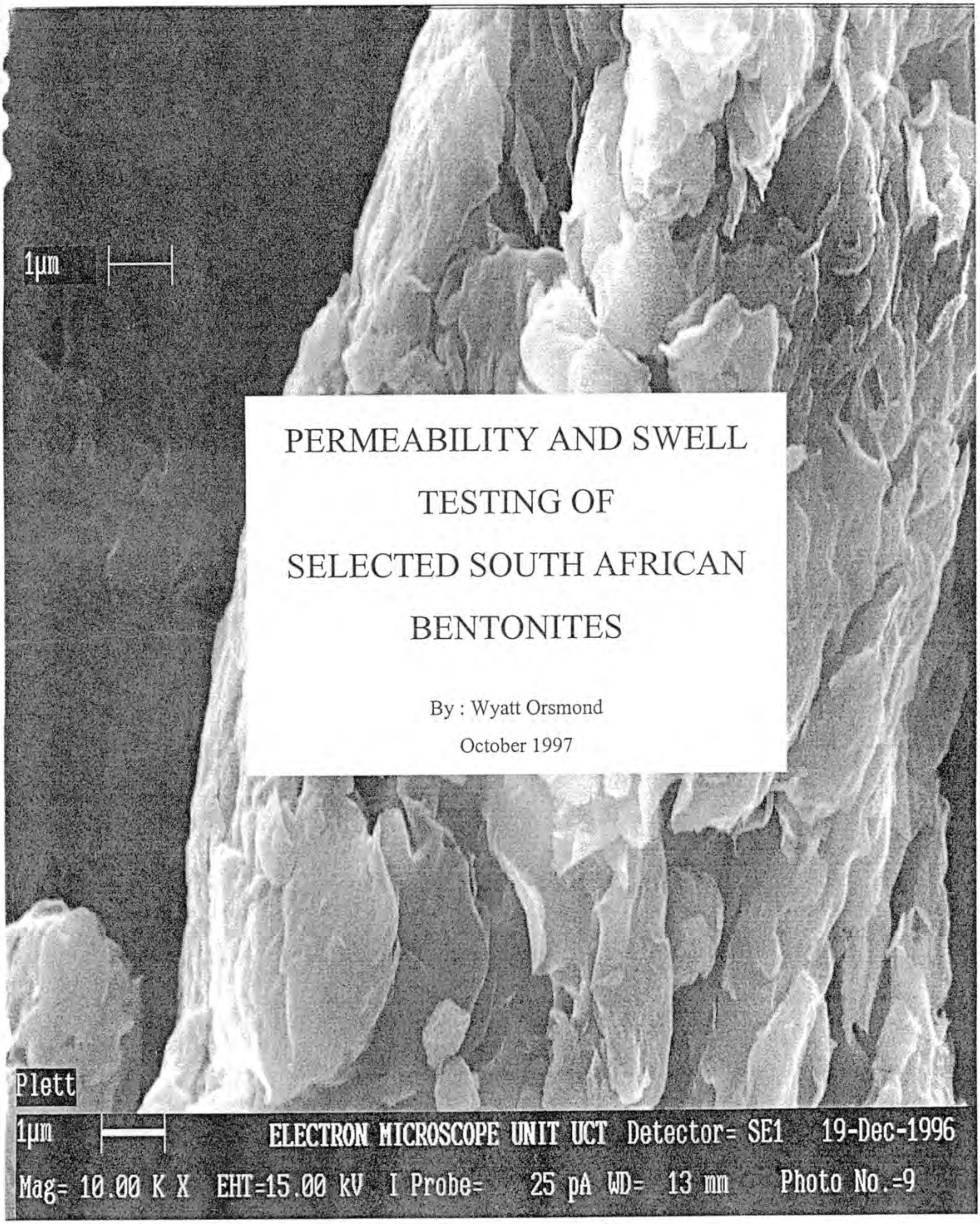

PERMEABILITY AND SWELL TESTING OF SELECTED SOUTH AFRICAN BENTONITES

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DEDICATION

To my wife whose patience has been my motivation.
Thank you !

Synopsis

Bentonite, a natural clay composed predominately of montmorillonite has become a very useful product in Civil Engineering due to its extremely low permeability and high swell potential. Bentonite has been used extensively in countries like the United States of America, Canada and Europe as an impervious liner in dams, landfill sites, retention ponds etc. where it is used in its pure form and as a composite material mixed with soil. Bentonite liners are rapidly gaining interest in many other countries including Southern Africa.

Bentonite is found world wide, and in South Africa (RSA) there are three deposits presently being mined namely Heidelburg (Western Cape), Koppies (Orange Free State) and Plettenberg Bay (Eastern Cape). The bentonite from each of these deposits are called Culseal / Culbond, Koppies and Plett respectively. They all vary with regard to chemical composition and little is known about there engineering properties.

The aim of this research is to provide information both general and with regard to Civil Engineering on the bentonites presently being mined in South Africa to facilitate design and future research in this field.

The triple layer crystalline sheets which make up a bentonite particle are held together by weak forces and exchangeable cations (which dictate the bentonite type). When brought into contact with water, the water is drawn onto the sheets to balance the charges forcing the layers apart resulting in swelling of up to 15 times the original particle thickness. This swollen, hydrated bentonite is almost impervious.

The quality of bentonite with regard to the engineering aspects of permeability and swell can be related to the ratio of certain elements present in the structure. For sodium bentonites the Silica:Alumina ratio is of importance (Reschke and Haug, 1991). Culseal, a sodium bentonite rates as medium quality in this regard., but if its magnesium content is examined as with Koppies and Plett (calcium bentonites), all of them rate as good quality bentonites. Very low percentages of impurities were found in all of the RSA bentonites tested. From an electron microscope analysis it was found that Koppies possesses the most compact structure with Plett showing a partly foliated structure and Culbond displaying a compact, granular structure.

Atterberg limits and specific gravity tests were attempted on the selected bentonites. The Atterberg limits provided results for Plett and Koppies which plotted perfectly in the montmorillonite zone on the Casagrande Plasticity Chart. Culbond plotted on the edge of the zone. The specific gravity tests however, proved an almost impossible task to perform in the standard method.

The results of the swell tests showed that even under a minor surcharge of 25 Pa, bentonite would swell to only a third of its free swollen volume. It was also found that a thicker layer of bentonite experiences a lower percentage swell than a thinner layer. This lead to the development on the Zero Thickness Free Swell Index which eliminates the effect of sample thickness and the problem of non-settlement of dispersive samples experienced in other swell tests. The Zero Thickness Free Swell Index for the RSA bentonites were : Culseal 681%, Koppies 378% and Plett 273%.

Permeability testing of the pure bentonites was conducted in a consolidometer permeameter, triaxial cell, standard permeameter and a new device called the Mini Permeameter developed as an economical alternative to standard geotechnical laboratory apparatus. Typical results achieved in the Mini Permeameter were: Koppies $6x10^{-11}$m/s, Culseal and Plett $8x10^{-11}$m/s, and Culbond $9x10^{-11}$m/s. Extensive testing found these values to reduce by about 40% after 120 days of permeating. The coefficients of permeability varied between the tests with the greatest variation coming from the consolidometer permeameter where the permeability coefficients were one order of magnitude lower.

The use of bentonite mixed with soil to form an impervious layer is favoured structurally and economically to a pure bentonite layer. Mix proportions are critical to achieve low permeability. Extensive testing was done in a double-ring compaction mold permeameter and a consolidometer permeameter.

The amount of bentonite in a mix has a direct effect on the permeability up to an optimum whereupon an increase in bentonite content does little to improve impermeability. Tests showed that for Cape Flats sand the optimum bentonite content was in the order of 10% by mass. This provided a coefficient of permeability of about $7.0x10^{-11}$m/s. Below 6%, the permeability increased dramatically indicating that the voids within the soil were not filled sufficiently with bentonite.

The coefficients of permeability achieved in the double-ring compaction mold permeameter were almost identical to those achieved in the consolidometer permeameter under a 56 kPa vertical stress. After complete hydration, typical results were : Plett $3.2x10^{-11}$m/s, Koppies $3.7x10^{-11}$m/s, Culbond and Culseal $6.0x10^{-11}$m/s. The tests conducted in the consolidometer showed that the permeability decreased by almost half an order of magnitude with an increase in stress from 56 kPa to 224 kPa.

The consolidometer tests revealed a connection between the coefficient of volume compressibility (mv) and the percentage of bentonite in a mix. It was found that the higher percentage mixes provided higher mv coefficients.

A great deal more research of the Engineering Properties of South African Bentonites is needed to achieve a proper understanding of these materials, and it is hoped that this thesis will form the base of the future research.

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Glossary

Sedigraph Analysis	Particle size analysis using x-ray diffraction
Deuteric	The alteration of molecular structure
Anion	A positively charged particle
Cation	A negatively charged particle
Van der Waal's forces	Forces of attraction between particles
Hydroxyl	An element containing one oxygen and four hydrogen atoms
Valence	The elective affinity, or attraction power
Lamellar	A flat sheet/layer
Foliated	A folded sheet in an open loose structure
Thixotropic	The ability of a viscous liquid to possess low viscosity when undisturbed, i.e. to gel

List of Symbols

C_1	value related to particle size distribution (Hazen, 1892)
D_{10}	soil grading dimension
e	void ratio
Es	constraint modulus (Young's Modulus)
g	gram
Δh	hydrostatic pressure
i	hydraulic gradient
k	coefficient of permeability
k_t	tortuosity factor
k_o	shape factor
kPa	kilo Pascal
K	empirical constant
$k_{ave.}$	average coefficient of permeability
LL	liquid limit
L	length
m	meters
mv	coefficient of volume compressibility
n	soil porosity
η	coefficient of viscosity of the liquid
ppm	parts per million
PI	plastic index
PL	plastic limit
q	flow volume
S_s	surface of voids
S.G.	specific gravity
s	seconds
v	flow rate or discharge velocity
v_s	apparent seepage velocity
δ_w	is the unit weight of water
$\varnothing$	diameter

PERMEABILITY AND SWELL TESTING
OF
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1. Introduction

Bentonite, a natural clay composed predominately of the clay mineral montmorillonite has become a very useful product in industry due to its extraordinary and varied properties. Two of these properties namely; extremely low permeability and high swell potential make bentonite a viable product in the field of Civil Engineering.

Bentonite has been used extensively in countries like the United States of America, Canada and Europe as an impervious liner in dams, landfill sites, retention ponds etc. both in its pure form and as a composite material mixed with soil, and is rapidly gaining interest in many other countries including Southern Africa.

The material is found world wide, and in South Africa there are three deposits presently being mined. They are Heidelburg in the Western Cape, Koppies in the Orange Free State and to a lesser extent Plettenberg Bay in the Eastern Cape. The bentonite from each of these deposits varies with regard to chemical composition and they also possess different values of hydraulic conductivity and swell. Extensive research has been done on the bentonites found in the United States and elsewhere but little is known about the engineering properties of the local deposits.

A number of failures have occurred in bentonite liners throughout the world and South Africa is no exception. Two of the main reasons for the failures are a lack of basic testing of the material and a lack of general knowledge and information on bentonite.

The aim of this research is to provide information both general and with regard to Civil Engineering on the bentonites presently being mined in South Africa to facilitate design and future research in this field.

The thesis is divided into three sections. The first section provides information on the more general nature of bentonite to facilitate in the understanding of the chemical and geotechnical properties leading into a more in-depth view of the selected South African bentonites. It starts with a brief geological history of bentonite and then provides an overview of clay minerals and the effects of contact with water. Before examining the seepage characteristics in bentonite, an overview of permeability in general is given. A variety of uses of bentonite both general and related to Civil Engineering is then presented before an in-depth analysis of the South African bentonites is done incorporating a description of the selected products, a chemical analysis, and a particle analysis using a Sedigraph and an electron microscope.

The second section concentrates on the testing of the engineering properties of the selected bentonites. The section starts with an experimental investigation of the basic engineering aspects incorporating specific gravity and the Atterberg limits. An analysis of the swelling characteristics is then presented with a new approach of providing a swell index for soils which remain suspended in water. The main emphasis of this research is directed towards the investigation of the permeability characteristics of the South African bentonites both in its pure form and mixed with sand. An overview of permeability testing apparatus and an analysis of the permeant liquid used is provided before presenting the permeability testing of bentonites in their pure form followed by permeability testing of soil/bentonite mixes. Discussions on permeability testing are provided after each relevant section and finally conclusions on the testing of South African bentonite are drawn.

The third section provides guidelines based on literature and information gained from this research for designing bentonite liners using the South African products. It offers selection criteria for the type of bentonite liner system and provides a preliminary theoretical design for soil/bentonite liners. Advice on construction method and control is also provided.

Section A General Review of Bentonite

2. Chemical and Geotechnical properties.

2.1. General Definition

Bentonite is a naturally occurring clay mineral composed predominantly of the clay mineral, montmorillonite. Bentonite has many extraordinary properties which have made it a very useful product in industry. It has an excellent swelling capacity, absorption ability, catalytic action, bonding power, cation exchangeability, thixotropic ability, and is almost impermeable. There are a wide variety of forms of bentonite as a result of the formation of the deposits millions of years ago, and the properties vary from one form to the next (van der Merwe, 1993; Noble, 1987; Grim and Güven, 1978).

The form of bentonite is related to its exchangeable cation which can be either calcium, sodium, magnesium or potassium. It is believed that most of the various forms of bentonite were created when volcanic ash or tuff was deposited in either salty, brackish, or fresh water. Pure calcium bentonite which is often referred to as 'low swelling bentonite' was produced as a result of deposition in fresh water. Sodium bentonite, a more rare form, was created when the ash or tuff was deposited in salt water, exchanging its calcium element for sodium, and is generally referred to as 'high swelling bentonite'. Bentonite is also thought to have originated from the hydrothermal and deuteric alteration of igneous rock. Deuteric is used in this instance to indicate the changes of a material caused by the reaction of gases and vapors within the igneous rock mass (van der Merwe, 1993; Grim and Güven, 1978).

The quality of bentonite is related to its pureness within its chemical structure and within the overall deposit. Not only does the quality vary between deposits but can fluctuate within a single deposit. This variation is usually the result of the initial alteration process and is evident by the presence of other clays essentially consisting of the minerals illite or kaolinite. Although some deposits may appear to be of a pure quality they could contain 10 % or more non-clay minerals (Vosloo, 1990; Boyes, 1986). Bentonite is usually mined in an open pit and is then dried, ground and sometimes given additional treatment before distribution (e.g. sodium activation also referred to as peptising).

The name 'bentonite' dates back to the year 1898 when it was first used to describe a specific type of clay found near Fort Benton in Wyoming , USA, a sodium bentonite. The name was later used to describe a clay derived from the alteration of volcanic ash or tuff consisting mainly of montmorillonite. In the United Kingdom, Calcium bentonite is referred to as Fullers earth, although Fuller's earth generally refers to any clay which has the ability to decolorize oil. (Vosloo, 1990; Cullinan, 1988)

The term bentonite is now used extensively to describe any clay which is composed predominantly of the montmorillonite clay mineral, and is used in this thesis.

2.2. Structure of Clay Minerals

All clay minerals are made up of two basic crystalline sheets, namely the tetrahedral or silica crystal structure and the octahedral or alumina crystal structure.

The tetrahedral crystal consists of four oxygen atoms surrounding one silicon atom in a tetrahedral (pyramid) form. The oxygen atoms at the base of the tetrahedral lie in one plane and are shared by two tetrahedrals to form a sheet referred to as the silica sheet.

The octahedral crystal comprises six oxygen or hydroxyls surrounding an aluminum (Al), magnesium (Mg), iron (Fe) or other atoms in an octahedral form. The oxygen and hydroxyl atoms (whichever present) are referred to as the anions and the central atom (e.g. Al, Mg, Fe etc.) is referred to as the cation. The rows of the oxygen atoms or hydroxyls in the sheet lie in two planes, and the sheet is referred to as the alumina sheet. Substitution of the different cations within the octahedral sheet is rather common and this results in a different clay mineral (Holtz and Kovacs, 1981)

All clays can be identified by the unique way in which the two basic sheets are stacked and by the presence of certain cations in the octahedral sheet.

The montmorillonite clay mineral also sometimes called smectite, consists of two silica sheets and one alumina sheet. In this specific clay, all the anions of the alumina sheet are hydroxyls and two thirds of the cation positions are filled with aluminum atoms. The alumina sheet is sandwiched between the two silica sheets which are arranged with the tips of the tetrahedrons (or the apex of the pyramid) combining with the hydroxyls of the alumina sheet. The triple sheeted montmorillonite layer is bonded to the next montmorillonite layer by weak van der Waal's forces, and a net negative charge deficiency exists in the alumina sheet. These are neutralized between the layers by loose exchangeable cations. A diagrammatic representation of a montmorillonite sheet is shown in Fig.1. The unhydrated montmorillonite sheets resemble a pack of cards or closed book appearance. When brought into contact with water, the cations and negative charges on the sheets attract the water molecules and the sheets are pushed apart. This is explained in detail in the following section on the 'Hydration and Swell Process'.

