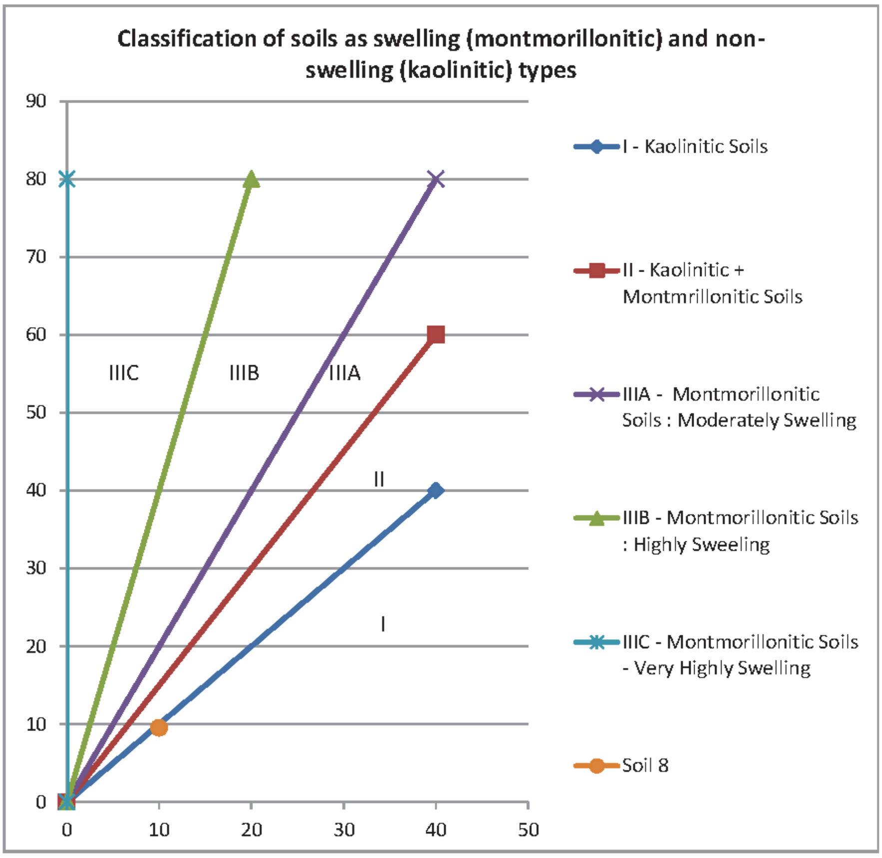


Reading (cc³):	W1 :	10	W2 :	9	K1 :	10

Classification of soils based on free swell ratio

Water (cc³)		Kerosene (cc³)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	10	K1	10	0,95	Negligible	Nonswelling	Kaolinitic
W2	9						
Ave	9,5						

The classification of the soil tested : The expansivity was Negligible, the clay was Nonswelling and the dominant was Kaolinitic



Classification
Class I : Kaolinitic Soils

The classification of the soil tested was Class I : Kaolinitic Soils

SOILS MECHANICS RESEARCH GROUP

FREE SWELL RATIO

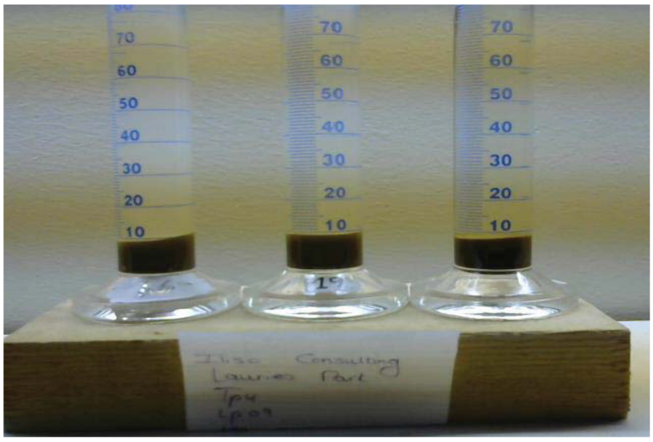
WS BADENHORST

Sample : Soil 9 Iliso Consulting, 9 Lauries park, TP4, LP09, 1870-3000

Date :
2015-05-22

Time:
24 hours

Photo:

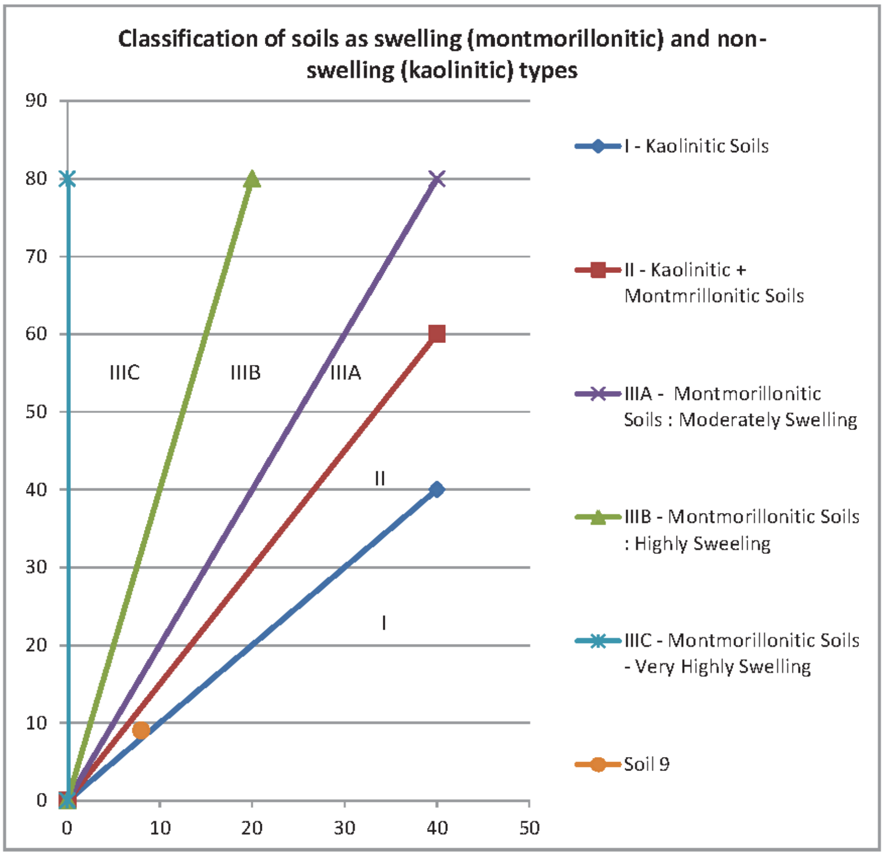


Reading (cc^3): W1 : 9 W2 : 9 K1 : 8

Classification of soils based on free swell ratio

Water (cc^3)		Kerosene (cc^3)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	9	K1	8	1,13	Low	Mixture of swelling	Kaolinitic and Montmorillonitic
W2	9						
Ave	9						

The classification of the soil tested : The expansivity was Low, the clay was Mixture of swelling and the dominant was Kaolinitic and Montmorillonitic



Classification
Class II : Kaolinitic Montmorillonitic Soils

The classification of the soil tested was Class II : Kaolinitic Montmorillonitic Soils

SOILS MECHANICS RESEARCH GROUP

FREE SWELL RATIO

WS BADENHORST

Sample : Soil 10 Iliso Consulting, 10 Lauries park, TP5, LP 11, 1420-1840

Date : 2015-05-28

Time: 24 hours

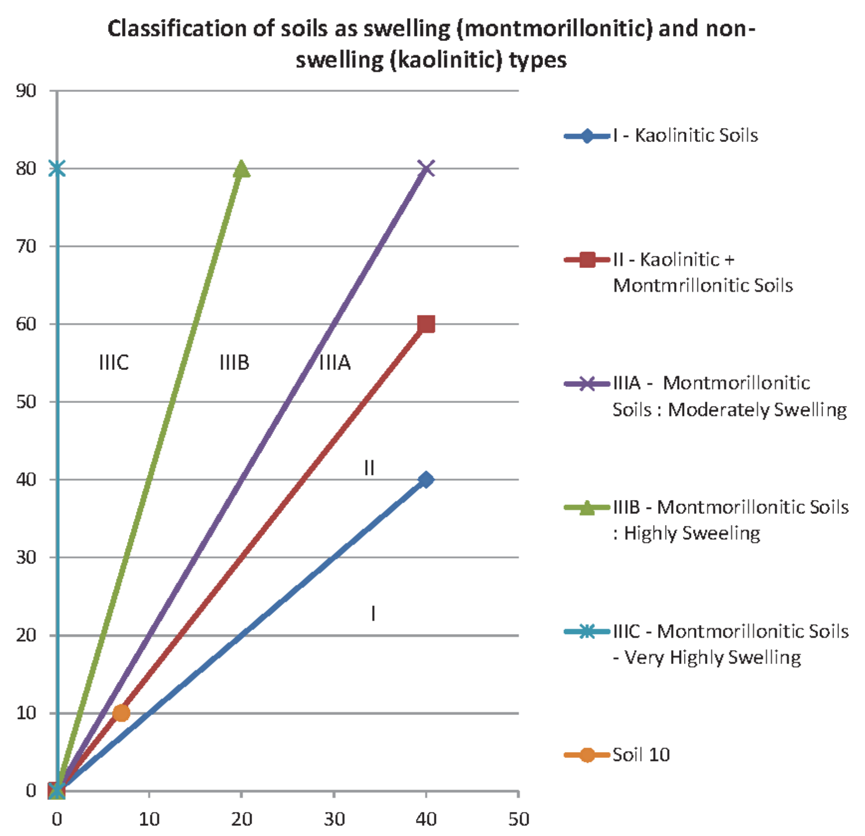
Photo:

Reading (cc^3): W1 : 9 W2 : 11 K1 : 7

Classification of soils based on free swell ratio

Water (cc^3)		Kerosene (cc^3)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	9	K1	7	1,43	Low	Mixture of swelling	Kaolinitic and Montmorillonitic
W2	11						
Ave	10						

The classification of the soil tested : The expansivity was Low, the clay was Mixture of swelling and the dominant was Kaolinitic and Montmorillonitic



Classification
Class II : Kaolinitic Montmorillonitic Soils

The classification of the soil tested was Class II : Kaolinitic Montmorillonitic Soils

SOILS MECHANICS
RESEARCH GROUP

Central University of Technology, Free State

FREE SWELL RATIO

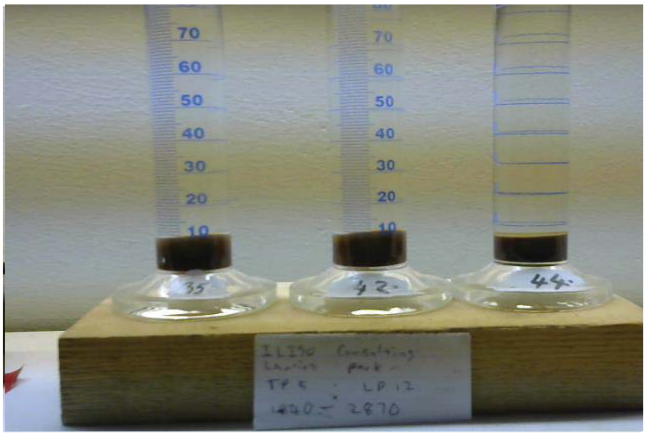
WS BADENHORST

Sample : Soil 11
Iliso Consulting, 11
Lauries park, TP5,
LP 12, 1840-2870

Date :
2015-05-15

Time:
24 hours

Photo:

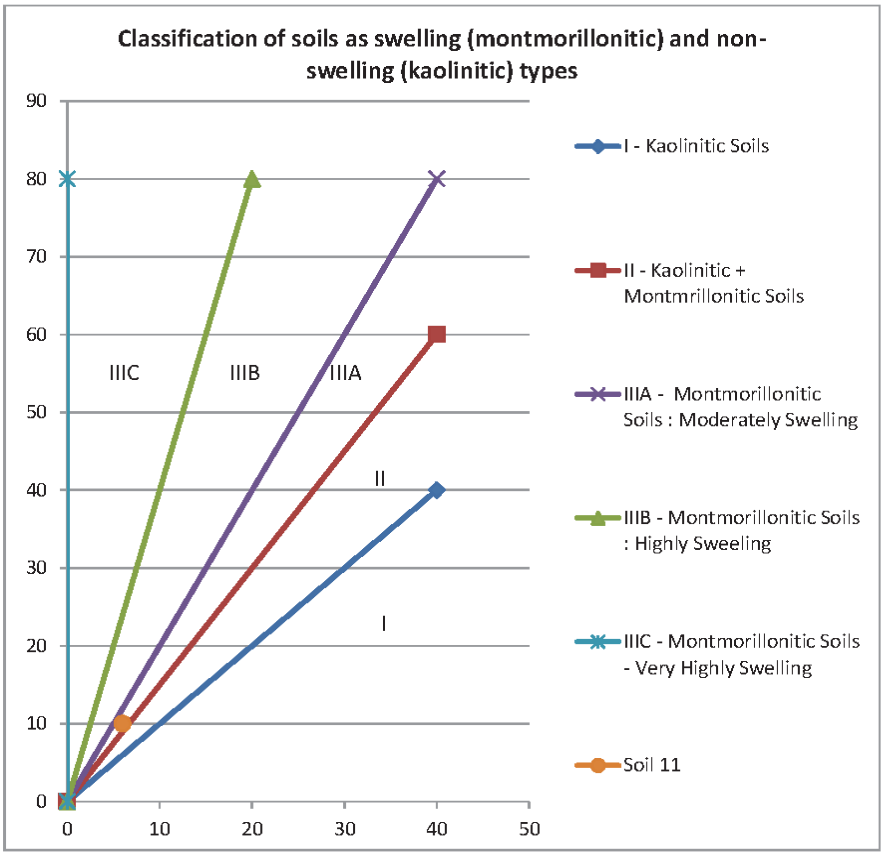


Reading (cc^3): W1 : 10 W2 : 10 K1 : 6

Classification of soils based on free swell ratio

Water (cc^3)		Kerosene (cc^3)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	10	K1	6	1,67	Moderate	Swelling	Montmorillonitic
W2	10						
Ave	10						

The classification of the soil tested : The expansivity was Moderate, the clay was Swelling and the dominant was Montmorillonitic



Classification
Class IIIA : Montmorillonitic Soils - Moderately

The classification of the soil tested was Class IIIA : Montmorillonitic Soils - Moderately Swelling

SOILS MECHANICS
RESEARCH GROUP
Central University of Technology, Free State

FREE SWELL RATIO

WS BADENHORST

Sample : Soil 12
Iliso Consulting, 12
Lauries park, TP 6,
LP13, 0-1890

Date :
2015-07-10

Time:
24 hours

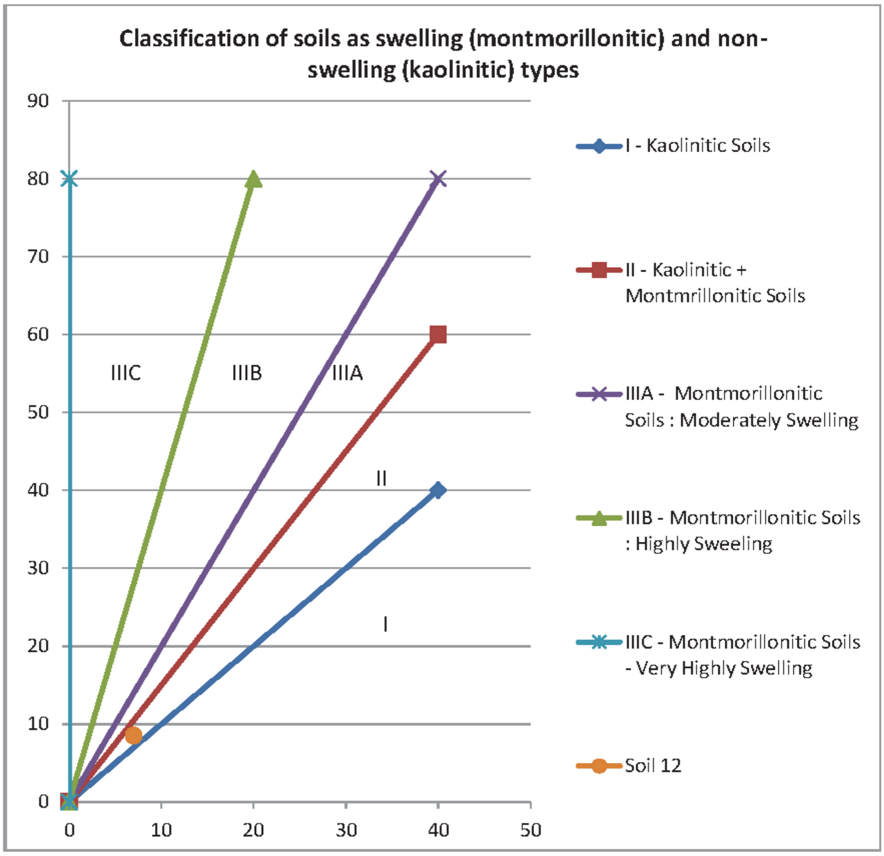
Photo:

Reading (cc^3): W1 : 9 W2 : 8 K1 : 7

Classification of soils based on free swell ratio

Water (cc^3)		Kerosene (cc^3)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	9	K1	7	1,21	Low	Mixture of swelling	Kaolinitic and Montmorillonitic
W2	8						
Ave	8,5						

The classification of the soil tested : The expansivity was Low, the clay was Mixture of swelling and the dominant was Kaolinitic and Montmorillonitic


Classification of soils as swelling (montmorillonitic) and non-swelling (kaolinitic) types

Classification
Class II : Kaolinitic Montmorillonitic Soils

The classification of the soil tested was Class II : Kaolinitic Montmorillonitic Soils