Bentonite is primarily composed of montmorillonite and is represented by the following chemical formula (Grim and Güven, 1978):

$$\boxed{M(Al, Fe, Mg)_4(AlSi)_8O_{20}OH_4.nH_2O}$$

The 'M' represents the exchangeable cations which is related to the form of bentonite and could be either calcium (Ca), magnesium (Mg), sodium (Na), or potassium (K). The other elements present are: iron (Fe), aluminum (Al), silicone (Si), oxygen (O), hydroxyl (OH_4) and water (H_2O).

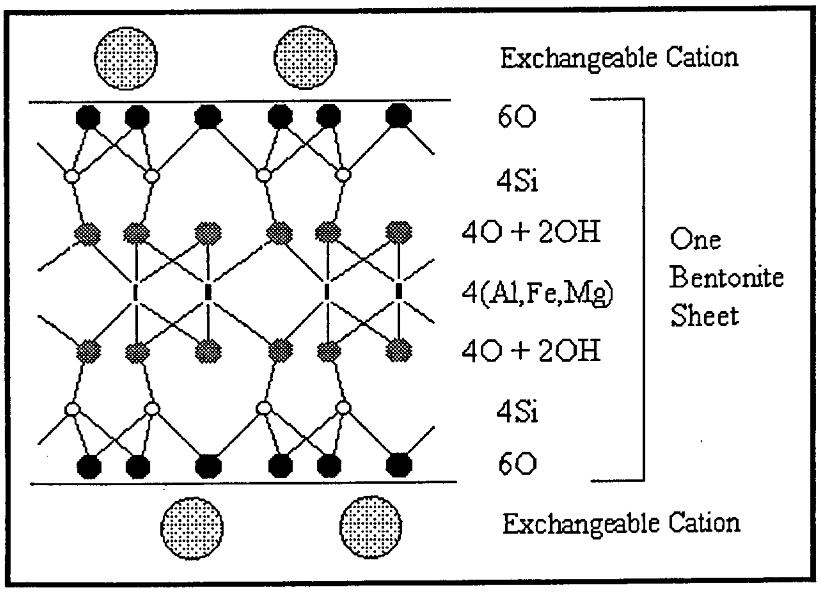


Fig. 1: Structure of Montmorillonite Clay Mineral after Grim and Güven (1978).

The montmorillonite group of clays has by far the smallest layer thickness of all the clays. Its typical layer thickness is about 3nm in comparison to illite and chlorite with a layer thickness of 30nm and kaolinite with a thickness of 50nm to 2000nm. The montmorillonite clays have therefore the greatest specific surface of approximately 0.8 km^2/gram, 10 times larger than illite and chlorite and 50 times larger than kaolinite. The result of this is that montmorillonite clays possess the greatest number of cations and net negative charges in all of the clays and are therefore regarded as the most active.

Soda ash (Na_2CO_3) is frequently mixed into calcium bentonites (termed peptizing or sodium activation) to improve the quality by providing more Na exchangeable cations. Bentonites are also improved through polymer treatment. This involves the addition of a polymer of the sodium polyacrylate and acrylonitrile type which attaches itself to the platelets through iron exchange (Alther, 1987;.Reschke and Haug, 1991) The result of using Soda ash and polymers are further explained in the 'Hydration and Swell Process'.

2.3. Hydration and Swell Process

When water is brought into contact with a bentonite particle, the cations between the layers of the crystal structure and the negative charges on the sheets draw the water into the particle between the sheets to balance the charges. The result of this is an increase in the volume of the material.

There are three factors according to Holtz and Kovacs (1981), which effect the manner in which water is adsorbed into clay. They are:

1. Water is dipolar, even though it is electrically neutral, it has two separate centers of charge, one negative and one positive. These are electrostatically attracted to the surface of the clay crystal due to the crystals negative charge.
2. The hydrogen within the water molecule is attracted to the hydroxyls on the surface of the clay. This is referred to as 'hydrogen bonding'.
3. The negatively charged clay surface attracts cations present in the water, and depending on the cations, they may in-turn also contribute due to their own attraction to the clay surface.

Of these three factors, hydrogen bonding is probably the most important. The water molecules right at the clay surface are bonded very tightly and are strongly orientated. The cations orientate the water molecules away from the sheets effectively pushing the sheets apart resulting in the particle of bentonite expanding. The water molecules drawn between the sheets have a rigid structure similar to that of an ice crystal. The water molecules behind the layer of tightly held molecules are held more loosely and are termed 'free water' (Holtz and Kovacs, 1981; Trauger, 1992; Kenny et al., 1992).

The 'broken' edges of the clay crystalline sheets contribute greatly to the available unbalanced valence charges. Since the clay crystals wish to be charge neutral, cations in the water are attracted to the clay in varying degrees depending on the charges present. Different clays have different charge deficiencies and therefore have different tendencies to attract exchangeable cations. They are called exchangeable cations since one cation can easily be exchanged for another of similar valence or by two cations with half the valence of the original cation.

Calcium and magnesium are the predominant exchangeable cations in soils, potassium and sodium are less common. The ions present in the soil structure are governed by the original formation of the soil as previously indicated.

The ease at which cations are replaced depends on several factors, but in particular the valence of the cation. Higher valence cations easily replace lower valence cations. An approximate order of the cation replacement ability in order of increasing replacement power is presented by Holtz and Kovacs (1981) and is as follows :

$$Li^+ < Na^+ < H^+ < K^+ < NH_4^+ < Mg^{++} < Ca^{++} < Al^{++}$$

Practical consequences of ion exchange are for example the lime stabilization of clayey soils which reduces the exchangeable ions and makes the clay more 'stable', and peptizing of bentonite which increases the ion exchange capabilities making the material more 'active'..

In the case of sodium bentonite, the cations are loosely held and easily exchangeable. When exposed to water, the fluid drawn into the particle forms thick, viscous diffuse ionic layers around the sheets pushing the sheets apart resulting in swelling of up to 15 times or possible total separation of the sheets. Salts, organics, acids etc. present in the hydrating fluid tend to neutralize the negatively charged sheets which in turn results in less water absorption and therefore reduced swell (Kenney et al.,1992; Daniel, 1985)

Calcium bentonites' dominant exchangeable cations are polyvalent. The calcium cation is very effective in holding the sheets of montmorillonite together resulting in substantially lower swelling than sodium bentonite. By adding soda ash to calcium bentonite, the Na atom in the ash adds to the bentonites exchangeable cation and therefore increases the water absorption between the sheets resulting in a swell increase. Similarly, when the bentonite is polymer treated, the polymers attach themselves to the sheets and in the presence of water, act like a spring forcing the platelets apart (Kenney et al., 1992; Alther, 1987; Grim and Güven, 1978).

2.4. Permeability Process

Liquids pass through soil by the processes of seepage and diffusion. Seepage occurs when there is a hydraulic gradient across the soil body, while diffusion occurs when there is a chemical gradient across the soil body. In this research only clean natural water was used as a permeant and it is presumed that the movement of liquid through the test specimens occurred as a result of seepage. This movement of water through a soil structure is refered to as permeability. Before an attempt is made to understand the process of how liquid moves through bentonite, it is necessary to understand permeability in the broader context.

2.4.1. Overview of Permeability in porous media

Permeability is one of the main properties of soil and is defined as the ratio of the rate of flow to the hydraulic gradient. The flow rate (v), also called the discharge velocity, is the average rate of flow through the sample. It is understood that the actual velocity of the liquid flowing through the sample is greater than the straight line velocity due to the liquid having to travel around the particles and not along a straight line as depicted schematically in Fig. 2.

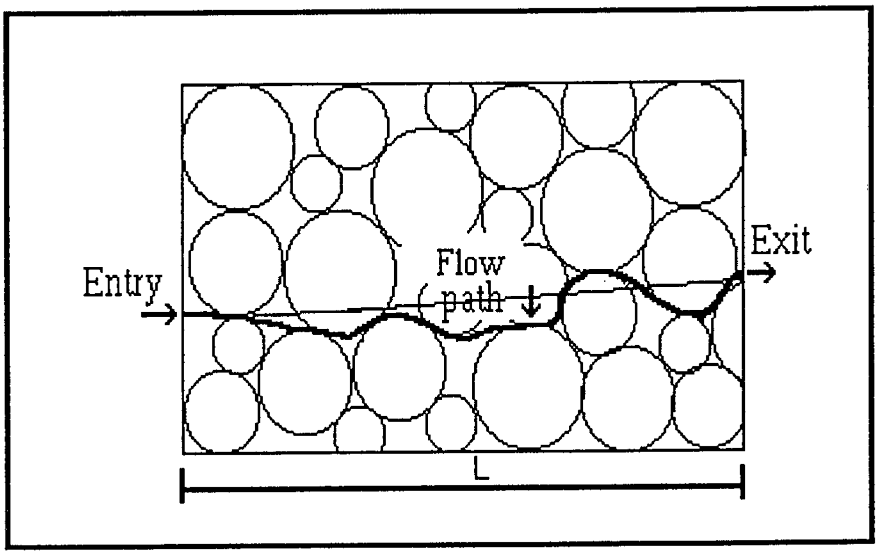


Fig. 2: Flow path in a soil sample of length L.

The actual velocity or seepage velocity (V_s) of the permeant is derived from the apparent discharge velocity (v) by:

$$\boxed{Vs = (v/n)}$$

where: n is the soil porosity.

The porosity is the ratio of the volume of voids to the total volume of a soil sample. This shows that as the particle size decreases, the porosity increases towards 100% and the real velocity approaches the value of the discharge velocity. In other words, the flow path tends towards a straight line.

In order to get the permeant to flow through the soil a difference in pressure is required across the sample. This is the same situation experienced in hydraulics and explained by the Bernoulli equation. The excess hydrostatic pressure (Δh) represents the energy which is required to push the water a straight line distance (L) through the soil. A dimensionless ratio representing this pressure difference is the hydraulic gradient given as:

$$\boxed{i = (\Delta h/L)}$$

For fine sands and silts the relationship between the hydraulic gradient (i) and the flow rate (v) is expressed by:

$$\boxed{v = \frac{K}{\eta} \delta_w \, i}$$

where: η is the coefficient of viscosity of the liquid and to some extent is dependent on temperature.

K (m^2) is an empirical constant

δ_w is the unit weight of water

In physics, K is called the coefficient of permeability and is dependent on liquid viscosity and temperature. In Civil Engineering the permeant liquid is generally water and the temperatures encountered are almost constant. It is therefore customary to assume that the value of η and δ_w are constant and to substitute

$$\boxed{k = \frac{K}{\eta}\delta_w}$$

into the above equation to eliminate liquid viscosity and water density to give:

$$\boxed{v = (ki) = k\left(\frac{\Delta h}{L}\right)}$$

The law expressed by the above equation is called Darcy's law (Darcy, 1856). This equation can be modified to represent a quantity of water flowing per unit of time and is expressed as:

$$\boxed{q = Aki}$$

The value of 'k' is also called the coefficient of permeability but is different to the 'K' used by physicists in that it has the dimension of velocity,
i.e. meter per second.

In an attempt to model permeability, Hazen (1892) took account of the particle grading and proposed the relationship:

$$\boxed{k = C_1 D^2_{10} \text{ (m/s)}}$$

where 10 % of the soil consists of particles smaller than D_{10} and
C_1 is a value related to particle size distribution.(e.g. $C_1=10^{-2}$ (mm) for sand)

This semi-emperical formula is very limited and can only provide an approximate permeability value for sands.

A formula covering more aspects and variations was presented by Kozeny-Carmen and takes the shape of (Pilot, 1982):

$$\boxed{k = \frac{1}{k_0 k_t^2} \frac{n^3}{(1-n)^2 S_s^2}}$$

where : k_o is the pore shape factor
k_t is the tortuosity factor
n is the porosity
S_s is the surface of voids per unit volume of solid

The above formula is also limited in that it does not take into account the effect of the hydraulic gradient across the sample.

The only way in which an accurate value of k can be determined is by physically testing the soil in question in a permeability test in the laboratory or in its natural formation using various techniques discussed later.

2.4.2. Seepage through Bentonite

Permeability measurements using Darcy's law is based on fine sands and silts, and it is understood that the velocity, which is measured on the straight line flow path, is affected by the soil porosity.

Pure dry processed bentonite is generally a fine powder with a bulk density of approximately 1000 kg/m^3 with a specific gravity in the order of 2.7. In theory this would result in a porosity of 0.63 and would therefore provide a seepage velocity of 59% greater than the discharge velocity. This is calculated using the formula:

$$\boxed{V_s = (v/n)}$$

When hydrated, this changes significantly due to the swelling action of bentonite. When brought into contact with water, the dry density of the swollen bentonite drops due to the particles being forced further apart. The lower density results theoretically in a higher porosity, and therefore the seepage velocity (V_s) would be nearer the value of the discharge velocity (v). An example may clarify this: if the swollen dry density is 750 kg/m^3, the porosity would increase from 0.63 to 0.72 and the seepage velocity would reduce from 59% to 38% greater than the discharge velocity.

This demonstrates that although the theory for permeability for porous media may hold if bentonite remained in a fixed grain form, it does not allow for the fact that when hydrated, the particles of bentonite are split apart into thin wafer-like sheets. The process of how water is absorbed into the particle between the sheets causing swelling is well understood, but uncertainty exists with regard to how water moves through a hydrated bentonite layer. The theory may apply to some degree to the soil component of a soil/bentonite mixture but the overall mode of seepage would be even more complex.

Although soil/bentonite mixes are regarded as uniform in structure and consistency allowing the use of Darcy's law which assumes that the flow is uniform throughout the specimen, this is not entirely correct. Edil and Erickson (1985) note that even after compaction near optimum moisture content, the mix appears to continue to hydrate while the permeability test proceeds. As the bentonite absorbs the permeant, zones of differential hydration may develop within the sample. Wetter or fully hydrated zones would transmit a larger portion of the flow through the sample resulting in unequal distribution of flow across the sample. This is a violation of the assumptions on which Darcy's law is based. The permeability of the wetter zones could be far higher than the measured value for the overall sample.

Once total hydration of all the bentonite within the mix has been reached, it could then be assumed that flow is now uniform and Darcy's law can be applied. However exactly when full hydration of the soil is achieved is uncertain.

In this report all the investigations into permeability are based on Darcy's law for the purpose of comparison, and no attempt was made to model the process of permeability in bentonites.

2.5. Industrial Applications of Bentonite

2.5.1. General Uses

Bentonites unique physical properties have given it great commercial value. The bentonite industry developed in the United States in the beginning of the twentieth century is spreading rapidly and bentonite is now mined world-wide. The development of bentonite for industrial purposes stimulated extensive research which resulted in wider applications and uses. A brief summary of the typical uses of South African bentonite presented by Vosloo (1990)and by van der Merwe (1993) are as follows:

- Foundries: - used as a bonding agent to make casting molds.
- Drilling mud: - it seals the porous rock, lubricates the drilling bit and due to its particle suspension ability carries cuttings to the surface.
- Refining of wine, beer, fruit juice, and water: - its colloidal and electronegative properties are used to flocculate suspended particles.
- Plasticiser: - added to concrete and ceramic mixes to improve plasticity and prevent water loss.
- Bonding: - used to bind fine ores into pellets or briquettes, as a binder in sheep, pig and cattle feed, and binding the flux onto welding rods.
- Other uses: - X-ray examinations, paper processing, chemical and pharmaceutical products, preventing acidosis in ruminants on low fiber diets.

2.5.2. Uses in Geotechnical and Environmental Engineering

When placed in a continuous layer, either pure, mixed with soil or as a slurry, bentonite can work as a very effective waterproofing liner which stays plastic (if kept moist) and has a self sealing ability. This makes it a very useful product in the construction industry. The utilization is vast but the general methods of application can be broken down into slurry trenches, blanket liners and geosynthetic clay liners.

2.5.2.1.Slurry Trenches

A slurry trench is an excavated trench filled with a bentonite slurry commonly used in diaphragm walls. Between 3% and 7% of bentonite is mixed into the water. The effect of this is twofold. Firstly, the added bentonite produces a thixotropic material which is heavier than water and therefore creates a lateral pressure greater than the hydrostatic pressure of the soil below the ground water table which limits the movement of water into the trench and thus prevents a collapse of the side walls. Secondly, as the slurry permeates out of the trench into the surrounding soil, the bentonite builds up on the wall of the trench and creates a 'filter cake' which limits the seepage of liquids both out and into the trench. The mixture also suspends small pieces of material left by the excavation process which would otherwise drop to the bottom of the trench and would create a weak layer under the reinforced concrete wall cast in-situ into the trench.

In past years, bentonite slurry was used only to stabilize deep trench excavations, but they are now being used extensively on construction sites and in earth dams as cut-off trenches to limit or completely cut off seepage. The effectiveness of the cut-off trench is dependent on the existence of an impervious layer into which the trench can be keyed. A more recent application and one which is gathering much attention is the use of slurry trenches to contain toxic waste in cleanup operations and in the construction of new storage sites. A novel application is the use of cement-bentonite slurries which consist of approximately 15% cement and 5% bentonite mixed with water. The mixture first acts to stabilize the trench during excavation and is then left to harden to form a permanent, plastic, impervious layer. (Gill, 1985; Cullinan, 1988)

2.5.2.2. Blanket Liners

In situations where local clay is unavailable, inferior, or does not possess low permeability i.e. requiring a layer several meters thick, it may be more economical to use a layer of bentonite as an impervious liner. Bentonite blanket liners offer an alternative to high density polyethylene (HDPE), asphalt and concrete liners and can easily provide extremely low coefficients of permeability. Blanket liners are constructed of either pure bentonite or a mixture of bentonite and any non-organic soil.

Pure Blanket Liners

Pure liners consist of a layer of pure bentonite spread evenly over an area to create a impervious layer. The liner is usually between 50mm and 100mm thick and is protected by a layer of selected fill. Due to its relative high cost, Alther (1987) recommends that pure blanket liners are only used for high toxic containment sites where top quality and performance is essential. The bentonite used to construct such a liner is usually in a powder form, making it very difficult for any installation on sloping ground and generally in any exposed environment.

Mixed Blanket Liners

Mixed blanket liners comprise of a mixture of bentonite and soil and are referred to as soil/bentonite (S/B) liners. They are usually about 100mm to 150mm thick with the percentage of bentonite to soil ranging from 3% to 15% by mass depending on the mixing soil and the permeability requirements of the application (Alther 1987).

Kenney et al. (1992) noted that a S/B mixture works as a double component material. In other words, the soil particles are in contact with each other and form the load bearing structure while the bentonite fills the voids between the soil particles and acts as a seepage barrier. The hydraulic conductivity of the mixture is related to that of the bentonite while the sand acts as an impermeable filler. The 'correct' bentonite content evenly mixed into the soil is critical if the dual functioning ability of the mixture is to be exploited. Too little bentonite will result in flow channels occurring through the unfilled voids whereas too much bentonite will limit the load bearing ability of the soil.

S/B liners have a variety of uses varying from wastewater treatment lagoons to solid waste landfill sites. Each application has its own particular design but a good landfill liner design should have a drainage blanket to remove leachant and a capping to stop water entering the landfill and damming up. In order to protect the liner from erosion, drying out and general damage a protective layer of about 300 mm sand or gravel is recommended. In the case of dam liners, rip-rap is required to protect the against wave action.

2.5.2.3. Geosynthetic Clay Liners

Placing bentonite in a pure continuous, even layer to form a hydraulic barrier is a very effective method of waterproofing but is almost impossible to do on site and especially on vertical surfaces. In 1965 a solution to this came with a product known as Volclay Panels, which is still widely used today. It consists of corrugated cardboard panels whose flutes are filled with high swelling sodium bentonite. A more recent development consist of a layer of pure bentonite contained between two geosynthetic sheets. These are referred to as Geosynthetic Clay Liners (GCL).

GCL's have been used successfully on a number of construction projects either as an impervious layer on its own or as the clay portion in a liner system. In toxic landfill sites and mine dump leach pad liners, the layer is used as a secondary liner to the main polymeric geomembrane liner to form a composite liner system (Breitenbach, 1994; Trauger, 1992).

Schematics of the three main types of systems in use today (Daniel, 1993) are shown in Fig.3. They are referred to as: needlepunched sandwiched, glued sandwich and geomembrane glued. The liners all contain about 5 kg/m^2 bentonite and are available in rolls of a width of 4m to 5m and a length of 25m to 60m.. Examples of the different makes are Claymax^R, Bentomat^R and Gundseal. GCL's are capable of achieving permeability coefficients of between 1.0x10^{-10}m/s and 1.0x10^{-12}m/s depending on the overburden stress (Daniel, 1993).

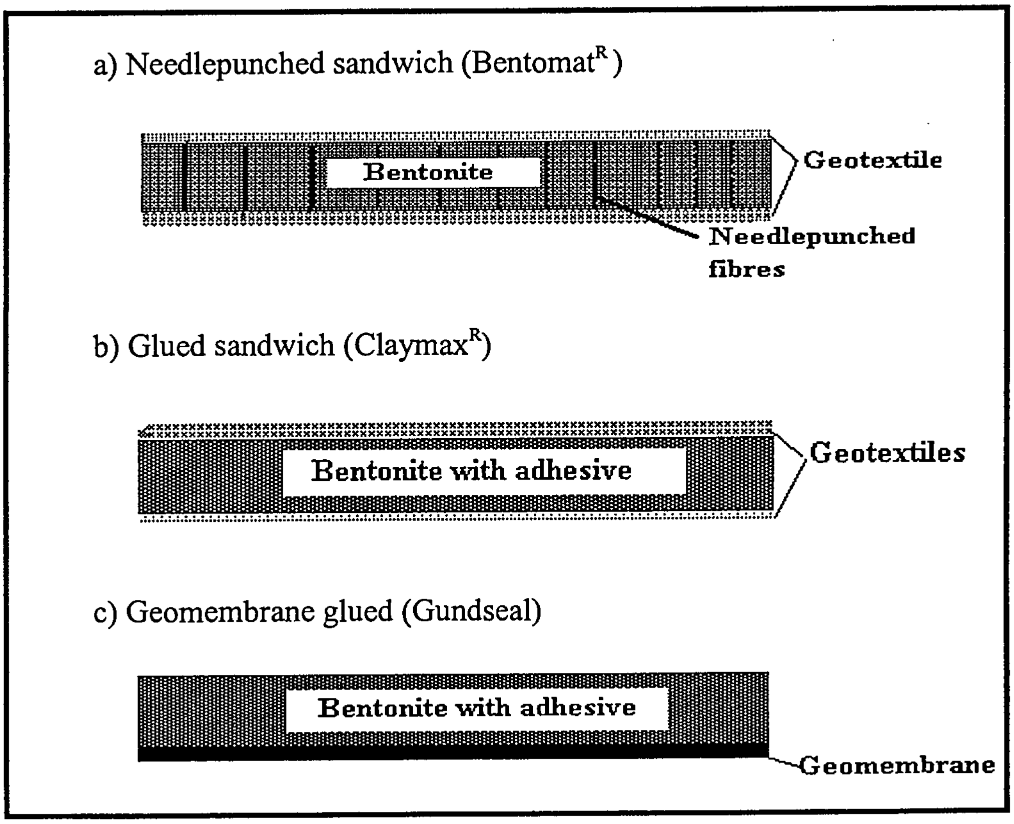


Fig. 3 : Main types of Geosynthetic Clay Liners. (Daniel, 1993)

Bentonite can be used in a number of ways to form an impervious liner, and extensive knowledge of the material is required to insure that it performs in the correct manner. In the United States, Canada and Europe bentonite has been used extensively and a great deal is known about the local deposits, the same can not be said for South Africa where little is known about the local deposits and there is skepticism about the usefulness of bentonite.

3. Analysis of South African Bentonites

In South Africa relatively pure deposits of bentonite exist but the resources are limited. Bentonites are presently being mined in the Southern Cape at Heidelburg and Plettenberg Bay, and at Koppies in the Orange Free State (OFS). Another good deposit occurs at Wodehouse in the Eastern Cape and an impure deposit occurs at Mkuze in Natal. It is interesting to note that the structure of the deposits at Koppies and in the Cape indicate that these bentonites were derived from volcanic ash, although there is no evidence of volcanic activity in these areas at the geological time of formation. The Mkuze deposit is also irregular in that it originates from a perlitic, rhyolitic lava altered by deuteric action (Grim and Güven, 1978; van der Merwe, 1993).

The location of the mines presently producing bentonite, the form and type of deposit, and the main uses of the products are covered briefly. A chemical analysis identifying certain quality characteristics of the South African bentonites is then presented followed by a sedigraph and microscopic analysis of the products. The 'selected bentonites' referred to in this report are the products mentioned in the following section.

3.1. Koppies Deposit

The Koppies deposit in the OFS is mined on the farm Oceaan and the products are available through Boland Base Minerals. The deposit is flat lying and has about 12 meters of shale overburden. Koppies bentonite is of the calcium type and is mined with a moisture content of 30%. The product is treated with 3% soda ash and is orange-white in colour. The products are used mainly for foundry molds, pelletising, water treatment, oil bleaching, and dam sealing. (van der Merwe, 1993; Leppänen, 1996)

The bentonite from the Koppies deposit used for this research is packaged as Oceaan bentonite and was supplied by Boland Base Minerals. The product was in a fine powder form with a bulk density of 997kg/m^3 and a moisture content of 13% and is referred to as 'Koppies' in this report.

3.2. Plettenberg Bay Deposit

Although this deposit is often referred to as the Knysna deposit it is only 5km outside Plettenberg Bay (40 km from Knysna). The mine is an open pit approximately $2500m^2$ and 8 meters deep with the bentonite layer varying from 0.3 to 1.0 meter thick dipping 30° NW to SE. The deposit has calcium and magnesium as its main exchangeable cations. The colour is generally grayish to yellow white but differs with quality and moisture which varies between 33% and 57% between the upper and lower boundaries of the layer. The products are marketed through Boland Base Minerals and its main uses are for foundry molds, dam sealing, fruit juice and wine clarification.
(van der Merwe, 1993; Leppänen, 1996)

The bentonite from the Plettenberg Bay deposit used for this research is packaged as Oceaan Clear (Ca) bentonite and was supplied by Boland Base Minerals. This material was in a fine powder form with a bulk density of 950 kg/m^3 and a moisture content of 14% and is referred to as 'Plett' in this report.

3.3. Heidelburg Deposit

At present sodium bentonite is extracted from three different quarries in the Heidelburg area and is blended according to its final use. The colour of the bentonite is grayish white. Some of the products are pretreated with soda ash depending on their final use. The products are marketed through SAMREC. The biggest market is the steel and iron casting foundries, and it is also used for drilling mud, dam sealing and the wine industry.

Two types of bentonite supplied by Cape Bentonite were used from the Heidelburg deposit for this research. The first type is marketed as Culbond N and is referred to as 'Culbond' in this report. It had a bulk density of 1000 kg/m^3 and a moisture content of 11%. The second product, marketed as Culseal, is a slightly granular material with a bulk density of 1290 kg/m^3 and a moisture content of 11% and is referred to as 'Culseal' in this report. It is not used in all the tests due to its granular nature whereas Culbond is comparable to the other products and is essentially from the same representative deposit. (SAMREC, 1997; van der Merwe, 1993; Leppänen, 1996)

3.4. Chemical Analysis of the selected Bentonites

As previously mentioned, the form of bentonite changes from one deposit to the next and even within a deposit. This is noticeable when examining the chemistry of South African deposits. Grim and Güven (1978) presented the following chemical analysis of Koppies and Plett bentonites:

Sample	SiO_4	Al_2O_3	Fe_2O_3	MgO	Na_2O	CaO	K_2O	Impurity
	%	%	%	%	%	%	%	%
Koppies	66.28	22.19	5.6	3.96	0.02	0.05	0.42	quartz
Plett	67.36	23.72	1.67	4.89	0.11	0.33	1.60	feldspar

Jones (1993) provided an analysis for Koppies, Plett and Culseal which differs from the above and is as follows:

Sample	SiO_4	Al_2O_3	Fe_2O_3	MgO	Na_2O	CaO	K_2O	LOI
	%	%	%	%	%	%	%	%
Koppies	59.1	18.6	4.4	4.0	2.5	1.9	0.3	8.9
Plett	61.0	18.9	4.38	2.40	2.2	1.37	0.73	7.13
Culseal	63.5	15.9	2.87	3.4	3.52	2.23	0.72	7.71

A typical chemical analysis of Culseal and Culbond was provided by SAMREC (1997) and again differs from the previous analysis. The results are as follows:

Sample	SiO_4	Al_2O_3	Fe_2O_3	MgO	Na_2O	CaO	K_2O	LOI
	%	%	%	%	%	%	%	%
Culbond	62.3	18.7	3.5	3.7	2.4	0.8	0.7	7.9
Culseal	62.3	18.7	3.5	3.7	2.4	0.8	0.7	7.9

Although the results of the analyses presented all differ, this is not uncommon. An analysis of the upper, middle and lower layers of Koppies bentonite provided by Grim and Güven (1978), shows variations of up to 5% for SiO_2.

The importance here in the context of engineering quality relating to permeability compared to other international bentonites is the ratio of SiO_2:Al_2O_3 and the MgO content. Reschke and Haug (1991) showed that the higher quality bentonites (with lower permeability coefficients) possess a lower SiO_2:Al_2O_3 ratio and that the quality increases with an increase in MgO content.

For the sodium bentonites tested by Reschke and Haug (1991), the lower quality products ($k_{Ave.} = 7.4x10^{-11}m/s$) had a SiO_2:Al_2O_3 ratio of 5.5:1 whereas the higher quality bentonite ($k_{Ave.} = 2.9x10^{-11}m/s$) had a ratio of 2.9:1. The k values were achieved in a 6% mixture with Ottawa sand and tested under triaxial conditions with a hydraulic gradient of 28.

The only South African sodium bentonites analyzed, namely Culseal and Culbond (from the same source) have a SiO_2:Al_2O_3 ratio of between 3.99 and 3.33:1, which is comparable to a medium quality bentonite tested by Reschke and Haug (1991).

The effect of MgO on permeability quality is related to the particle layer charge. Grim and Güven (1978) showed that the layer charge increases with an increase in MgO. A average quality bentonite with a 3% MgO content has a layer charge of 0.25 and a high quality bentonite with a maximum MgO content of 7.2% has a layer charge of 0.6. In comparison to the bentonites tested by Reschke and Haug (1991) (Na & Ca), and other international bentonites e.g. Wyoming (Na), Brazil (Ca), Canada (Ca) , (Grim and Güven, 1978), all of the South African bentonite show MgO contents in the order of 4% representing good quality bentonite. Comparing the two South African Ca bentonites, Koppies has a lower MgO content than Plett with regard to Grim and Güven's analysis but a greater content with regard to Jones's analysis.

A further aspect affecting quality is the percentage of impurities and non-hydratable montmorillonite. Both of these do not contribute to the swelling and therefore don't contribute to the overall impermeability of the particle structure of the respective bentonite. Layers within the bentonite particle may not hydrate due to having lost their charges and are irreversibly collapsed.

A specific surface test can be done on clays to determine the area available under hydrating conditions. The results are usually lower than the theoretical value of 810 m^2/g (Komine and Ogata, 1996). In comparison to the bentonites tested by Reschke and Haug (1991), the lower quality bentonites had a specific surface area of 467m^2/g and the higher quality bentonites about 668m^2/g. From theoretical calculations (Komine and Ogata (1996)), Culseal and Culbond which possess about 6% impurities have a specific surface area of hydration of 761m^2/g although this does not take into account the collapsed layers. A recent analysis done on Koppies, Plett and Culbond in the Laboratory of Engineering Geology at Tampere University of Technology, Finland (Leppänen, 1996), showed only small amounts of impurities of quartz and feldspar in the bentonites and from this it can be deduced that the specific surface area of hydration is approximately 761m^2/g disregarding the possible collapsed layers. Specific surface tests of Koppies, Plett and Culbond were also conducted at Tampere University using a Flowsorb 2300 (nitrogen adsorption) on the unhydrated dry bentonite. The results were as follows:

Sample	Specific Surface (m^2/g)
Koppies	38.9
Plett	24.7
Culbond	29.7

Although these results do not identify the area available for hydration and therefore the quality of the bentonites, they do however give some idea of the finess of the particles after processing. A more in-depth study of this is done using a Sedigraph and an electron microscope.

3.5. Particle Size Analysis of the selected Bentonites

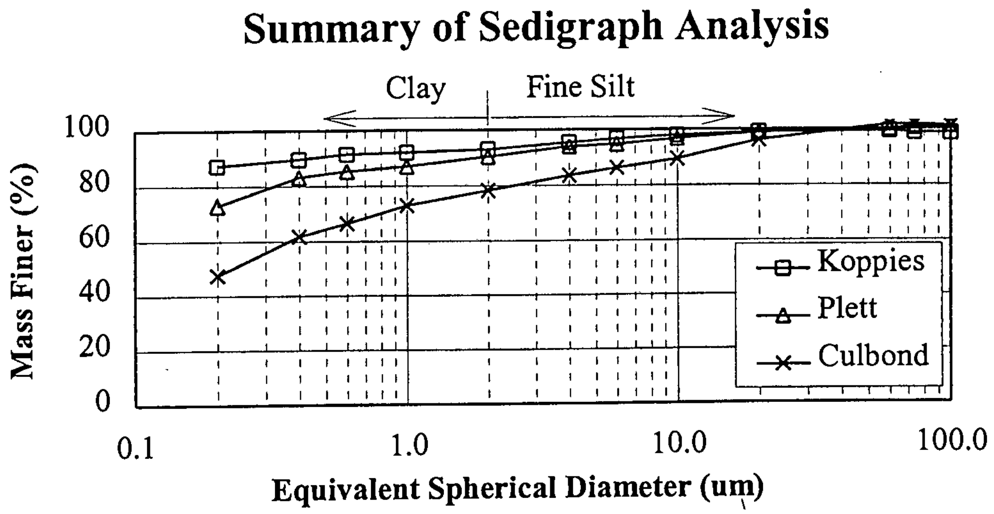
Two important factors affecting bentonite quality are the size and the shape of the particles. The particle size is determined with the aid of a Sedigraph analysis and additionally by the use of an Electron Microscope which is also used to study the particle morphology.