SOILS MECHANICS RESEARCH GROUP

Central University of Technology, Free State

FREE SWELL RATIO

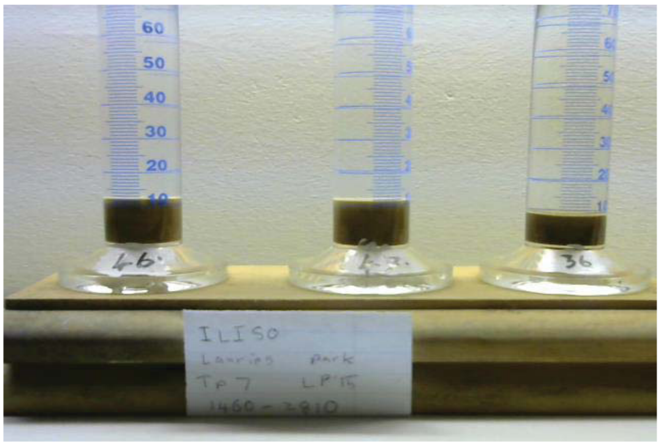
WS BADENHORST

Sample : Soil 13
Iliso Consulting, 13
Lauries Park, TP7,
LP15, 1460-2810

Date :
2015-05-15

Time:
24 hours

Photo:

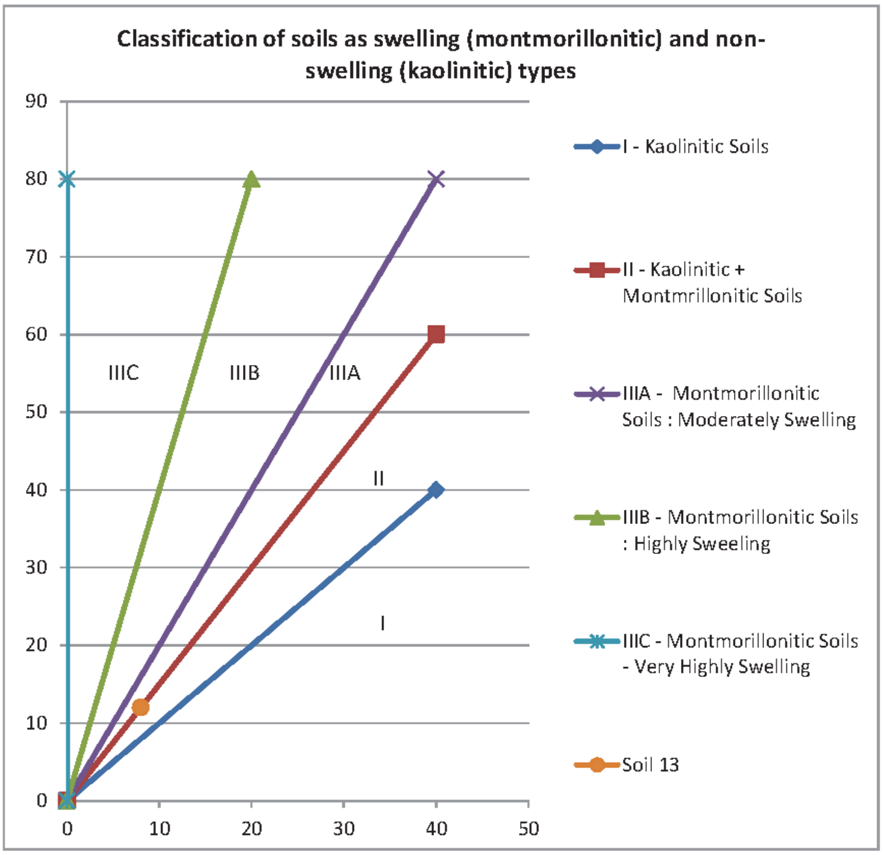


Reading (cc^3): W1 : 12 W2 : 12 K1 : 8

Classification of soils based on free swell ratio

Water (cc^3)		Kerosene (cc^3)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	12	K1	8	1,50	Moderate	Swelling	Montmorillonitic
W2	12						
Ave	12						

The classification of the soil tested : The expansivity was Moderate, the clay was Swelling and the dominant was Montmorillonitic



Classification
Class IIIA : Montmorillonitic Soils - Moderately

The classification of the soil tested was Class IIIA : Montmorillonitic Soils - Moderately Swelling

SOILS MECHANICS RESEARCH GROUP

FREE SWELL RATIO

WS BADENHORST

Sample : Soil 14 Iliso Consulting, 14 MK Square, TP 1, MK 01, 0-710

Date : 2015-05-25

Time: 24 hours

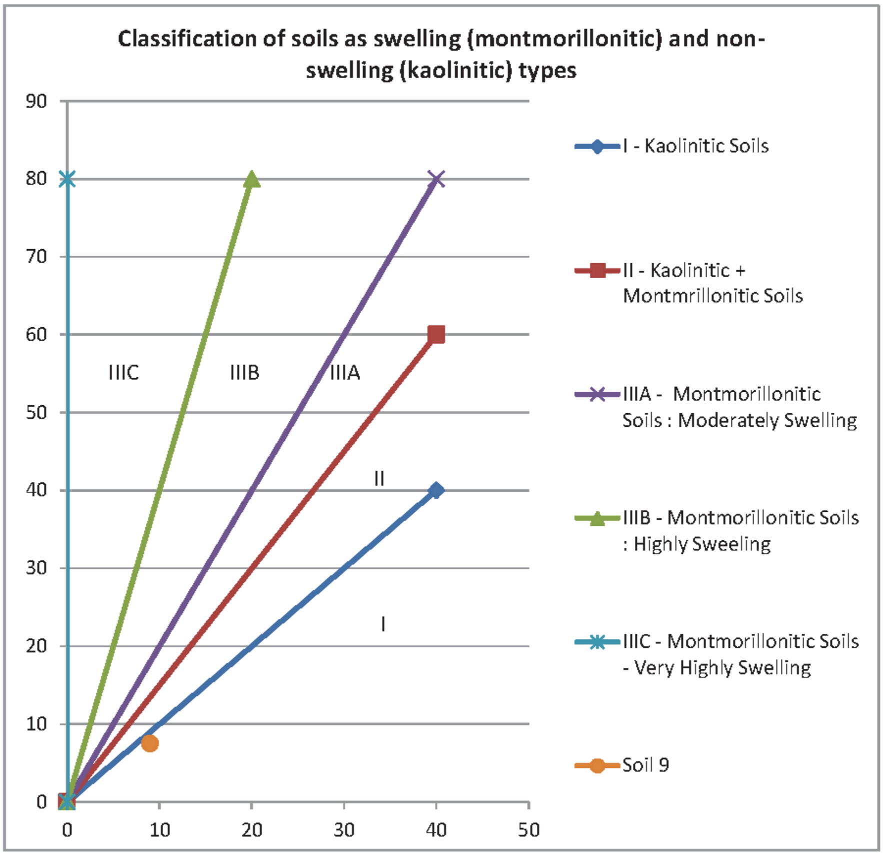
Photo:

Reading (cc^3): W1 : 7 W2 : 8 K1 : 9

Classification of soils based on free swell ratio

Water (cc^3)		Kerosene (cc^3)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	7	K1	9	0,83	Negligible	Nonswelling	Kaolinitic
W2	8						
Ave	7,5						

The classification of the soil tested : The expansivity was Negligible, the clay was Nonswelling and the dominant was Kaolinitic



Classification
Class I : Kaolinitic Soils

The classification of the soil tested was Class I : Kaolinitic Soils

SOILS MECHANICS
RESEARCH GROUP

FREE SWELL RATIO

WS BADENHORST

Sample : Soil 15
Iliso Consulting, 15
MK Square, TP1,
MK 02, 710-1220

Date :
2015-07-17

Time:
24 hours

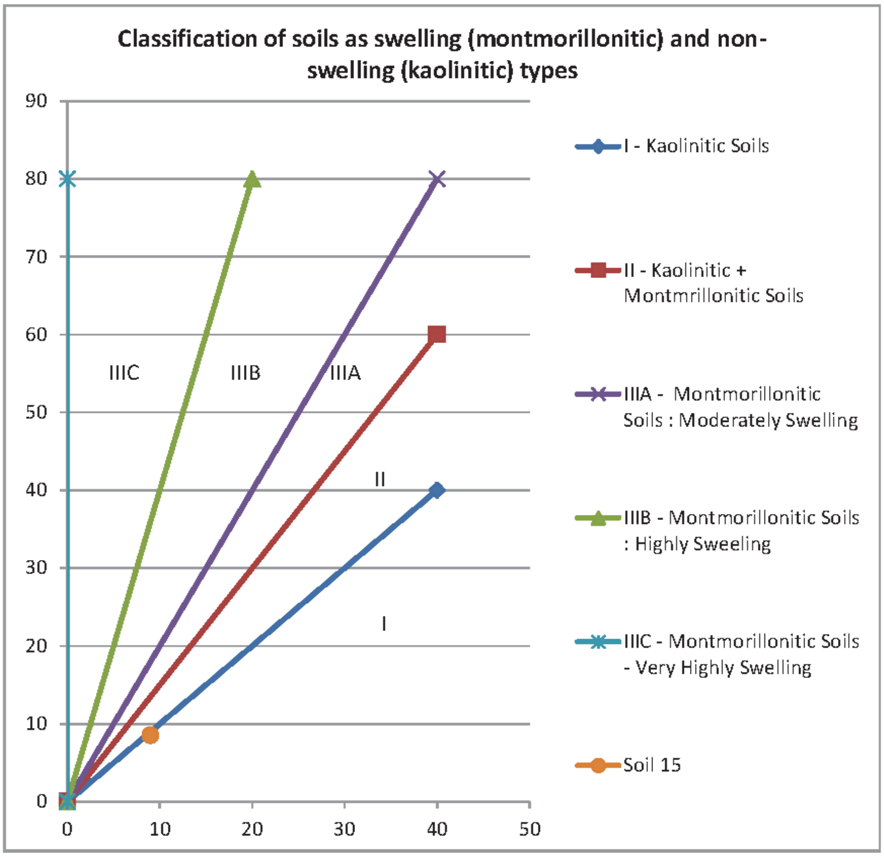
Photo:

Reading (cc³): W1 : 7 W2 : 10 K1 : 9

Classification of soils based on free swell ratio

Water (cc³)		Kerosene (cc³)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	7	K1	9	0,94	Negligible	Nonswelling	Kaolinitic
W2	10						
Ave	8,5						

The classification of the soil tested : The expansivity was Negligible, the clay was Nonswelling and the dominant was Kaolinitic


Classification of soils as swelling (montmorillonitic) and non-swelling (kaolinitic) types

Classification
Class I : Kaolinitic Soils

The classification of the soil tested was Class I : Kaolinitic Soils

SOILS MECHANICS RESEARCH GROUP

Central University of Technology, Free State

FREE SWELL RATIO

WS BADENHORST

Sample : Soil 16 Iliso Consulting, 16 MK Square, TP2, MK04, 940-2810

Date :
2015-07-10

Time:
24 hours

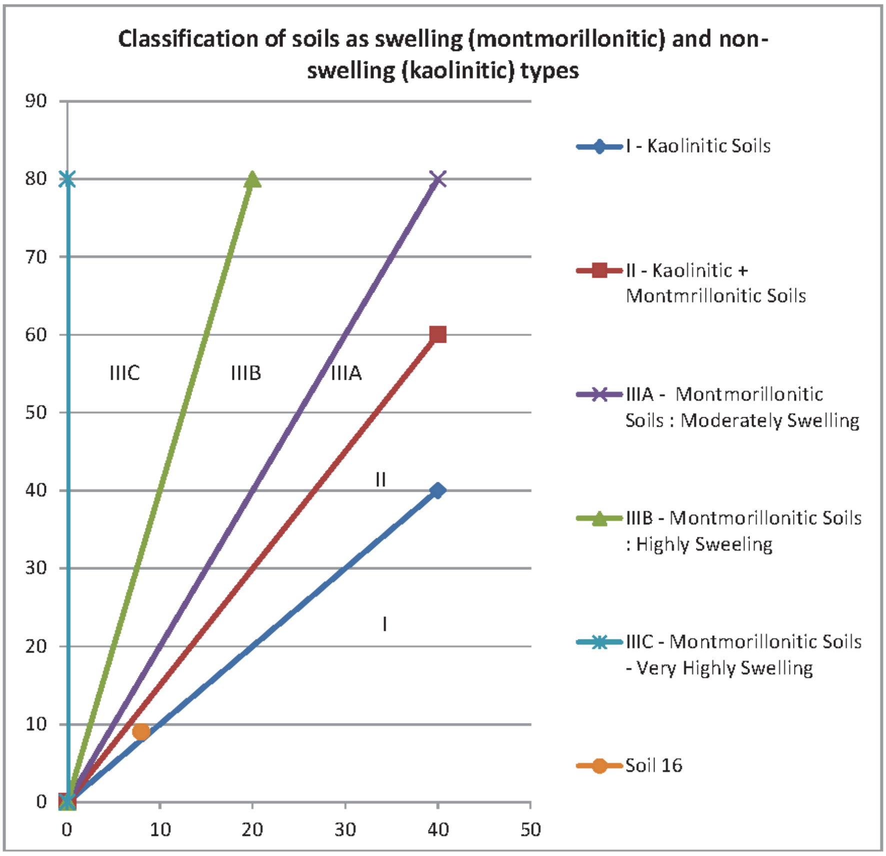
Photo:

Reading (cc^3): W1 : 10 W2 : 8 K1 : 8

Classification of soils based on free swell ratio

Water (cc^3)		Kerosene (cc^3)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	10	K1	8	1,13	Low	Mixture of swelling	Kaolinitic and Montmorillonitic
W2	8						
Ave	9						

The classification of the soil tested : The expansivity was Low, the clay was Mixture of swelling and the dominant was Kaolinitic and Montmorillonitic


Classification of soils as swelling (montmorillonitic) and non-swelling (kaolinitic) types

Classification
Class II : Kaolinitic Montmorillonitic Soils

The classification of the soil tested was Class II : Kaolinitic Montmorillonitic Soils

SOILS MECHANICS
RESEARCH GROUP

FREE SWELL RATIO

WS BADENHORST

Sample : Soil 17
Iliso Consulting, 17
MK Square, TP 3,
MK 05, 0-800

Date :
2015-06-26

Time:
24 hours

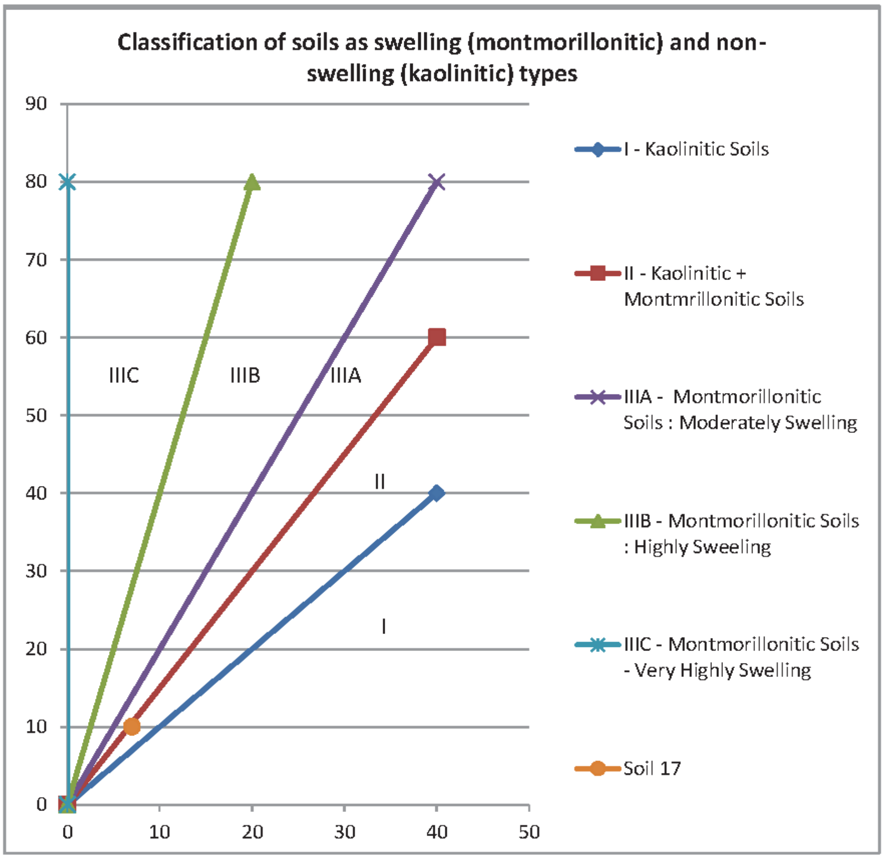
Photo:

Reading (cc^3): W1 : 10 W2 : 10 K1 : 7

Classification of soils based on free swell ratio

Water (cc^3)		Kerosene (cc^3)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	10	K1	7	1,43	Low	Mixture of swelling	Kaolinitic and Montmorillonitic
W2	10						
Ave	10						

The classification of the soil tested : The expansivity was Low, the clay was Mixture of swelling and the dominant was Kaolinitic and Montmorillonitic



Classification
Class II : Kaolinitic Montmorillonitic Soils

The classification of the soil tested was Class II : Kaolinitic Montmorillonitic Soils