3.5.1. Sedigraph Analysis

The finess of the bentonite particles is important when the material is mixed with soil to form an impervious blanket liner. The finer the particles are, the better distribution occurs within the soil mass, and when hydrated, the probability of filling all the voids in the soil with bentonite is higher.

Due to the minute size of the bentonite particles, a sieve analysis does not provide much information. For example, with Culbond only 4% was retained on the 53 micron sieve (SAMREC, 1997). A hydrometer analysis was attempted by Prevôst (1995) and proved unsuccessful due to the thixotropic nature of bentonite and due to a large percentage of the particles being smaller than 2μm and out of the test range of a hydrometer.

With the help of Minna Leppanen of the Finnish Environmental Agency Laboratory, a Sedigraph analysis was conducted on Koppies, Plett and Culbond to establish the particle distribution from 0.20μm to 100μm. The Sedigraph analysis was done at 30.2°C with a 'high speed' run type lasting about 28.5 minutes. Two driving conditions were used namely till and fat clay. The latter is better for fine soils like bentonite. A summary of the results using fat clay is shown in Graph 1 and full details are available in Appendix A. (Fat clay is termed 'lihavasavi' in Finnish in Appendix A.)



Graph 1 : Sedigraph analysis of the selected bentonites showing the particle distribution from 0.20μm to 100μm.

The results of the Sedigraph show that for instance, 70% of the mass of Culbond is finer than 1.0μm which is the smallest particle size shown on a typical grain size distribution chart. The other bentonite particle sizes are even smaller.

3.5.2. Electron Microscope Analysis.

In order to further analyze particle size distribution between 0.20μm and 100μm, and to do a visual assessment of the particles, an Electron Microscopic study was conducted on Koppies, Plett and Culbond at the Electron Microscope Unit at the University of Cape Town (Dec. 1996).

An analysis of the samples was first done at a magnification of 250 which provides a display of the particles from 100μm down to about 5μm. These are shown in Photo 1, Koppies, Photo 2, Plett, and Photo 3, Culbond. From the photographs it is evident that even in the unhydrated state the majority of the particles are of an equivalent spherical diameter of 20μm or less although the distribution varies considerably to the Sedigraph analysis which is done under hydrated conditions. What is also evident in all the photographs is the 'pack of cards' or 'closed book' structure of the bentonite particles.

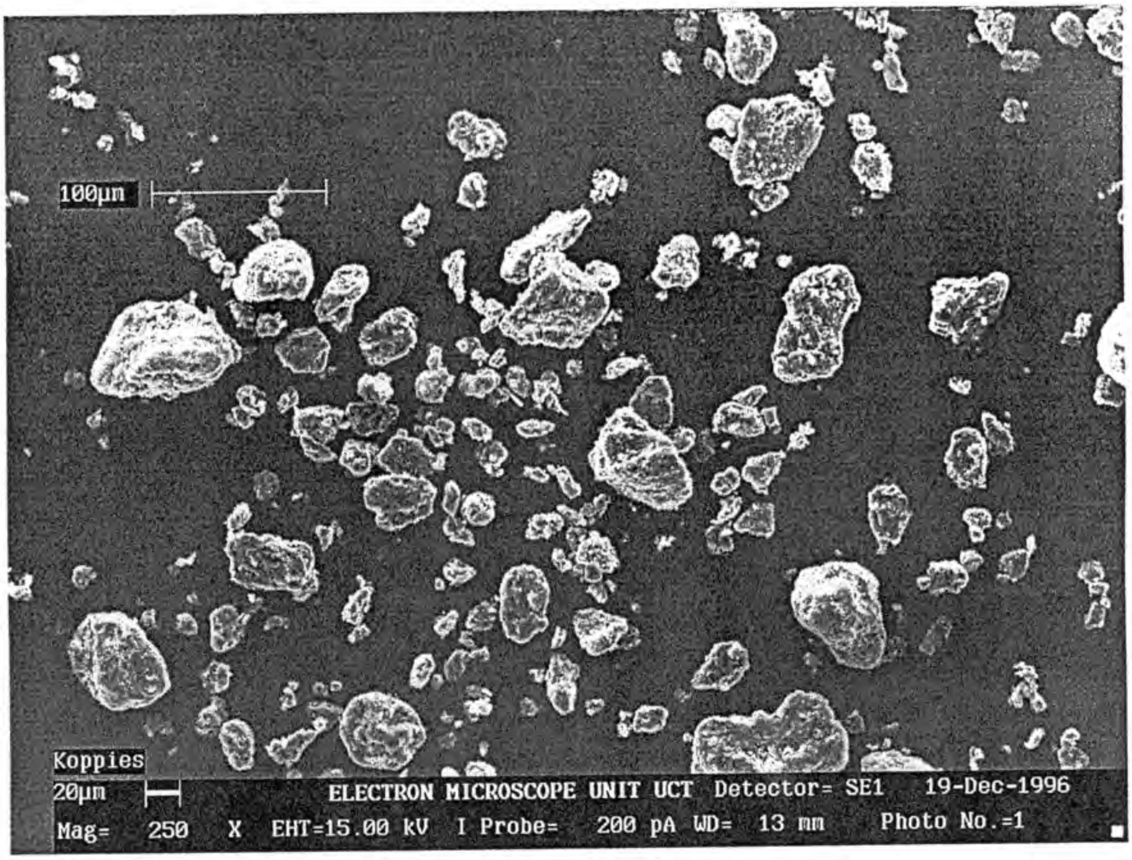


Photo 1 : Koppies bentonite at a magnification of 250.

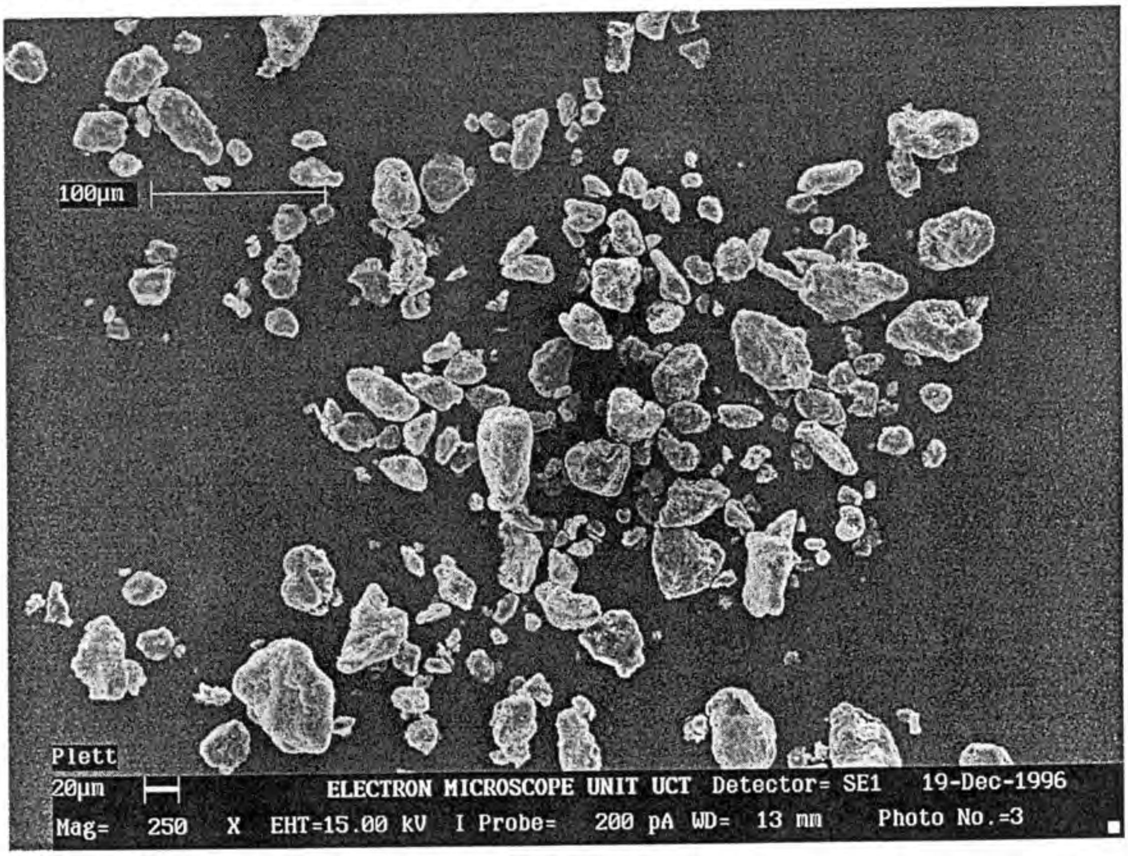


Photo 2 : Plett bentonite at a magnification of 250.

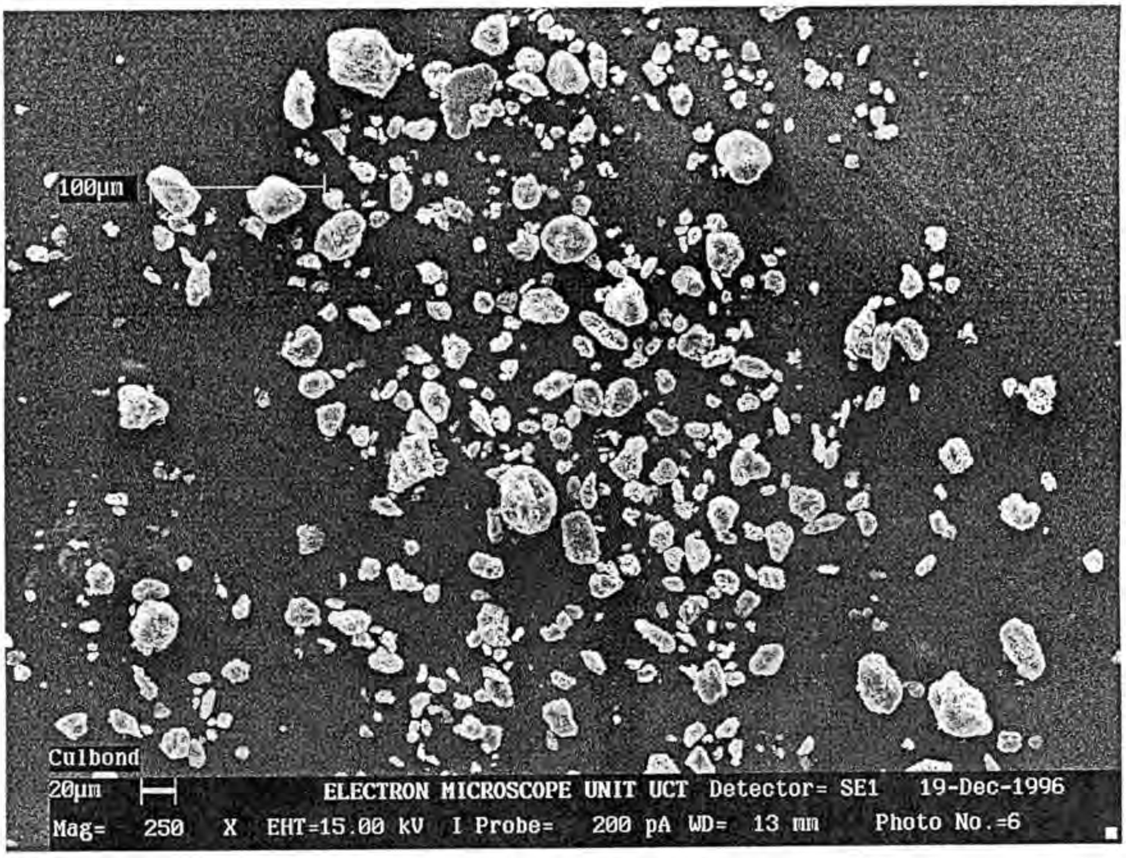


Photo 3 : Culbond bentonite at a magnification of 250.

The particles were then analyzed at a magnification of between 20000 and 20240 so that the sheet structure and habit could be viewed. Grim and Güven (1978) describe several types of smectite aggregates which can be observed within a sample. They include:

1) Globular aggregates which consist of tiny, randomly arranged globules.
2) Mossy aggregates which comprise of crystals resembling flexible fibers which seem to form by the curling of thin layers.
3) Lamellar aggregates can be further divided into:
    a) Foliated aggregates which consist of individual folded and crumbled sheets loosely packed together.
    b) Compact lamellar aggregates consist of individual subhedral lamellae bound together to form a compact aggregate.
    c) Reticulated aggregates consist of hexagonal shaped platelets.

As mentioned by Reschke and Haug (1991) and encountered by the author, the interpretation of the aggregates and crystalline structure within a particle are highly subjective. The analysis is also limited to a minute representation of the greater sample. But even with these limitations, the variations in the crystalline structure quality of the bentonites analyzed is noticeable.

The three samples analyzed are shown in Photos 4 to 6 and are interpreted as follows:

- Koppies (Photo 4): At a magnification of 20240 the 'pack of cards' structure of this bentonite is clear. The aggregate structure is of the compact lamellar type with a small amount of foliated aggregates visible. A planar habit is clear within the structure indicating a layer-after-layer growth of the crystal mechanism which according to Grim and Güven (1978) generally occurs at low crystallization rates.
- Plett (Photo 5): The first two samples of Plett bentonite were analyzed at 20000 magnification and varied considerably from each other. The first showed a very compact structure whereas the second sample showed a highly foliated structure. These Photo's are shown in Appendix B. A third sample was analyzed at a lower magnification of 10000 and was found to possess both of these forms. The photo of this sample is presented. What is clearly evident in this sample is the contrast between a compact lamellae structure and a foliated structure. The foliated structure is representative of the hydrated form. In other words, the sheets have lost their surface charges which normally hold them together and when water is introduced, these areas of the particle will not swell. The lamellar aggregates which do exist are very compact and represent a slow crystallization rate common to a layer-after-layer crystal growth.
- Culbond (Photo 6): This sample was analyzed at a magnification of 20000 and in general, the particle consisted of compact lamellae although its structure is not as clear as the other samples analyzed. Two photographs were taken of the particle. The first, which is presented here was taken towards the center of the particle and shows a mixture of compact, slightly

foliated, and granular lamellae. The second photo (shown in Appendix B) was taken on the edge of the particle in order to obtain a better view of the 'pack of card' structure. This confirmed that the majority of the particle consisted of compact lamellar aggregates with a small amount of foliated lamellae present. A striking difference between this sample and the other two bentonite samples analyzed, are the granular particles scattered across the surface. Exactly what these particles are is uncertain, but due to the way they are clinging to the main structure is an indication that they possess surface charges and could in all probability be small particles of bentonite and not impurities.

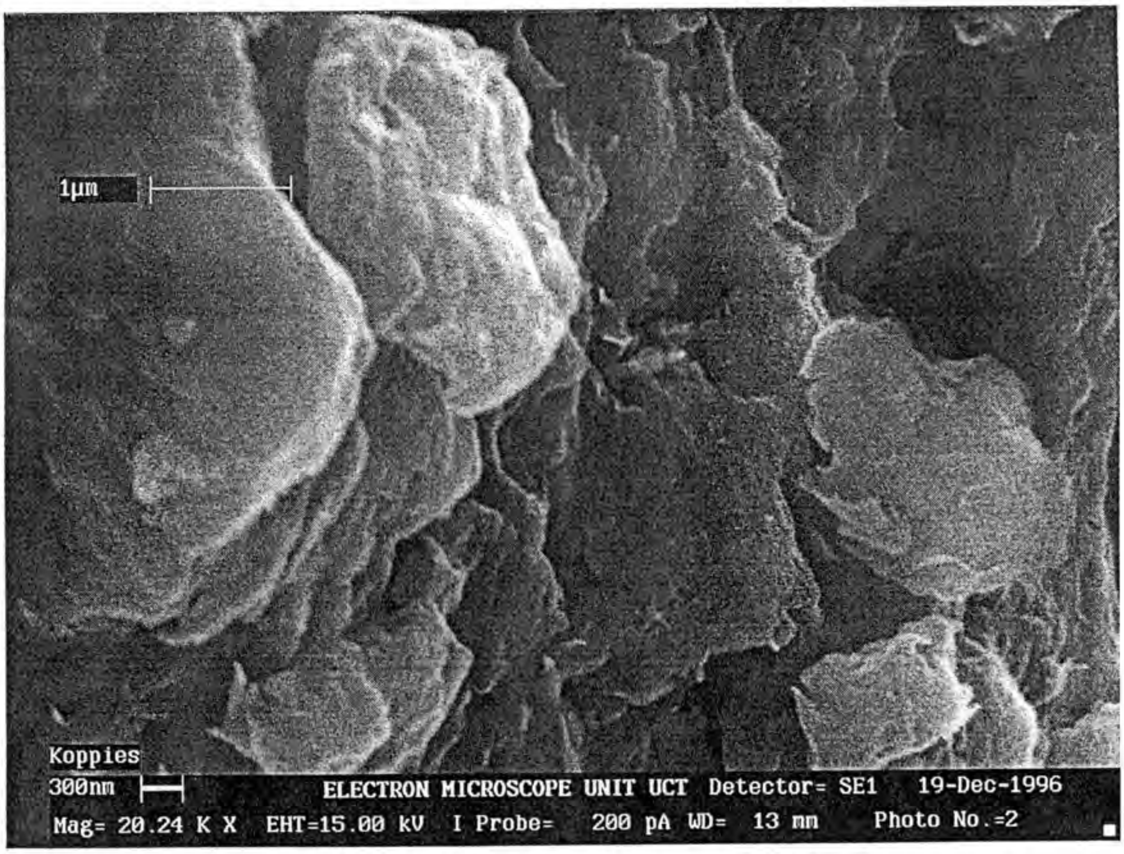


Photo 4 : Koppies bentonite at a magnification of 20240.

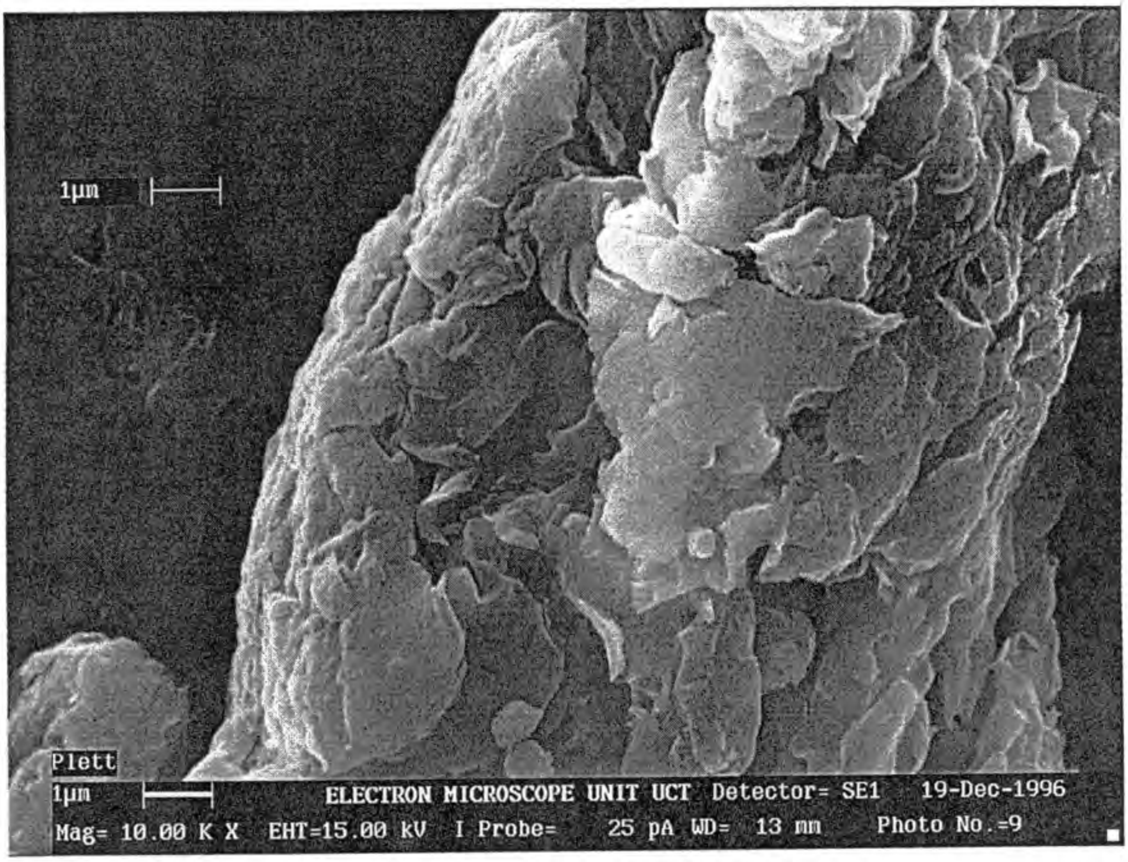


Photo 5 : Plett bentonite at a magnification of 10000.

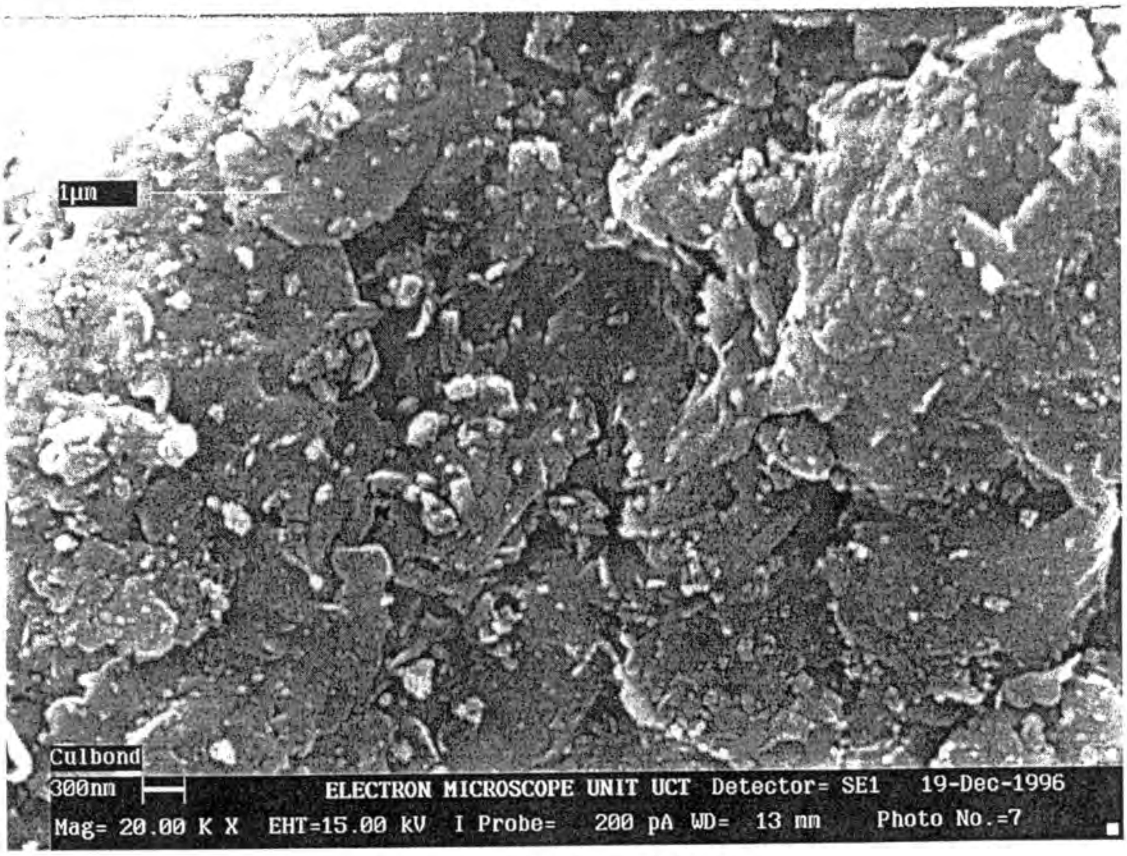


Photo 6 : Culbond bentonite at a magnification of 20000.

Section B Laboratory Testing :Method and Results.

4. Overview

This section concentrates on the laboratory testing of selected South African bentonites done in the Geotechnical Laboratory of the Department of Civil Engineering at the University of Cape Town. In general, information on the engineering aspects of bentonite is very limited, and with regard to the South African bentonites the information is almost none existent. The aim of this laboratory work was to provide the present and future users of South African bentonite with some basic, comparable engineering information. Wherever possible, standard geotechnical laboratory equipment has been used for the tests. Modifications have been limited and new test apparatus were built as economically as possibly using readily available materials.

In order to understand more of the engineering aspects of the South African bentonites, the specific gravity test and Atterberg Limits were performed on Koppies, Plett, Culbond and Culseal. After the presentation of these tests, a series of swell tests on the selected bentonites under a variety of conditions is presented. A new approach of providing an index for swelling is included in this section.

The main emphasis of the work is on the permeability of pure bentonite and bentonite/soil mixes. Permeability tests were performed in a number of ways using standard or slightly modified laboratory equipment. In addition to this, a new cost effective way of measuring the permeability of pure bentonite is also presented.

5. Specific Gravity and Atterberg Limits

The specific gravity and Atterberg Limits are vital to the engineer not only for calculating the basic physical geotechnical properties but also to provide invaluable information in understanding the way the material will act when used in the field.

5.1. Specific Gravity of Bentonite

5.1.1. Test method and apparatus

The Density Bottle Method was used for the calculation of the specific gravity (S.G.). The test method employed is presented in BS 1377:1975, test 6. For this particular test series , 50ml density bottles were used with de-aired tap water. An analysis of the tap water with regard to chemical composition water is shown in Appendix C.

Although a 5g sample is recommended when performing the S.G. test on fine grained material, it was found that due to the high swelling and impermeability characteristics of bentonite, thorough mixing of the material with water was virtually impossible. Results were eventually achieved with samples weighing between 1g and 2g.

In brief, the test procedure was performed as follows: the samples were oven-dried for a 24 hour period before being weighed on an electronic scale to an accuracy of 0.001g. They were then transferred into the density bottles, filled to approximately a third with de-aired water and then placed under vacuum for a 24 hour period. In some cases the samples needed to be shaken and placed under a vacuum for a second time to achieve full saturation. The density bottle was filled with de-aired water and weighed, a temperature correction applied and the specific gravity calculated.

5.1.2. Test results

Calculating the specific gravity of pure bentonite using the Density Bottle Method is not as simple as for a silt or sand. Full saturation of the material is essential in obtaining accurate S.G. results. In a number of occasions air trapped in the bentonite sample would cause the sample to 'pop' when placed under vacuum resulting in spillage and losses in bentonite mass. The final values obtained were the result of numerous attempts and re-tests.

A minimum of 3 error free tests were performed on Koppies, Plett, Culseal and Culbond bentonite. A summary of the average results of the tests are shown in Table 1.

BENTONITE SAMPLE	S.G.
Koppies	2.83
Plett	2.58
Culseal	2.58
Culbond	2.61

Table 1 : Results of specific gravity tests of the selected bentonites

5.1.3. Discussion and analysis of results

The specific gravity of clays typically range between 2.68 and 2.72 according to Head (1982). Bentonite seems to be an exception. The results obtained for the South African bentonites vary considerably from 2.58 to 2.83. The results are not uncommon when compared to those used by a number of other authors. For example: Chapuis (1990) obtained an S.G. value of 2.55; Komine and Ogata (1996) obtained 2.79; Mollin, Stewart and Cousens (1996) obtained 2.76; Sivapullaiah et. al. (1987) obtained 2.87; Kenny et. Al. (1992) obtained 2.74. The specific gravity results obtained for Culseal (S.G.=2.58) and Culbond (S.G.=2.61) are not in agreement with the results presented in the data sheet provided by SAMREC (1997) which assigns the S.G. value for Culseal as

2.34 and Culbond as 2.51. The data sheet results are questionable due to the two types of bentonite essentially coming from the same source and should therefore have similar S.G. values. The results obtained in this work differ by only 0.03 and can therefore be regarded as more precise. The exceptionally high S.G. value obtained for Koppies bentonite could be attributed to the high iron content particularly prevalent in the chemical analysis provided by Grim and Güven (1978).

5.2. Specific Gravity of Soil/Bentonite mixes

5.2.1. Test method and apparatus

The test procedure used to determine the specific gravity for soil/bentonite mixes was basically the same procedure used in the pure bentonite test. The soil greatly aided the dispersion of the bentonite in the water and allowed for the use of a 3g sample as opposed to the 1g to 2g sample used in the pure bentonite test.

The mixes were prepared with 8%, 10%, 12%, and 14% by mass of bentonite mixed with Cape Flats sand at a moisture content of 10%. The samples were compacted to approximately 1950 kg/m^3 into a compaction mold and hydrated for one month before testing commenced.

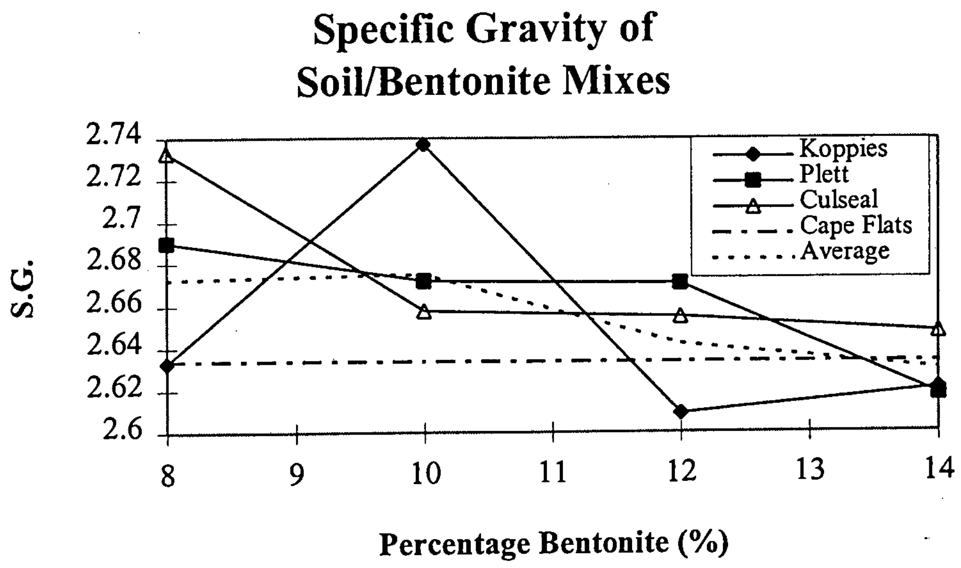
5.2.2. Test results

The problems encountered with the specific gravity testing of pure bentonite were not experienced in the tests conducted on the bentonite soil mixtures and generally only three tests were performed on each sample.

Specific gravity tests were performed on mixtures of Koppies, Plett, Culseal and on the mixing sand, Cape Flats sand. A summary of the results are shown in Table 2. Graph 2 depicts the S.G. results obtained for the mixes in relation to the S.G. of the mixing sand.

SAMPLE	S.G.	AVERAGE
8% Koppies and sand	2.633	2.685
8% Plett and sand	2.690	
8% Culseal and sand	2.733	
10% Koppies and sand	2.737	2.689
10% Plett and sand	2.672	
10% Culseal and sand	2.658	
12% Koppies and sand	2.609	2.645
12% Plett and sand	2.671	
12% Culseal and sand	2.655	
14% Koppies and sand	2.621	2.629
14% Plett and sand	2.618	
14% Culseal and sand	2.648	
Cape Flats Sand	2.634	

Table 2 : Specific Gravity of the selected bentonites mixed with Cape Flats sand and the specific gravity of Cape Flats sand.



Graph 2 : Specific Gravity of the selected bentonites and Cape Flats sand

5.2.3. Discussion and analysis of results

The specific gravity tests conducted on soil/bentonite mixes initially seemed easier to perform due to the sand aiding the dispersion of the bentonite. Although the results are spread over a small range varying by only 0.13, erratic variations in S.G. occurred with the changes of bentonite content. This is visible in Graph 2 and particularly clear in the tests on Koppies. Plett and Culseal however followed a trend of decreasing S.G. with an increase in the percentage of bentonite. In the majority of the cases, the S.G. of the soil/bentonite mix is higher than the S.G. of the mixing sand, which was not expected for the Plett and Culseal mixes due to these bentonites having a lower S.G. than the mixing sand.

The erratic results could be attributed to the sample size and test method. Although the bulk samples were mixed with a mechanical mixer, it is likely that the bentonite was not evenly distributed within the mixture. The test sample had a mass of only 3g and may not have been a representative mixture and thus would have caused inconsistencies. Four tests were conducted on the mixing sand and these results varied by only 0.02. This is a good indication that the test method and procedure were accurate.

Another possible reason for the variations in S.G. could be the result of air trapped within the bentonite or between the bentonite and sand. This would exasperate the decrease in S.G. with an increase in bentonite content due to the amount of trapped air increasing with the increased bentonite content. This would also help to explain the erratic results obtained with Koppies bentonite which is the most cohesive and was the most difficult specimen to prepare.

No results of the specific gravity of soil/bentonite mixtures of any form were found in the available literature with which to compare the results presented here.

5.3. Atterberg Limits

The Atterberg limits are values derived for the purpose of comparing the effect of the amount of water in a fine grained soil on the engineering behavior of that soil. The values are therefore regarded as the water contents at which certain limiting or critical stages in soil behavior occur. The Atterberg limits relevant to geotechnical engineering comprise of the liquid limit (LL), plastic limit (PL) and linear shrinkage (SL). The values of the plastic limit are subtracted from the values of the liquid limit to provide a value termed the plasticity index (PI), which is used in the classification of a soil and correlating the engineering behavior of the soil.

5.3.1. Test method and apparatus

Standard soils laboratory tests were used to obtain the Atterberg limits. Two tests methods were used to calculate the liquid limit. The first method used the Casagrande device and the second method used the 30° cone penetrometer or 'fall cone' apparatus.

The plastic limit test is a more arbitrary test and is defined as the water content of a soil sample at which a thread of soil 'just' crumbles when it is carefully rolled out to a diameter of 3mm. It should break up into threads of between 3mm and 10mm long.