SOILS MECHANICS RESEARCH GROUP

FREE SWELL RATIO

WS BADENHORST

Sample : Soil 18
Iliso Consulting, 18 MK Square, TP4, MK07, 0-690

Date :
2015-07-13

Time:
24 hours

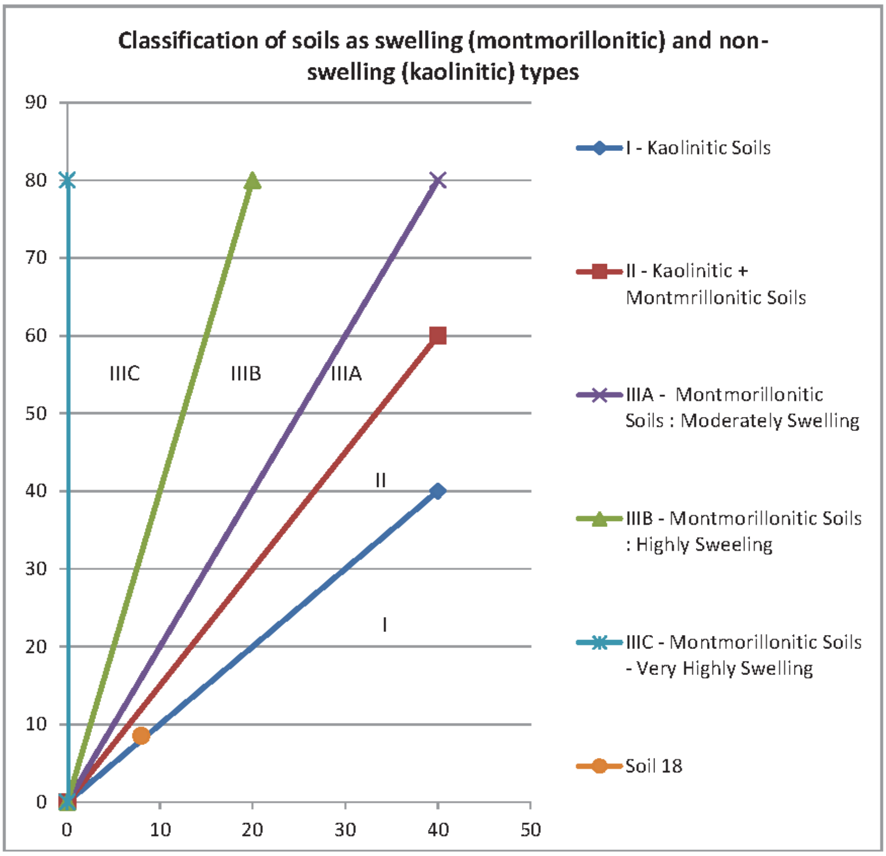
Photo:

Reading (cc^3): W1 : 9 W2 : 8 K1 : 8

Classification of soils based on free swell ratio

Water (cc^3)		Kerosene (cc^3)		Free swell ratio	Soil Expansivity	Clay type	Dominant Clay
W1	9	K1	8	1,06	Low	Mixture of swelling	Kaolinitic and Montmorillonitic
W2	8						
Ave	8,5						

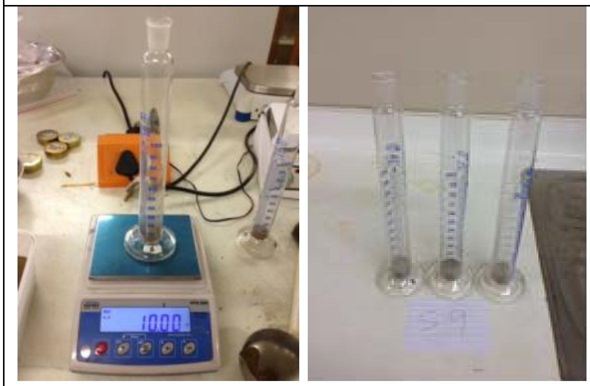
The classification of the soil tested : The expansivity was Low, the clay was Mixture of swelling and the dominant was Kaolinitic and Montmorillonitic



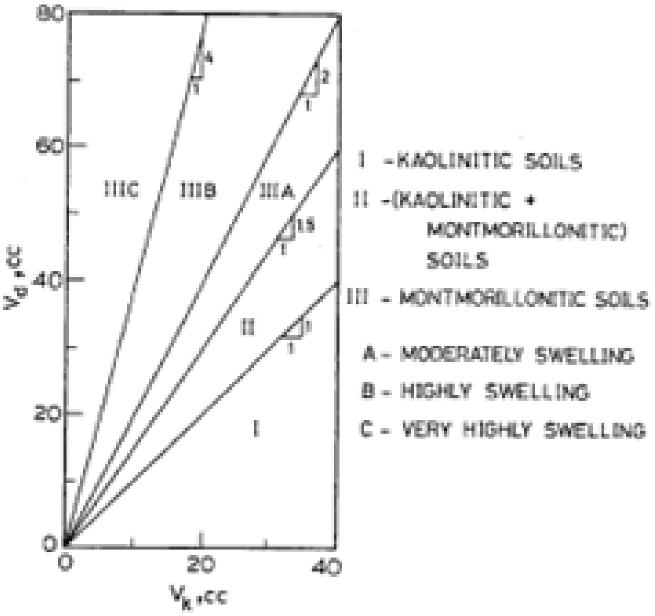
Classification
Class II : Kaolinitic Montmorillonitic Soils

The classification of the soil tested was Class II : Kaolinitic Montmorillonitic Soils

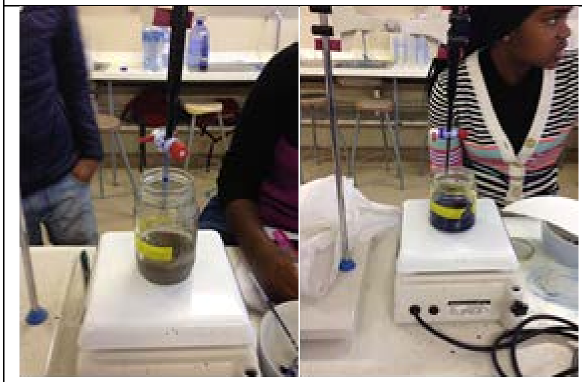
APPENDIX C: TEST METHODOLOGY FOR FREE SWELL RATIO

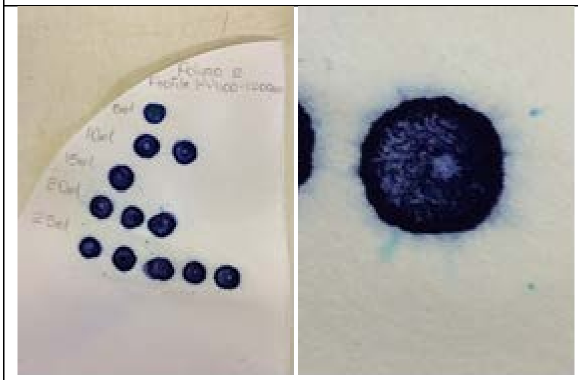
FREE SWELL TEST RATIO – TEST METHODOLOGY	
	Dry soil passing 0.425mm aperture size in an oven at 105 - 110°C until constant mass is achieved Soil not passing the 0.425mm aperture may be crushed with mortar and pestle, but do not break coarse particles down. It is recommended to apply limited pressure; the idea is to break down clay lumps that may form.
	Weigh out 10grams of dry material and add it to an empty, dry measuring cylinder that can accurately measure to 1mm or 1ml. Do this for three cylinders; two will be used with water (distilled) and one with kerosene (toluene or paraffin).

	Add approximately 40ml of water (distilled or de-ionised) to two of the three cylinders and thoroughly mix the soils through in order to ensure no soil is caught on the sides of the cylinders. Do the same with kerosene (paraffin or toluene) for the third cylinder. Top up the corresponding liquids until a final level of 100ml are achieved. The voids should be filled with water or kerosene at this time, but some air may still be trapped inside. This may prove problematic to get the final value to 100ml. Put the three cylinders in front of the test screen and take readings after 1, 2 and 24hours. Note that the meniscus formed during mixing may complicate this somewhat.

	Record the level of the kerosene and the level of the water in each cylinder at the corresponding times. Calculations: Free Swell Ratio = ((Vd – Vk) / Vk) x 100 Vd = Volume in Distilled Water Vk = Volume in Kerosene Report: Report on the chart indicated on the left reflecting the sample details and numbers.

APPENDIX D: TEST METHODOLOGY FOR METHYLENE BLUE VALUE TEST

	Dry soil passing 0.425mm aperture size in an oven at 105 - 110°C until constant mass is achieved Soil not passing the 0.425mm aperture may be crushed with mortar and pestle, but do not break coarse particles down. It is recommended to apply limited pressure; the idea is to break down clay lumps that may form.
	A methylene blue suspension is made by adding 10grams of methylene blue to 1liter of distilled water Add 5-7grams of material to a mixing jar (glass) and add 50ml of distilled water Mix the soil and water at 700 RPM for 5minutes

<table>
<tr><td></td><td>Add 5ml of methylene blue suspension into the glass container and stir for a further 2 minutes at 700RPM</td></tr>
<tr><td>
</td><td>Test the suspension by dropping a single droplet onto filter-paper. If it forms a blue halo, repeat the test after 1 minute. If the test remains positive after 5 tests (5minutes) the test is completed

If a blue halo did not appear, add 5ml of methylene blue solution and repeat as before. If it did appear initially but did not remain positive, add 2ml of methylene blue and test as before.</td></tr>
<tr><td colspan="2">Report the total ml of methylene blue added per grams of materials.