The Casagrande method and plastic limit is covered in detail in TMH 1 (1986) Ref. A2 and A3. The penetrometer method is described by Craig (1987) and is fully detailed in BS 1377.

Linear shrinkage tests were attempted using the standard methods but it was found that the degree of shrinkage of the bentonite samples was so extensive in both the length and breadth that no readings could be obtained.

5.3.2. Test results

Liquid Limit

Liquid limit tests were first conducted using the Casagrande device on Koppies, Plett and Culbond bentonites. The tests were repeated on Koppies and Plett bentonites using the 'fall cone' LL test method. The results of the LL tests done in the two different ways are presented in Table 3. The PL tests conducted in conjunction with the 'Casagrande' LL tests referred to as Test 1, and the PL tests done in conjunction with the 'fall cone' LL tests are referred to as Test 2. The results of the PL tests are presented in Table 4.

The plastic index of the selected bentonites is calculated using the formula :

$$LL\% - PL\% = PI\%$$

The PI results are presented in Table 5.

LIQUID LIMIT (%)			
Test Method	Bentonite Types		
	Koppies	Plett	Culbond
Casagrande	468	366	147
Fall Cone	480	217	

Table 3 : Liquid limit results of the selected bentonites

PLASTIC LIMIT (%)			
Test Number	Bentonite Types		
	Koppies	Plett	Culbond
1	61	46	47
2	74	53	

Table 4 : Plastic limit results of the selected bentonites

PLASTIC INDEX (%)			
Test Method	Bentonite Types		
	Koppies	Plett	Culbond
Casagrande	407	320	100
Fall Cone	406	164	

Table 5 : Plastic Index of the selected bentonites.

5.3.3. Discussion and analysis of results

The liquid limit results obtained using the 'fall cone' method were consistent during testing and plotted almost linearly relationship between the number of blows and the water content from a penetration of 14.5mm to 24mm in both tests. The same can be said for the Casagrande method. There is therefore uncertainty as to which method is more suitable for testing bentonites. Due to the thixotropic properties of bentonite, it is suggested that the Casagrande device would be more accurate due to the continuous disturbance of the sample which would prevent the gelling of the bentonite. The 'fall cone' method disturbs the sample to a lesser extent and the thixotropic properties could effect the sample in an inconsistent manner.

Reschke and Haug (1991) showed that the Atterberg limits were able to distinguish the sodium bentonite from the calcium bentonites but the results did not clearly show difference in bentonite quality. They noted that the LL of a high quality sodium bentonite is between 600% and 800% whereas a good sodium activated bentonite will have a LL approaching 450%. In all the bentonite tests, the calcium bentonite had the highest PL at 49% and the lowest LL of 129%. Sivapullaiah et al.(1987) presented the following comparison of the liquid limit of a bentonite to its swelling potential:

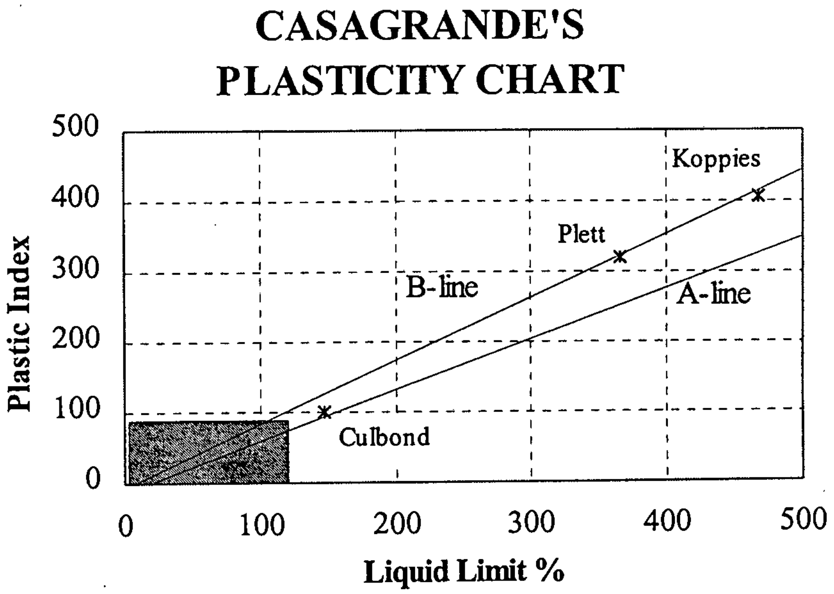
LL below 200% - negligible swell potential
LL between 200% and 400% - moderate swell potential
LL between 400% and 550% - high swell potential
LL values above 550% - very high swell potential.

The results of the Atterberg limits conducted on Koppies, Plett and Culbond bentonites do not match these trends with confidence. The LL of Culbond, a sodium bentonite, is well below the expected value of at least 500% and its PL is also at a level expected for a calcium bentonite. The LL of Koppies matches the value of Reschke and Haug (1991) for a good sodium activated bentonite, although its PL is well above the predicted value and should be in the order of

30 to 35. The LL of Plett, a calcium bentonite, appears to be over double the expected value whereas its PL is very close to the fore mentioned value.

As presented in the next section on 'Swell Testing', Culbond has similar swelling properties to Koppies under free swell conditions and similar properties to Plett under restricted swell conditions. The comparison of Sivapullaiah et al. (1987) would rate this material's swell as negligible where its swell potential is significant. Koppies and Plett, both calcium bentonites, match the findings of Sivapullaiah et al.(1978), although the swell potential of Plett under certain circumstances is regarded as negligible.

For the purpose of fine grained soil classification, the PI and LL are plotted on the Casagrande Plasticity Chart which allows the soil to be grouped and identified by descriptive letters. (e.g. CH = inorganic clays of high plasticity). A silt will usually plot below the A-line and a clay will plot above the A-line. The B-line is the upper limit line for all soils. Some highly active clays like bentonite will plot high above the A-line close to the B-line. Casagrande has shown that soils plotting close to the B-line with a LL above approximately 22%, are Montmorillonites (Holtz and Kovacs, 1981). The results of Koppies, Plett and Culbond plotted on the Casagrande Plasticity Chart are shown in Graph 3 to point out the exceptional characteristics of the bentonites. (The shaded area is the standard chart used in geotechnical engineering.)



Graph 3 : The location of the selected bentonites on the Casagrande Plasticity Chart.

6. Swell Testing

One of bentonites most valued characteristic is its high swelling ability. As mentioned in Section A, when bentonite is brought into contact with water, the water is drawn onto the surface of the bentonite sheets pushing the layers apart causing a swell of up to 15 times its dry volume. The high swelling of the material is one of the properties contributing to its very low permeability. Although the total swell is not directly related to permeability, the rate of swell and the final swell volume can be used as an indication of the bentonites quality with regard to permeability. A new way of presenting the total swell of bentonite is introduced, called the 'Zero Thickness Swell Index' and the effect of the swell rate on permeability is also explained.

When subjected to a surcharge, the total swell decreases tremendously. The reduction in swell will occur under an applied load or under a hydrostatic pressure. The effects of these are monitored in the consolidometer permeameter and the Mini Permeameter respectively. The results obtained on all four of the selected South African bentonites are presented.

6.1. Free Swell Testing

At the start of this research on South African bentonites in 1994, no set standard for free swell testing was found in the available literature. A decision was made to attempt to test the swell using readily available equipment and to develop a comparative standard test method. Since then, more information has become available and it was found that there exist a number of swell tests. The results obtained with the various methods all differ and no set standard exists (Sivapullaiah et. Al., 1987).

The method presented here offers a non-dimensional, simple test and is referred to as the 'Zero Thickness Swell Index'.

6.1.1. Test method and apparatus

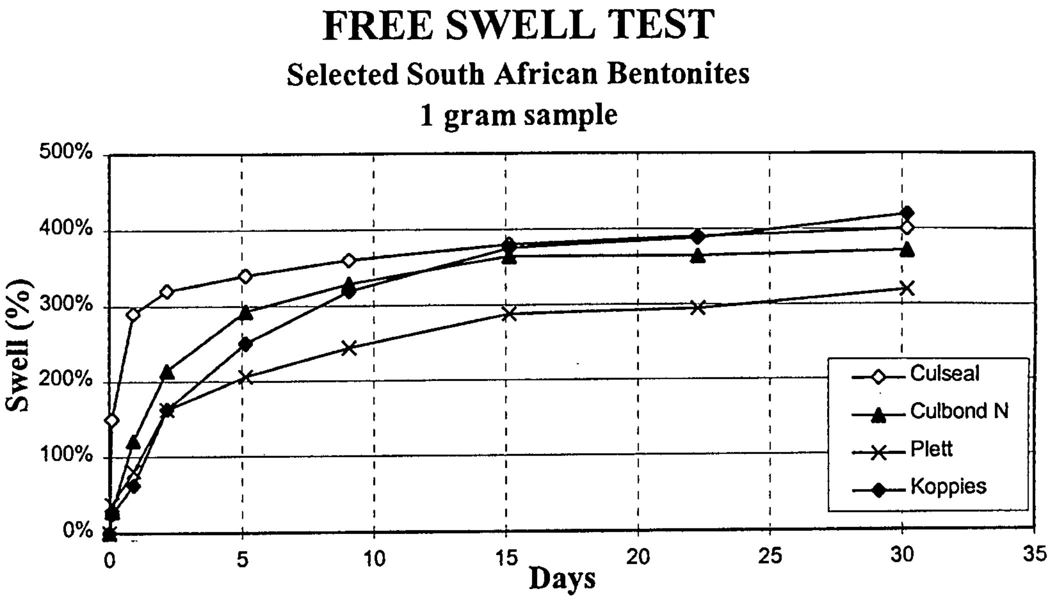
The apparatus used in these tests consists of standard test tubes with an internal diameter of 10mm. In the initial tests, the bentonite sample was placed in the test tube which was then filled with water and the swell was monitored. The problem encountered here was that the air within the bentonite would become trapped and would result in uplift and unrealistic swell results. This was solved by making a small hole of about 1mm diameter in the bottom of the test tube to allow the air to escape when the water permeated in from the top. The method was further improved by the placing of a piece if geotextile in the hole and filling the tube with a 10mm layer of fine sand. The sand not only acted as a filter layer, but also provided a level datum on either side of the tube from which the swell could be monitored (and also lifted the bentonite sample out of the curved base of the test tube).

The test procedure consists of placing a layer of bentonite of between 5mm and 25mm thick (mass = 0.5g to 3.0g) into the test tube above the filter sand. A height reading is taken and the remainder of the test tube is slowly filled with de-aired water and the level is maintained throughout the test. The swell is monitored with time and the test is terminated when the bentonite stops swelling, usually after about 14 days. An example of typical results from a swell test series are shown in Graph 4.

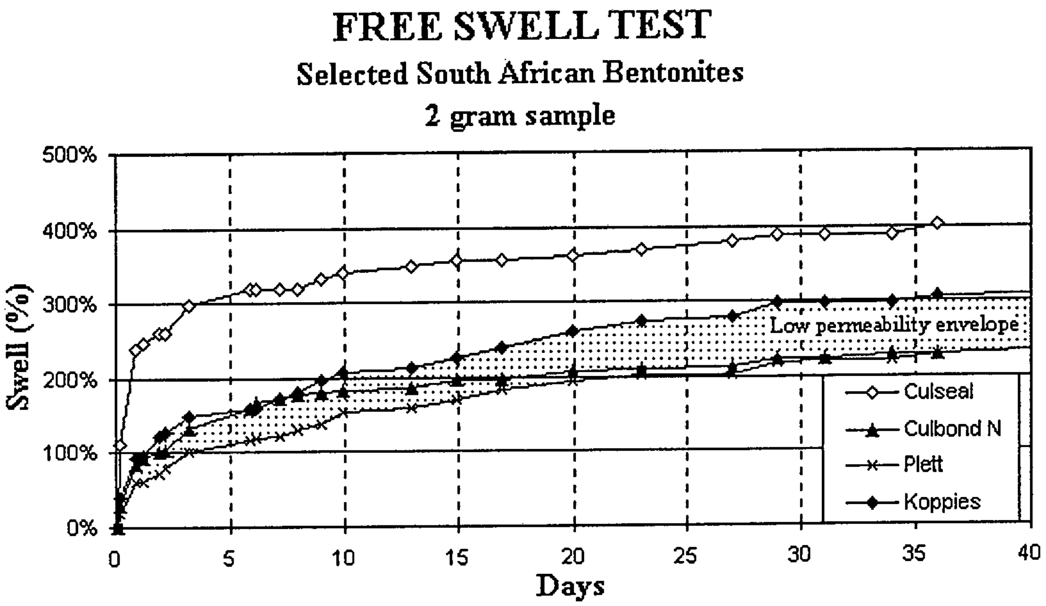
6.1.2. Test results.

The results of the tests conducted on 1g, 2g, and 3g samples of Culbond, Culseal, Plett and Koppies bentonite are presented in terms of swell versus time in Graphs 4 to 6. These results are the average results of a minimum of two tests. Full details of these results and the results of the 0.5g tests are presented in Appendix D.

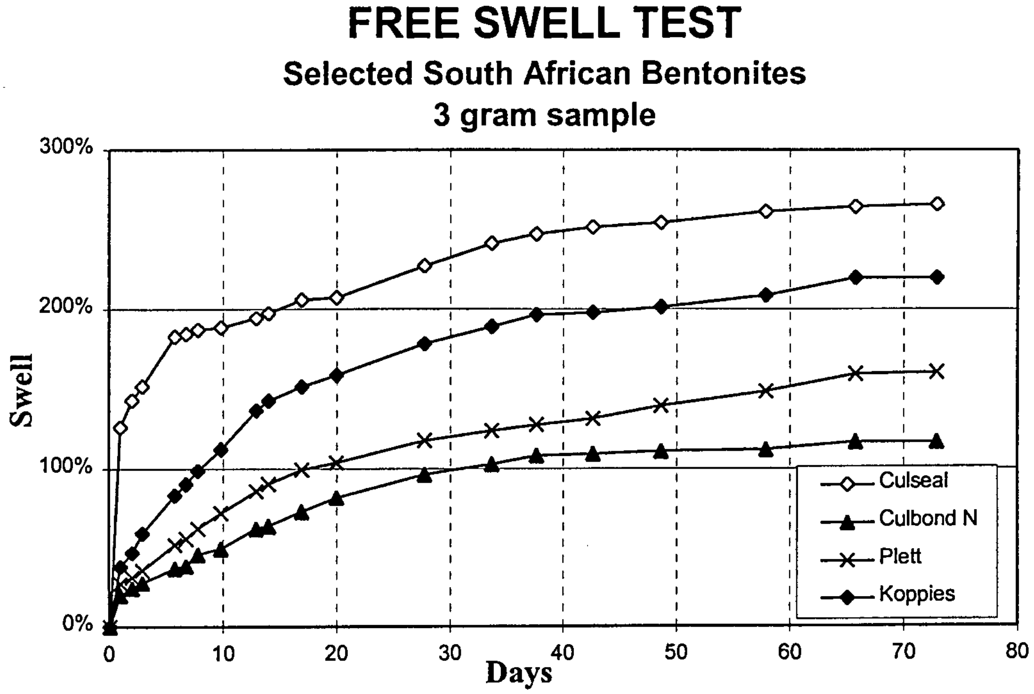
Note: the percentage swell is the ratio of the swell height to the initial height.



Graph 4 : Free Swell Test results of the selected bentonite samples weighing 1gram each.



Graph 5 : Free Swell Test results of four South African bentonite samples weighing 2 grams each



Graph 6 : Free Swell Test results of the selected bentonite samples weighing 3 grams each

6.1.3. Discussion and analysis of results

It was found that in all the samples the percentage swell decreased with an increase in sample thickness. In general, Culseal produced the highest swell which could be expected due to it being a sodium bentonite and of bigger particle size. Being more coarse grained allows the water to flow easier through the sample before swelling commences and the flow is impeded. Thus a large amount of free water has access to the sample for hydration. In the bentonites with a finer particle size, swelling of the upper layers occurs more rapidly which effectively limits the water penetration to the lower layers. Only after the upper layers have absorbed all the water they require for hydration does the water permeate further into the sample to hydrate the lower layers. This could take weeks as in the case of the 3g samples where swelling ceased only after 2 months.

The permeability of the bentonite will have an effect on the rate at which the water reaches the lower layers of the sample and a bentonite with a low permeability should therefore provide a more constant increase in swell. A bentonite with a low overall swelling capacity is regarded as inferior (see Section A) since it has a higher permeability. Considering these two criteria, it is possible to establish a trend line or envelope within which a bentonite with low permeability would lie. Full details are provided in a later section on the permeability of the bentonites presented here, but, in general, Koppies has the lowest permeability with Culbond and Plett only slightly higher and Culseal in some instances one order higher. For this series of selected South African bentonites, a 'low permeability' envelope is plotted on Graph 5. By simply testing the swell of a bentonite sample of the same mass, an indication of the quality of the bentonite with regard to permeability can be achieved.