MBV = Vcc (ml) / f’(g)

Vcc = volume of methylene blue solution added to the soil suspension

f’ = dry weight of the sample used (g)</td></tr>
</table>

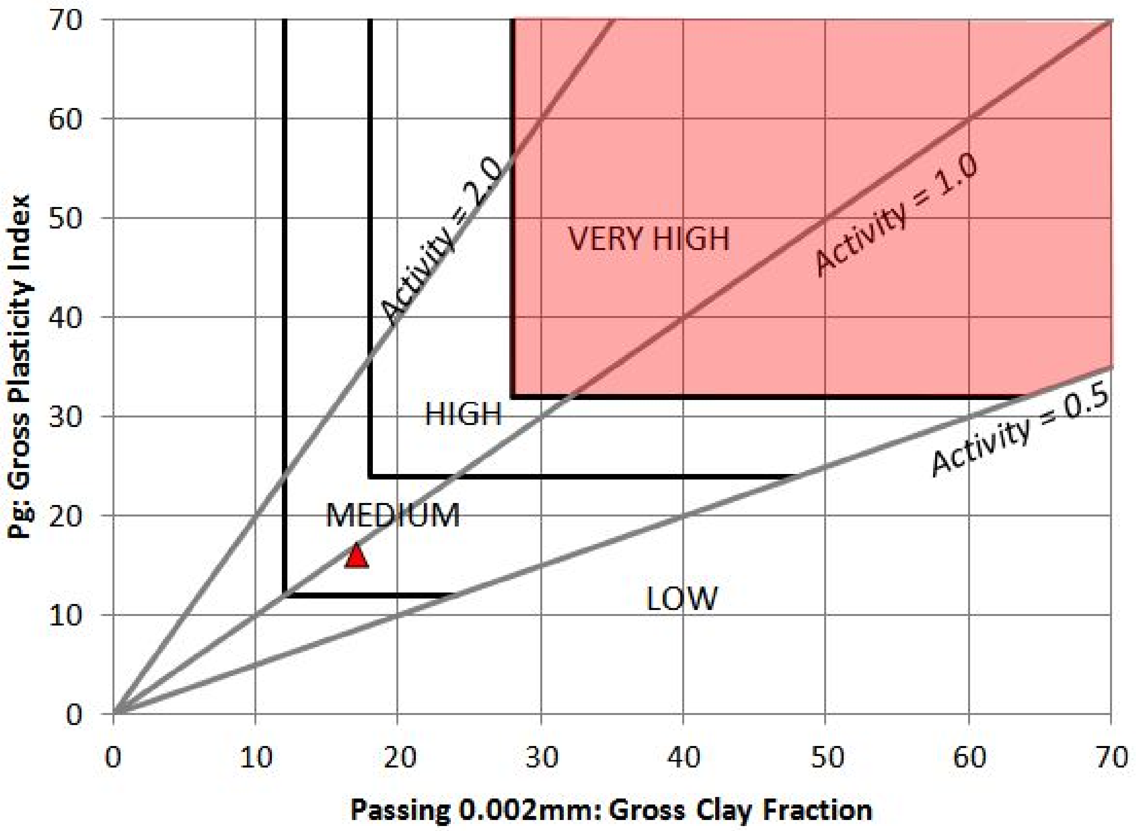
APPENDIX E: DETERMINING TOTAL ESTIMATED HEAVE USING VAN DER MERWE’S METHOD

Assuming laboratory results indicating the following:

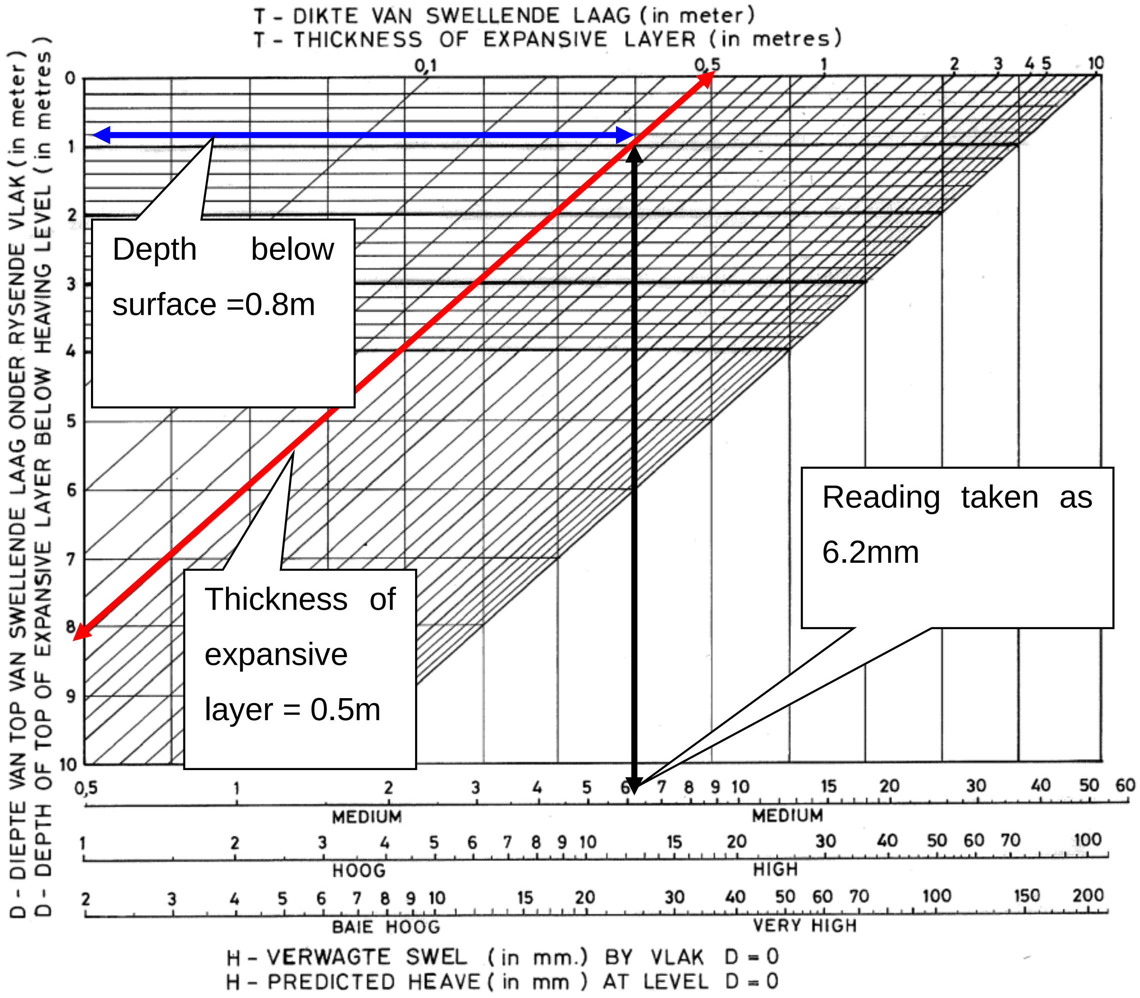
Sample Name / Description	Sample 1 – Greyish brown clayey sand
Percentage passing 0.425mm	89%
Percentage smaller than 0.002mm	17%
Plasticity Index of Whole Sample (Weighted)	16%
Thickness of the layer (in metres)	0.5m
Depth of top of expansive layer below heaving level (surface) (in metres)	0.8m
Potential Expansiveness	Medium (Based on the graph below)
Total Estimated Heave (from graphs) (mm)	6.2mm
Total Estimated Heave (from formula) (m)	6.99mm

The formula and graph was adapted by van der Merwe (1976) to reflect SI units, van der Merwe’s 1964 work was reflected in inches and feet.

$$\begin{matrix}\text{SWEL}\\\text{HEAVE}\end{matrix}\quad H=-\left[F.e^{-0,377.D}\left(e^{-0,377.T}-1\right)\right]$$


70
60
50
40
30
20
10
0
0
10
20
30
40
50
60
70
Pg: Gross Plasticity Index
Passing 0.002mm: Gross Clay Fraction
Activity = 2.0
Activity = 1.0
Activity = 0.5
VERY HIGH
HIGH
MEDIUM
LOW

VERWAGTE SWEL VANAF POTENSIËLE UITSETTING
HEAVE PREDICTION FROM POTENTIAL EXPANSIVENESS



Depth below surface =0.8m

Thickness of expansive layer = 0.5m

Reading taken as 6.2mm

UIT :
FROM: D H VAN DER MERWE: "THE PREDICTION OF HEAVE FROM THE PLASTICITY INDEX AND PERCENTAGE CLAY FRACTION OF SOILS."
TRANSACTIONS S.A. INSTITUTE OF CIVIL ENGINEERS, June 1964

APPENDIX F: MOTIVATION FOR THIS STUDY

In 2014, the author undertook a geotechnical investigation at a school in Luckhoff located in the Free State province of South Africa. By this time, geotechnical investigations of this nature became a routine and nothing interesting was expected. During the investigation the author identified expansive cracking on adjacent classrooms and recorded it.

After all the laboratory analysis were completed, the author compiled a geotechnical report on the findings and was surprised that none of the laboratory tests indicated heaving clays as a geotechnical constraint. If not for the structural failure evident on site, the author would not have regarded the materials on site to be expansive.

Excerpts from the geotechnical investigation conducted at Luckhoff Combined School.

F.1 Site Location

The site is located within the Municipal boundary of Luckhoff, between the CBD of Luckhoff and Relebohile suburb, approximately 1.0km north-east of the CBD. Luckhoff is located in the Free State province of South Africa, approximately 180km west of Bloemfontein and approximately 120km south of Kimberley. Figure F.1 indicates the location of Luckhof relative to Bloemfontein.

Figure F.1: Site location

F.2 Site Climate

The climate around Luckhoff is essentially continental one with warm, wet summers and relatively cold winters. The average summer maximum is 32.6°C and the average winter minimum is 2.7°C. The average annual rainfall varies between 250mm and 500mm. Luckhoff is a dry climatic region.

F.3 Site Geology

The site is situated within the Tierberg Group (Pt) within the ECCA group which forms part of the KAROO SUPERGROUP. The Tierberg Group (Pt) pertains to underlain shale.

Light brown, reddish brown, grey and greyish brown sandy silt and silty sands were typically encountered on site with weathered shale encountered in most of the test pits with occasional pedogenic calcrete.

F.4 Topography and Drainage

With the slope being relatively flat, drainage is a possible concern. It is recommended that a contour map be utilised to determine the best possible design in terms of drainage. It is to be ensured that the drainage provided on site should be sufficient in terms of its general requirements and design life.

F.5 Method of Investigation

Eight (8) test pits were excavated to cover the proposed development area. The test pits were excavated with a TLB (8ton) and the soil profiles were described according to the standard method proposed by Jennings, Brink and Williams (1973).

The test pit positions (Figure F.2) are indicated by GPS coordinates on the Profiles.

Figure F.2: Test pit locations throughout the study area

Disturbed samples of the most prominent soil horizons were taken and submitted for foundation indicator, and CBR tests. Undisturbed samples were taken at prominent soil horizons.

F.6 Summary of Test Results and Heave Potential

Disturbed samples were subjected to analysis according to the test methods stipulated in TMH1:1986. Test methods A1(a) to A8 were conducted to determine the various parameters summarised in tables F.1 and F.2.

Table F.1: A summary of the particle size distribution of the samples analysed

Test Pit No.	Layer Thickness (mm)	Gravel >4.750mm	Sand >0.075-4.750mm	Silt >0.002-0.075mm	Clay <0.002mm
Test Pit 1	0 – 1200	0	60	28	12
	1200 – 2800	1	68	19	12
Test Pit 2	0 – 250	3	71	12	14
	250 – 800	0	72	14	14
	800 – 1800	2	69	16	13
	1800 – 2800	8	84	3	5
Test Pit 3	0 – 800	0	75	11	14
	800 – 2000	2	79	13	6
Test Pit 4	0 – 600	1	74	12	13
	600 – 1400	3	70	7	20
	1400 – 2500	3	86	6	5
Test Pit 5	0 – 1100	20	40	31	9
	1100 – 2500	2	82	11	5
Test Pit 6	0 – 200	15	71	8	6
	200 – 2200	16	77	4	3
Test Pit 8	800 – 2800	3	83	11	3

Table F.2: Plasticity index, potential heave classification and total estimated heave using van der Merwe’s method (1964)

Test Pit	Depth (mm)	Plasticity Index	0.02mm Material Fraction	Potential Expansiveness	Total Estimated Heave (van der Merwe, 1964)
Test Pit 1	0 – 1200	0	12	Low	0.0mm
	1200 – 2800	15	12	Medium	13.9mm
Test Pit 2	0 – 250	4	14	Low	0.0mm
	250 – 800	3	14	Low	0.0mm

Test Pit	**Depth (mm)**	**Plasticity Index**	**0.02mm Material Fraction**	**Potential Expansiveness**	**Total Estimated Heave (van der Merwe, 1964)**
	800 – 1800	5	13	Low	0.0mm
	1800 – 2800	12	5	Low	0.0mm
Test Pit 3	0 – 800	4	14	Low	0.0mm
	800 – 2000	13	6	Low	0.0mm
Test Pit 4	0 – 600	11	13	Low	0.0mm
	600 – 1400	9	20	Low	0.0mm
	1400 – 2500	14	5	Low	0.0mm
Test Pit 5	0 – 1100	9	9	Low	0.0mm
	1100 – 2500	11	5	Low	0.0mm
Test Pit 6	0 – 200	4	6	Low	0.0mm
	200 – 2200	9	3	Low	0.0mm
Test Pit 8	800 – 2800	11	3	Low	0.0mm

It should be noted that only one sample was described as expansive, namely Test Pit 1 (1200 – 2800). Analysing the results using van der Merwe’s method indicates that it should also be classified as “Low”, but the author adjudged the sample to be near one of the boundaries and conservatively classified it as “medium”.

F.7 Findings during Site Investigation

During the investigation, the author noticed expansive cracks on the adjacent structures (Figures F.3 to F.5) and noted a concern in terms of expansive soils. The investigation was immediately more focussed on expansive materials, but despite specific focus on expansive soils the majority of the materials encountered on site were described as either “silty” or “sandy”. The site conditions at the time of the investigation was dry and water was used to aid in identifying clays, but none of the materials encountered on site were described as clayey.

Figure F.3: Expansive cracking visible on the side of one of the current classrooms (photo by author)

Figure F.4: Diagonal cracking below the windowsill, note the lack of a concrete apron slab (photo by author)

Figure F.5: Cracking near the roof of the class room (photo by author)

F.8 Photos of Typical Soils Encountered during the Investigation

The test pit profiles were summarised in tables F.3 to F.7 with photos by the author of the prominent materials encountered on site labelled as Figures F.6 to F.10.

Table F.3: Test pit 1 profile descriptions

Layer (mm)	Material description
0 – 1200	Slightly moist light greyish brown medium dense silty sand (transported) with calcrete particles
1200-2800	Slightly moist light brown medium dense silty sand (residual).

Figure F.6: Test pit 1 materials and test pit

Table F.4: Test pit 2 profile descriptions

Layer (mm)	Material description
0 – 250	Slightly moist light olive brown medium dense silty sand (mixed origin) - imported material.
250 – 800	Slightly moist light reddish brown medium dense silty gravel .
800 – 1800	Slightly moist greyish brown dense silty sand (residual) with calcrete particles.
1800 – 2800	Slightly moist grey very dense sand (residual) with fractured shale.

Figure F.7: Test pit 2 materials and test pit

Table F.5: Test pit 3 profile descriptions

Layer (mm)	Material description
0 – 1800	Slightly moist orange medium dense silty sand (transported).
1800 – 2000	Slightly moist grey very dense silty sand (residual) with fractured shale.

Figure F.8: Test pit 3 materials and test pit

Table F.6: Test pit 4 profile descriptions

Layer (mm)	Material description
0 – 600	Slightly moist orange medium dense silty sand (transported).
600 – 1400	Slightly moist orange mixed with grey dense silty sand with calcrete particles.
1400 – 2500	Slightly moist grey very dense sand (residual).

Figure F.9: Test pit 4 materials and test pit

Table F.7: Test pit 5 profile descriptions

Layer (mm)	Material description
0 – 1100	Slightly moist orange medium dense silty sand (transported).
1100 – 2500	Slightly moist orange mixed with grey dense silty sand with calcrete particles.

Figure F.10: Test pit 5 materials and test pit

F.9 Summary of Test Results

Table F.8: Summary of disturbed test results

Test Pit	Layer mm	Clay Fraction %	Grading Modulus	Liquid Limit %	Plasticity Index %	Linear Shrinkage %	CBR Swell %	Optimum Moisture Content %	Maximum Dry Density kg/m³
1	0 – 1200	12	0.73	0	Non-Plastic	0.0	1.0	13.3	1892
1	1200 – 2800	12	0.88	40	15	6.5	-	-	-
2	0 – 250	14	0.99	21	4	1.3	1.5	13.5	1900
2	250 – 800	14	0.74	21	3	1.2	-	-	-
2	800 – 1800	13	0.84	25	5	2.4	-	-	-
2	1800 – 2800	5	2.20	32	12	6.4	0.2	11.5	2034
3	0 – 800	14	0.81	21	4	1.7	0.2	12.7	1886
3	800 – 2000	6	1.58	28	13	6.7	1.4	12.8	2000
4	0 – 600	13	0.83	25	11	6.3	-	-	-

Test Pit	**Layer mm**	**Clay Fraction %**	**Grading Modulus**	**Liquid Limit %**	**Plasticity Index %**	**Linear Shrinkage %**	**CBR Swell %**	**Optimum Moisture Content %**	**Maximum Dry Density kg/m³**
4	600 – 1400	20	0.90	27	9	3.5	-	-	-
4	1400 – 2500	5	1.87	37	14	7.0	1.5	11.5	1980
5	0 – 1100	9	1.23	26	9	5.1	0.6	16.5	1804
5	1100 – 2500	5	1.83	30	11	6.0	-	-	-
6	0 – 200	6	1.55	23	4	1.7	-	-	-
6	200 – 2200	3	2.30	31	9	4.5	1.2	12.6	2006
8	800 – 2800	3	1.88	25	11	5.4	-	-	-

The undisturbed samples were typically subjected to consolidation tests to determine whether the materials on site are considered to be collapsible, compressible or expansive. The consolidation test indicated below is of transported (Aeolian) sands found on site to confirm collapse potential. None of the consolidation tests indicated heaving clays as a constraint.

Tables F.9 and F.10 indicate the effective stress, strain and void ratio of the samples while Figures F.11 and F.12 indicate the visual representation of the effective stress compared to strain and void ratio.