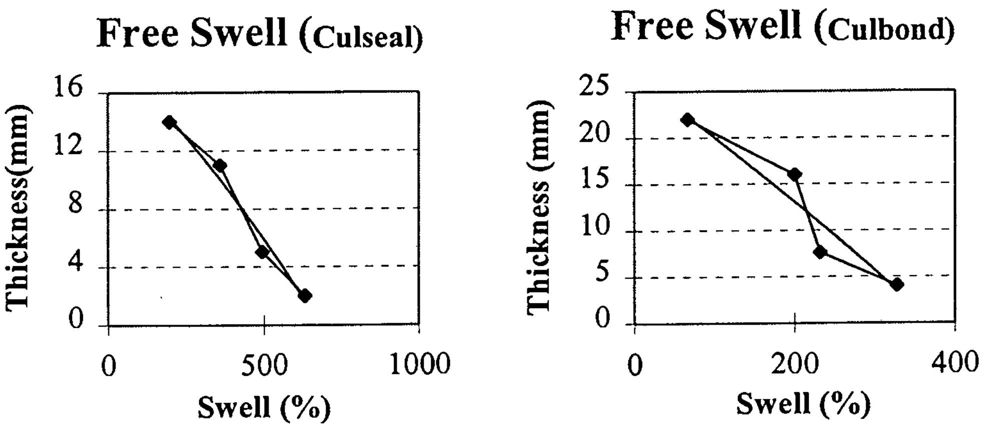
6.1.4. Zero Thickness Swell Index

There have been a number of ways presented in the past to measure swell and to provide an index to represent the swell ability Sivapullaiah et. al. (1987) present various swell tests and show the differences that exist between them. The swell tests presented are : Percentage Free Swell, Differential Free Swell and the Free Swell Index. The results of all the tests differ considerable and are not comparable. They also presented a new nondimensional method, the Modified Free Swell Index test which removes the inherent limitations of the Free Swell Test. This test still relies on mixing a small amount of the material (3g) in a large amount of water (100ml) and allowing it to settle for 24 hours before measuring the swell volume. Although this may work for most materials, some bentonites are highly dispersive in water. Even after a 24 hour period an accurate change in volume due to swell is not measurable, as experienced by Reschke and Haug (1991). A different approach is therefore required to test bentonite.

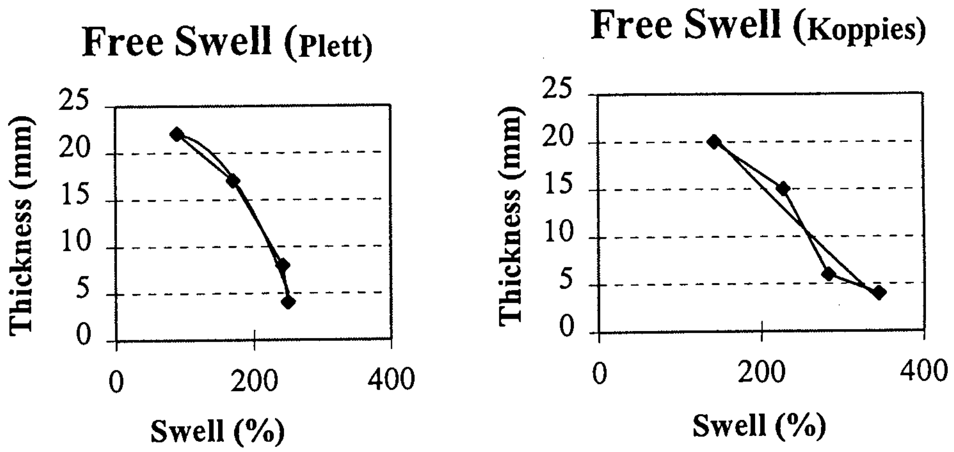
From Graphs 4 to 6 it is visible that when a material is allowed to swell upwards only, the pre-swelling thickness affects the final total swell. In other words, the vertical stress applied onto the lower layers by the upper layers reduces the swell potential of the lower layers. It is therefore necessary to use an infinitely thin layer to obtain a free swelling layer with no restricting stress. Although this is physically not possible, it can theoretically be achieved in the following manner. If the results of the swell tests at 14 days are plotted against the original thickness, an average line matched to a second order polynomial can by drawn through the points. This is shown on Graphs 7 to 10. If this line is then extended to the base line where the thickness equals zero, a swell percentage can be read off (or calculated using the polynomial formula), which will be defined as the 'Zero Thickness Swell Index ' for that sample. For the South African bentonites tested, the Zero Thickness Swell Indices obtained are shown in Table 6.

Zero Thickness Free Swell Index	
Sample	Index (%)
Culseal	681
Culbond	378
Koppies	378
Plett	273

Table 6 : Zero Thickness Swell Index for the selected bentonites



Graph 7 and 8 : Percentage swell versus initial sample.



Graph 9 and 10 : Percentage swell versus initial sample thickness.

6.2. Swell Under Pressure

It was noted in the previous section on Free Swell that as the thickness of the bentonite sample increased the full swell potential was not achieved. This was the result of the gravitational stresses induced onto the lower layers by the upper layers. The effect of a hydrostatic pressure and even the effect of a small applied load on a layer of bentonite greatly effects the final swollen thickness of the material. As the applied load increases it would be expected that the swell would decrease. This was not entirely correct when subjected to larger stresses in the consolidometer where the change in thickness due to the swell tended to remain the same.

The effect of hydrostatic pressure and small applied loads on bentonite which are presented first, were measured using a 'Mini Permeameter', an apparatus developed in the Geotechnical Laboratory at UCT. Secondly, the results obtained from the tests conducted in a consolidometer permeameter are presented.

6.2.1. Mini Permeameter

6.2.1.1. Test method and apparatus

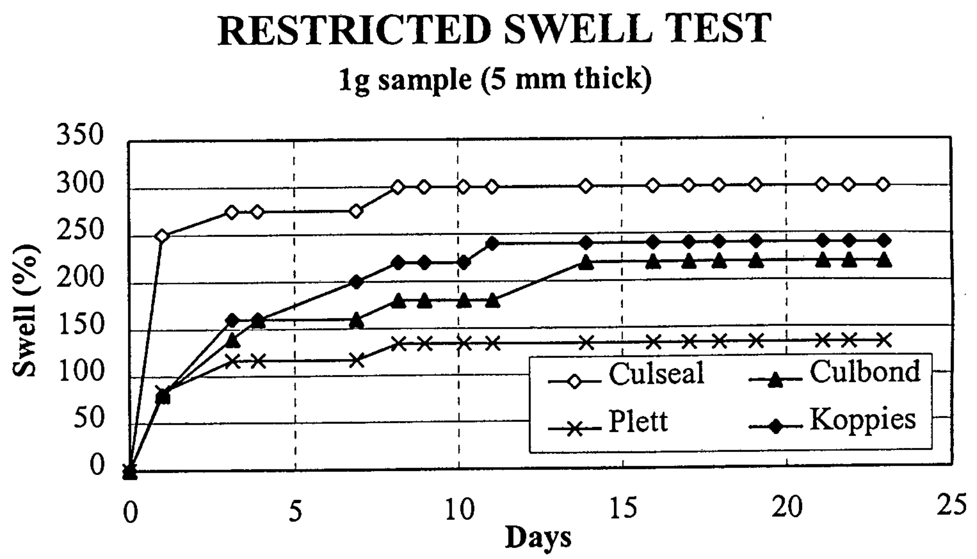
The test apparatus is explained in detail in Sub-section 8.4 but, in brief, consists of a 16mm Ø glass tube 100mm long with a base filter system and a top inlet connected to a constant head supply. The filter system consists of a layer of fine sand and a layer of filter paper separating the sand from the bentonite.

The sample is prepared by placing an even layer of dry bentonite onto the filter system. A light surcharge can be applied by placing a layer of filter sand separated by filter paper onto the bentonite layer. The remainder of the tube is carefully filled with de-aired tap water. The top cap is then connected and a constant head of hydraulic pressure applied. At the time of testing, the available head was 2.15m.

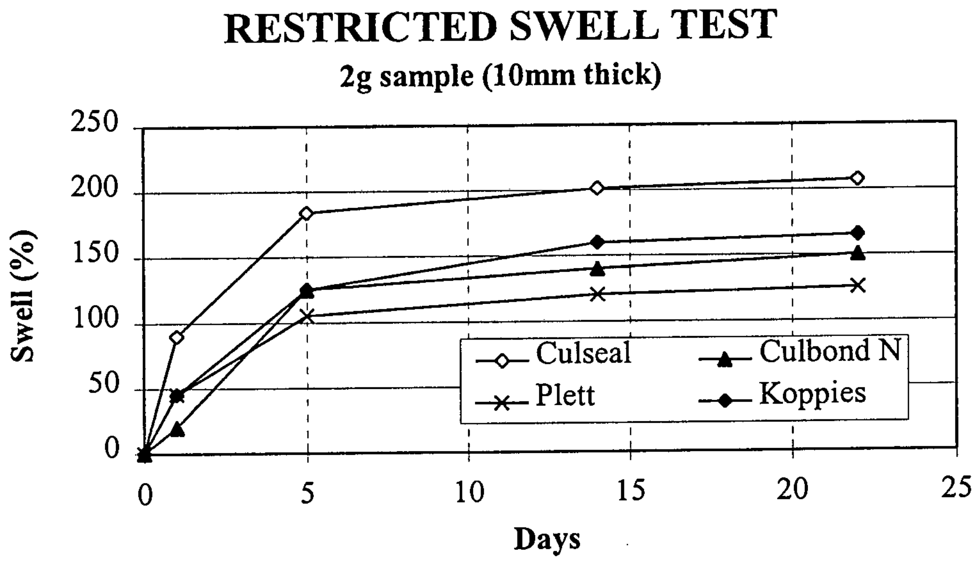
The Mini Permeameter provided two types of results with regard to the swelling potential of bentonite: firstly, it showed the effects of a hydraulic pressure, and secondly the effect of different surcharges on the bentonite could be monitored.

6.2.1.2. Test results

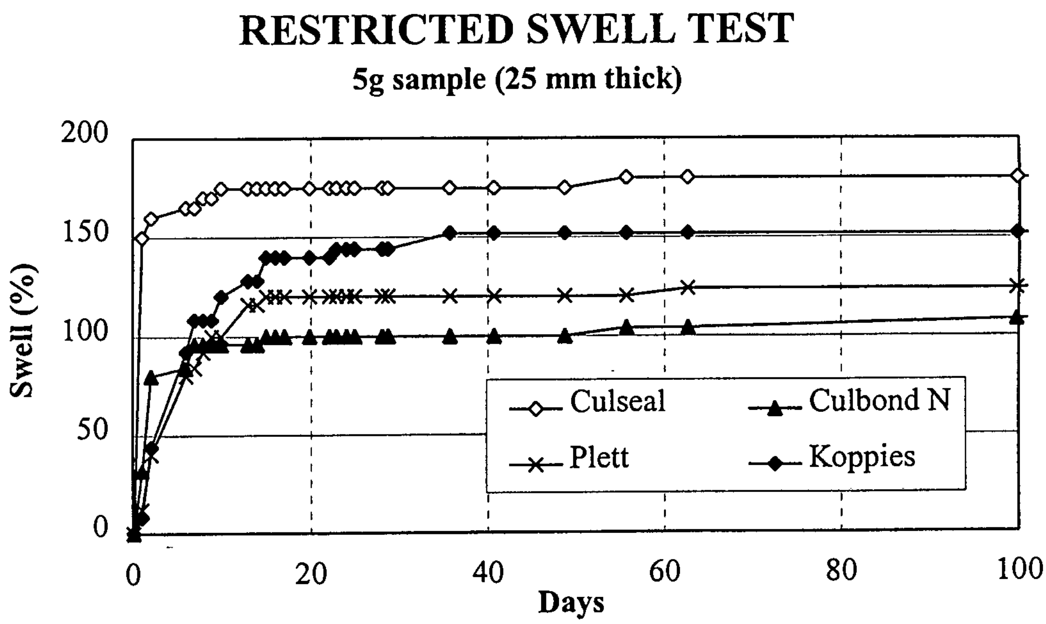
Three sets of samples were tested without a surcharge load but under a hydraulic head of 2.15m. The samples weighed 1g, 2g, and 3g, and provided initial heights similar to the samples used in the Free Swell tests of approximately 5mm, 10mm and 15mm respectively. The results of these tests are shown in Graph 11 to 13. Two sets of samples were tested with an additional surcharge load of 5g of filter sand. The results were similar and an average of the results is presented in Graph 14.



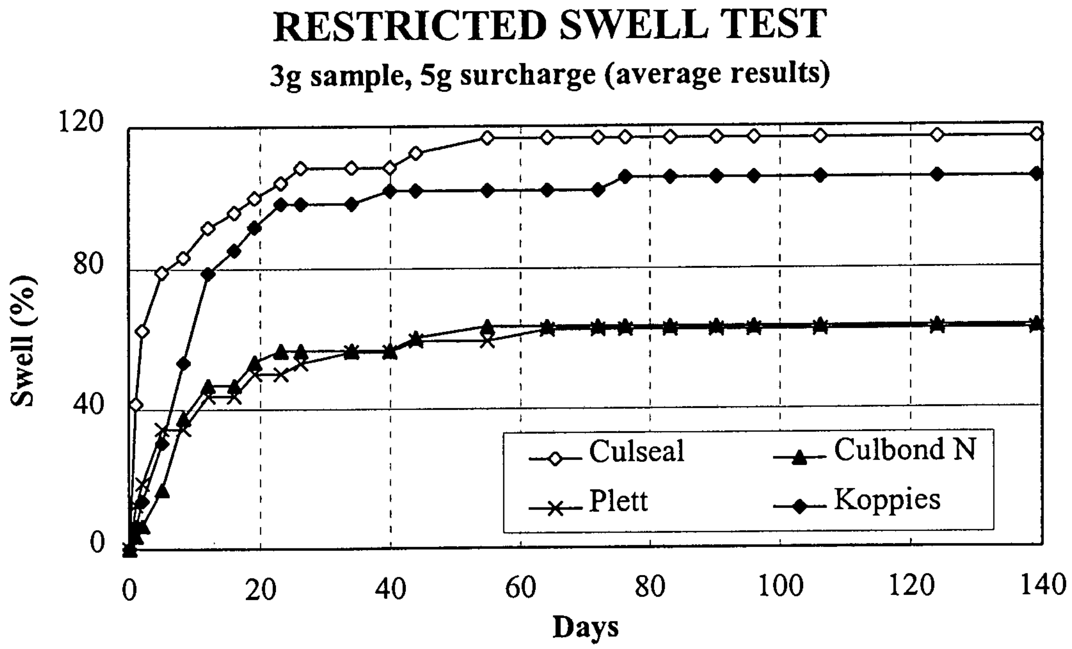
Graph 11 : Restricted Swell test results of 1 g samples of the selected bentonites under an applied hydraulic head of 2.15m



Graph 12 : Restricted Swell test results of 2 g samples of the selected bentonites under an applied hydraulic head of 2.15m



Graph 13 : Restricted Swell test results of 5 g samples of the selected bentonites under an applied hydraulic head of 2.15m



Graph 14 : Restricted Swell test average results of two tests of 3 g samples of the selected bentonites under an applied hydraulic head of 2.15m and a surcharge load of 5g of filter sand.

6.2.1.3.Discussion and analysis of results

Even though the bentonite samples weighed exactly the same, the initial heights of the different bentonites are not exactly the same and the initial sample heights also differ to those achieved in the Free Swell test. But, even with these variances, clear differences can be observed between the results of the Restricted Swell tests with and without a surcharge and the results of the Free Swell tests.

The most noticeable results are shown in Graph 14 where the effect of only a 5g surcharge (25 Pa) reduces the swell by almost one third when compared to a 3g sample without surcharge derived from a linear interpolation of Graph 12 and 13. The effect of a hydraulic head is very noticeable when the results of Graph 11 and 12 are compared to those of Graphs 4 and 5 of the Free Swell tests. Both the 5mm thick and the 10mm thick sets of samples produced final swell heights of about half of those achieved in the Free Swell tests done with a 1g (7mm thick) set and a 2g (12mm thick) set of samples.

The reduction in swell occurring when the thickness of the bentonite layer is increased as experienced in the Free Swell tests, also happened to the samples subjected to a hydraulic head. The effects are clearly visible in Graphs 11 and 12 where the layer thickness was increased from 5mm to 10mm. A reduction in total swell of between 75% and 100% occurred in all of the samples except Plett bentonite which in fact experienced only a 5% decrease. The same trend was experienced when samples of 25mm thickness were tested although the reduction in swell was much smaller ranging between 5% and 50% with Culbond experiencing the largest reduction from 150% to 100%.