Table F.9: Consolidation test on the undisturbed sample taken at test pit 1, 0 – 600mm – Strain (%) vs. Effective Stress (kPa)

Effect.Stress (kPa)	10	51	100	198	198	398	100	10
Strain (%)	0.12	0.40	0.69	1.22	2.41	3.59	3.06	1.58
Mv (1/MPa)		0.0683	0.0594	0.0544		0.0591	0.0177	0.1648
Void Ratio	0.3085	0.3048	0.301	0.294	0.2785	0.263	0.2699	0.2893

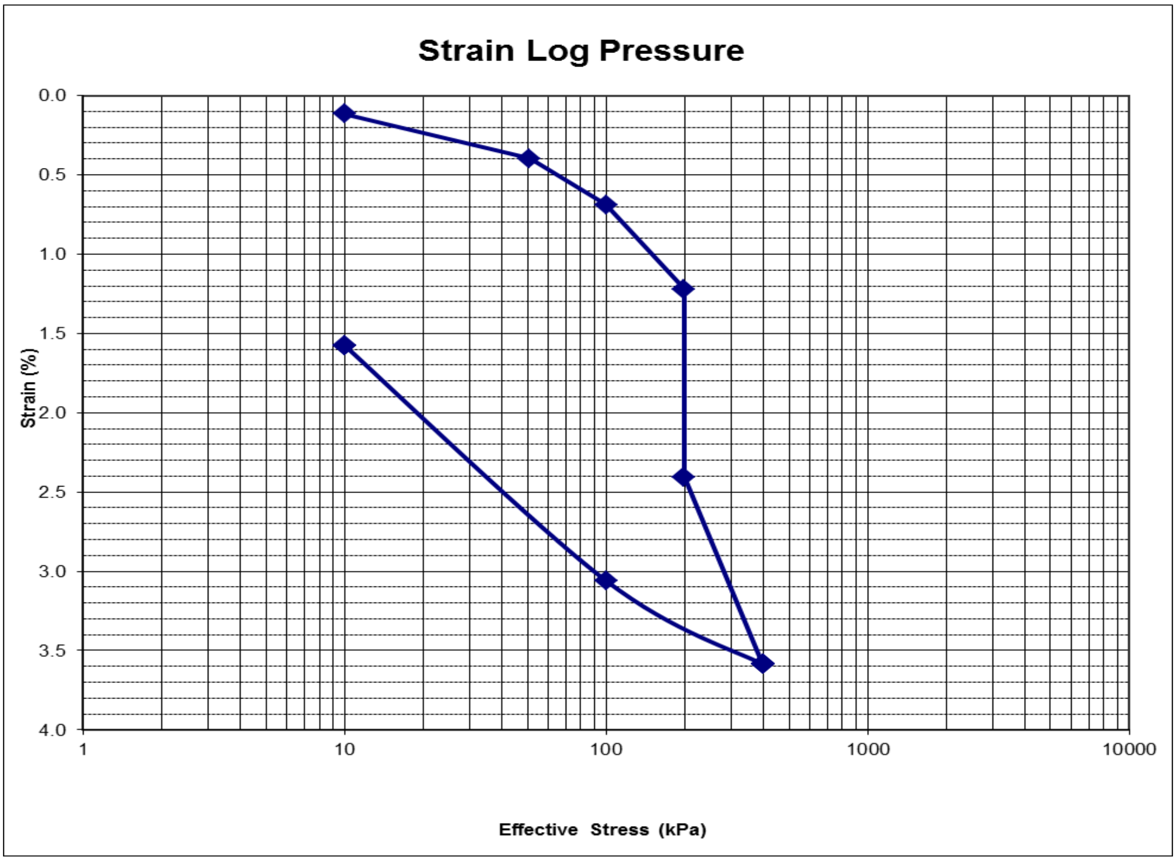


Figure F.11: Strain (%) – Effective Stress (kPa) graph

Table F.10: Consolidation test on the undisturbed sample taken at test pit 1, 0 – 600mm – Void Ratio vs. Effective Stress (kPa)

Effect. Stress (kPa)	10	51	100	198	198	398	100	10
Strain (%)	0.12	0.40	0.69	1.22	2.41	3.59	3.06	1.58
Mv (1/MPa)		0.0683	0.0594	0.054		0.0591	0.0177	0.1648
Void Ratio	0.3084	0.3048	0.3009	0.294	0.2784	0.2629	0.2698	0.2893

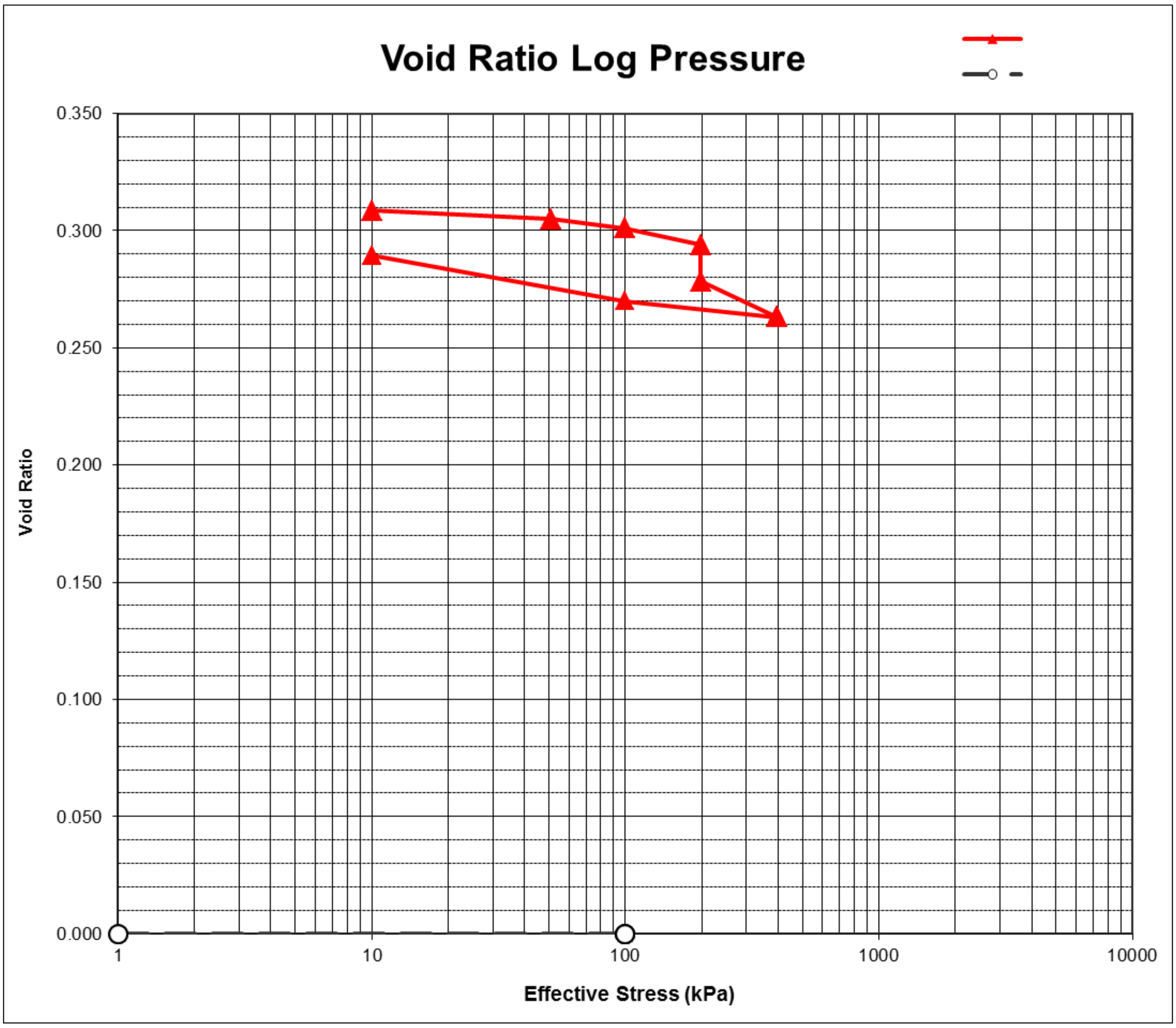


Figure F.12: Void Ratio – Effective Stress (kPa) graph

F.10 Conclusion of the Geotechnical Report

Finally the geotechnical report indicated that the structural engineers should design for expansive soils with a total differential heave of 15 – 30mm based on the visual findings made on site. The empirical method by van der Merwe (1964) failed to predict the heave potential of the materials but was utilised to determine the total estimated heave as 13.9mm.

The upper strata was identified as collapsible (Aeolian sands). The geotechnical report suggested removing the *in-situ* transported soils up to a depth of 800mm on the foundation perimeters. Wetting and high impact compaction was also advised to facilitate potential collapse due to the depth of the Aeolian sand profile.

The laboratory analysis and field investigation under estimated the heave potential of the clays on site and a previous report on the same study area, presumably prior to the construction of the first classrooms, also failed to predict that heaving clays were a geotechnical constraint. The author had the benefit of witnessing expansive failure on site to adjust the report to reflect heaving clays as a geotechnical constraint. The original investigator did not have this luxury.

This geotechnical report and a similar one at Botshabelo Section K was the motivation for this study.