6.2.2. Consolidometer Permeameter.

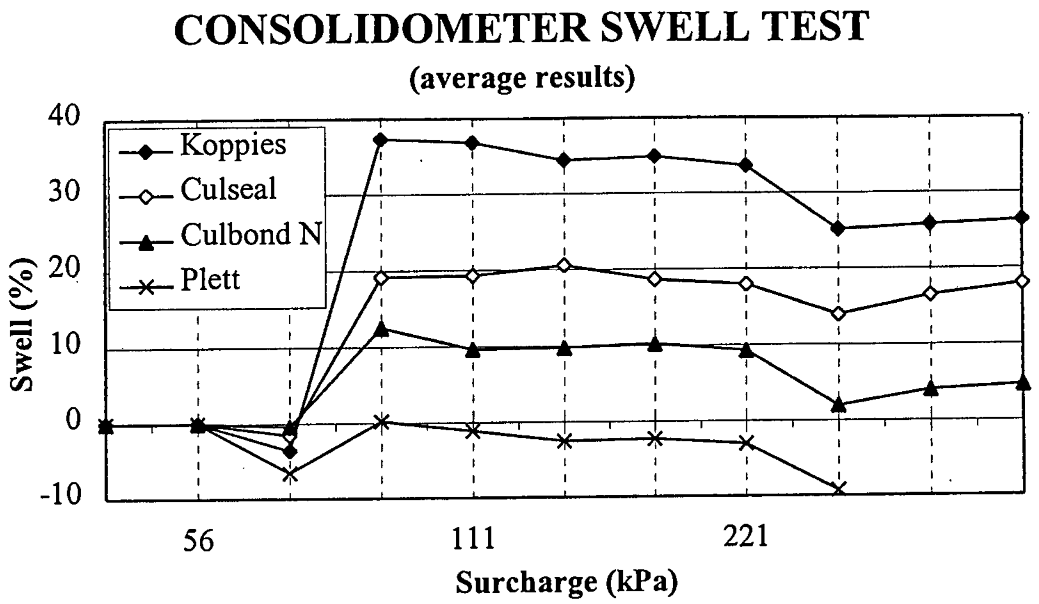
6.2.2.1. *Test method and apparatus*

The Consolidometer Permeameter at the University of Cape Town's Geotechnical Laboratory is a standard Farnell consolidometer with a 79.7mm Ø cutting ring 20mm high. The base is modified to allow water to be permeated upwards through the sample. Full details are presented in sub-section 8.1 on permeability testing. The swell tests were conducted in conjunction with the permeability tests which guaranteed full saturation of the sample and control of possible sidewall leaks. In order to shorted the waiting period before permeability readings could be achieved, a sample thickness of 5mm was used. The remainder of the 20mm high ring was filled with a fine sand and a 5mm ring layer of bentonite was placed between the sand and the ring to prevent sidewall leakage. The preparation of the samples is explained in detail in Sub-section 8.1. Before any swell tests result were calculated, a consolidometer test was first conducted on the sand and the bentonite ring. The swell results were then corrected for the consolidation experienced by the sand and the bentonite ring. The results of the sand/bentonite ring consolidation are shown in Appendix E.

The samples were subjected to three load increments. The first load placed on the loading arm was 2.55kg which provided a surcharge pressure of 55.7kPa on the sample. Permeation of the sample started immediately after the first load was applied. The load was then doubled to 111kPa and the last increment was to 221kPa. The time between the loading intervals varied between two and six days depending on the permeability tests being performed on the samples.
Tests were performed on Culseal, Culbond, Plett and Koppies bentonite.

6.2.2.2.Test results

Two tests were conducted on each of the selected South African bentonite samples. An average of the results of the two tests is presented in Graph 15.



Graph 15 : Average results of the swell tests performed in the Consolidometer Permeameter on the selected bentonites with 3 load increments of 56 kPa, 111kPa and 221kPa.

6.2.2.3.Discussion and analysis of results

The general trend of a decrease in swell with an increase in stress experienced in the Free Swell tests and the Restricted Swell tests occurred in the consolidometer test as well. Although the trend was similar, the effect of the reduction in swell with an increase in stress was much smaller even though the changes in stress were far greater than those experienced in the Free and Restricted swell tests.

Initially, a negative swell occurred in all the samples directly after loading due to the consolidation of the bentonite layer, and swelling only commenced when the sample became saturated. All of the samples except Plett bentonite produced swells in excess of the initial dry bulk thickness, with Koppies bentonite producing the largest swell of 37%.

An interesting phenomenon occurred approximately 24 hours after applying the third load increment of 221kPa where a slight swell recovery was experienced. A reason for this could be the correction factor brought in after each load increment for the sand layer, or that the excess pore water created due to a load increase was forced between the bentonite sheets resulting in additional hydration and therefore an increase in swell. It is also likely that under these stresses, the dry bentonite particles are 'broken open' and the trapped air is forced out allowing water to get in contact with the unhydrated bentonite.

7. Permeability Testing

7.1. Overview of Permeability Testing

Bentonites most valued property in the field of Civil Engineering is its almost impervious nature. This makes it the ideal natural material to use as an impervious liner in landfill sites, oxidation ponds, dam walls or any structure where control over the flow of liquid is required. Although the permeability of bentonite can be detrimentally affected by certain chemicals, with correct pre-testing, design and installation, it offers an environmentally friendly alternative to plastic, concrete, asphalt and other liner systems. The emphasis of this research was therefore aimed at providing comparable permeability test results performed using common laboratory equipment. The tests were all performed with water as the permeant and it is envisaged that the results will form the foundation of future permeability testing of South African bentonites and provide the user with the basic information for liner design.

Bentonite is used to form a liner either in its pure form or mixed with soil. Both applications have their benefits and drawbacks. The pure liner offers the lowest permeability but lacks shear stability and is difficult to install whereas the soil/bentonite mixture is structurally more stable, easier to construct but has a higher permeability and requires stringent control while constructing. Laboratory permeability tests were performed on selected South African bentonites in their pure form and mixed with local soils. Pure bentonite permeability tests were performed in a Consolidometer Permeameter, a Triaxial cell, a standard falling head permeameter and a new device called the Mini Permeameter (see also Section 6.2.1) which was developed to offer an economical alternative for permeability measurement. The permeability of soil/bentonite mixtures was tested in a Consolidometer Permeameter and in a Double-Ring Permeameter. Before the tests on pure bentonite and soil/bentonite mixtures are presented, a short description of the permeating water is given. This is followed by a review of the methods used to test the hydraulic conductivity of soils.

7.2. Permeant Liquid

As previously mentioned, water was used as the permeant in all the tests. The water used was clean tap water which was filtered and de-aired. This was achieved with the aid of the D/CHWSS, a de-airator developed by the UCT Civil Engineering Laboratory Workshop (Hassenpflug, 1995). An analysis of the water before de-airing, after de-airing and after permeating through a pure bentonite sample was done by the Geology Department of UCT and the full results are shown in Appendix C. The concentrations of the components present in the de-aired water sample are shown in Table 7.

The analysis showed no chemical difference between the normal water and the de-aired water which was expected, but of interest is the result of the sample taken after permeating through a layer of pure bentonite. It was noted that the Calcium content in the water reduced from 15ppm to 10ppm (concentrations in parts per million).

Component	Li	Na	NH4	K	Mg	Ca
Concentration (ppm)	0.000	10.721	0.000	0.590	2.166	15.211

Table 7 : Concentrations of the components present in the tap water used as a permeant in all the permeability tests.

7.3. Permeability Testing Apparatus

Liquid flows through soil in two ways, it is either drawn by a pressure difference or by a chemical difference across the sample. As explained in Section A in the 'Permeability Process', the flow of liquid due to a pressure difference is of concern when water is used as a permeant. This flow of water, called seepage, caused by a hydraulic force (gradient) can be measured and the results are referred to as the hydraulic conductivity of the soil. The term 'permeability' or 'coefficient of permeability' which includes both the chemical and hydrostatic forces, is used in this case to represent only the hydraulic forces.

The permeability is measured using a permeameter and can be done in the field or in a laboratory. (The permeameters used in the field rely on the process of infiltration and are generally called infiltrometers). The permeameters used in the laboratory can be divided into two main categories: Fixed-wall permeameters and flexible-wall permeameters. Both systems have their advantages and disadvantages.

7.3.1. Fixed-Wall Permeameters.

Common fixed-wall permeameters include the ASTM standard permeameter cells, compaction mould permeameters, double-ring permeameters and consolidometer permeameters. As the name implies, these permeameters have a rigid side wall. The main advantages of the permeameters is that they are in general simpler and more economical than the flexible-wall permeameters. Certain types also allow samples to be compacted into the permeameter before testing. The main disadvantages are sidewall leakage due to improper contact between the sample and the walls of the permeameter. This problem becomes more evident when samples are permeated with organic chemicals which could cause shrinkage of the sample. Applying back pressure and vacuum saturation is also not advisable in a fixed-wall permeameter due to the possibility of creating sidewall flow paths and cracks within the sample (Daniel et. al., 1985).

To overcome the problem of sidewall leakage, the compaction mould permeameter was modified by dividing the permeant collecting base into an inner ring and an outer ring. This modified permeameter is referred to as the double-ring permeameter. It enables the volume of the inner permeant collected to be compared to that of the outer ring and any discrepancies would indicate the possibility of sidewall leakage.

The possibility of sidewall leakage can also be reduced by applying a load onto the sample which effectively increases the contact pressure between the sample and the permeameter wall. This is achieved in a consolidometer. Permeability is measured by modifying the base of the loading cell to allow liquid to permeate up through the loaded sample under an applied head. The modified system is called a consolidometer permeameter.

The falling head test method and the constant head test method can both be used with all the fixed-wall permeameters. In the case of testing bentonite, the falling test method is recommended due to providing accurate results in a relatively short period of time.

7.3.2. Flexible-Wall Permeameters.

A flexible-wall permeameter is essentially a triaxial cell. The sample is isolated by a flexible membrane and supported by water which is pressurized to provide an equal stress in both the vertical and horizontal direction. Other than providing an in-situ condition, the main advantage of this system is that the possibility of sidewall leakage is greatly reduced. This is due to the flexible membrane being able to accommodate the uneven surface of the soil and if the cell pressure is applied, creates a flawless seal around the sample.

The sample can also be vacuum and/or back pressure saturated and higher hydraulic gradients can be used which assists in permeability results being available after a shorter period. The main disadvantages are that the system is expensive and requires accurate control at all times. The rubber membrane is also susceptible to degradation if chemicals are used as a permeant.

8. Permeability Testing of Pure Bentonite

The coefficient of permeability of bentonite is usually lower than $1.0\text{x}10^{-10}$ m/s and therefore requires specialized apparatus and testing periods lasting several weeks to obtain only a few readings. Therefore, a number of identical apparatus are required so that tests can be run concurrently and a sufficient number of results can be obtained.

The permeability testing of the selected South African bentonites was done using four different methods. The reason for this was threefold, firstly it was done to provide permeability coefficients of South African bentonites, secondly, it provides a comparison between the various test methods, and thirdly, a benchmark could be established to test the accuracy of a new permeameter developed for this purpose. Although not initially planned, it was found during testing that the degree of free swell affects the permeability. No previous research was undertaken on this and no information was found in the available literature. It was therefore decided to investigate the effect of swell on permeability using the standard permeameter.

The first three methods were conducted using standard laboratory equipment and the fourth method was done in an apparatus developed as an economical alternative to the standard methods. The four methods were: the Consolidometer Permeameter, the Triaxial apparatus, the ASTM standard permeameter and the new apparatus called the Mini Permeameter.

8.1. The Consolidometer Permeameter

8.1.1. Test method and apparatus

The consolidation permeameter is in essence a consolidometer or oedometer cell which is modified so as to allow liquid to be permeated through the sample while a vertical stress is applied. This is achieved by sealing off the lower disk area from the upper area of the loading cell. A detail of the modified cell developed in the Geotechnical Laboratory and built in the Civil Engineering Workshop at UCT is shown in Appendix F. The cell is placed in a Farnell Consolidometer with a beam ratio of 11.077. De-aired water is permeated upwards through the sample and the permeability is measured using the falling head test. Three load increments are applied to the sample, namely; 55.70kPa, 110.98kPa and 221kPa. Water is constantly supplied to the base of sample throughout loading and the thickness of the sample in monitored with a dial gauge. Permeability tests are performed on each sample between load increments with at least one permeability test being done per load increment.

In order to shorten the test period and to limit sidewall leakage, a bentonite sample of only 5mm thick was used. The remaining 15mm of the cutting ring was filled with sand and a sealing ring of bentonite was placed between the sand and the sidewall. This is shown in a schematic if Fig. 4. Although a standpipe of only 2.6mm diameter was used in this series of tests, test periods exceeding 48 hours were required to obtain enough drop in the standpipe to achieve accurate readings.

Two samples of each type of bentonite were tested simultaneously under the same load conditions. A 3.4mm ∅ standpipe was used on the one sample and a 2.6mm ∅ standpipe was used on the second sample. The permeability and the consolidation of the two samples was compared under each load increment to check for sidewall leakage or other possible inconsistencies. A test cycle usually lasted at least six days but could take longer to obtain a sufficient drop in the standpipe for permeability measurement.

A data logging system was developed using pressure transducers and an HP computer for monitoring the drop in height of the water in the falling head standpipe. Although this provided a number of accurate readings it became obsolete due to the long time intervals required to have a sufficient drop in the standpipe to obtain an accurate reading, a task best performed manually. For the analysis of the data, a spread sheet was developed on Excel to process all the data relating to consolidation and permeability.

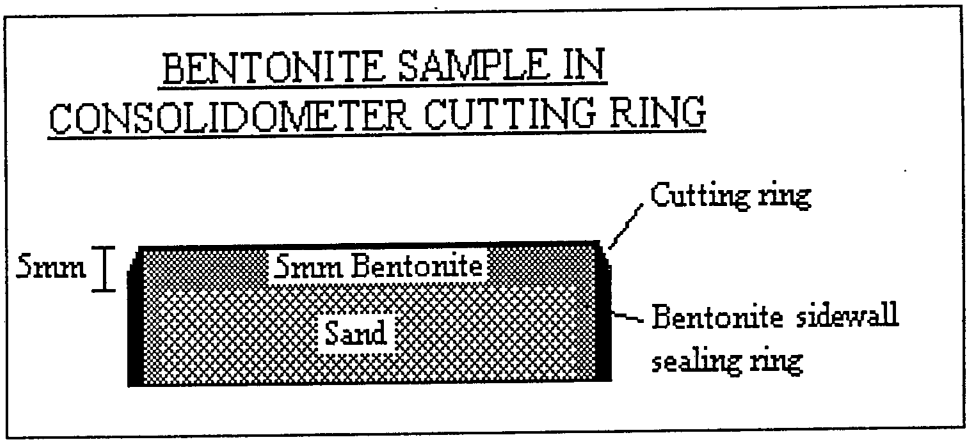
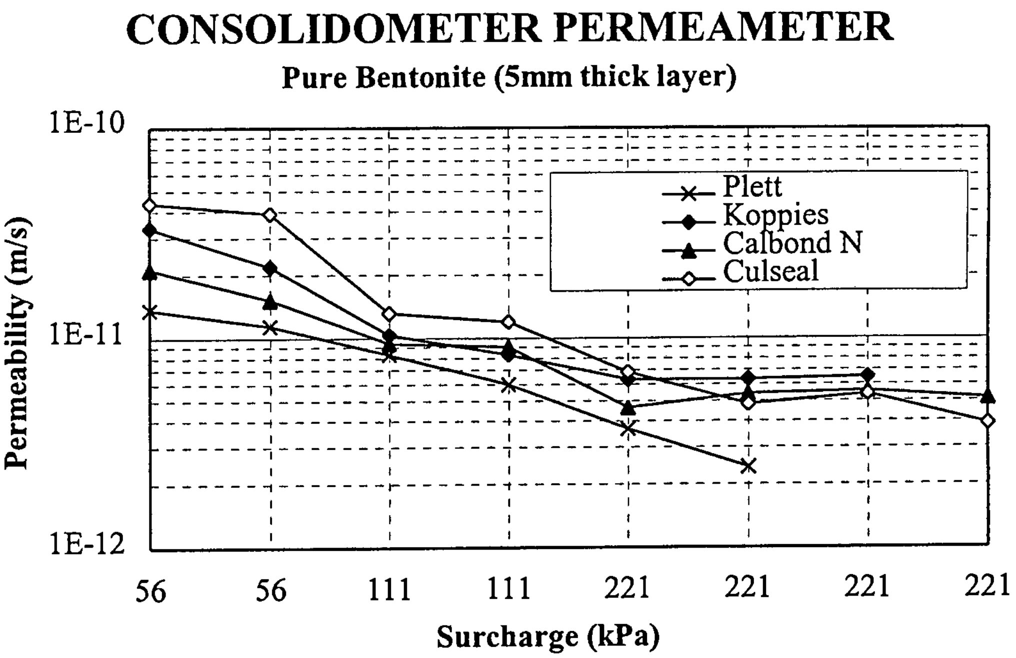


Fig. 4 : Schematic of the 5mm bentonite sample with the filler sand and bentonite sidewall seal prepared in the consolidometer cutting ring

8.1.2. Test results

All four of the selected South African bentonites were tested namely; Koppies, Plett, Culbond and Culseal. Two tests were performed on each type of bentonite and if the results of the two tests compared 'favourably', an average result was taken, otherwise the outlier was disregarded, or if both results seemed unreasonable, the test was redone. A summary of the average results of the tests performed in the consolidometer permeameter are presented in Graph 16 and further details are available in Appendix G.



Graph 16 : Consolidometer permeability results on the selected bentonites under three load increments.

8.1.3. Discussion and analysis of results

In general, the permeability decreased with an increase in vertical stress. Under these load increments the difference in permeability between the different types of bentonite is minimal. For example; the permeability achieved under the 221kPa load increment ranges from about 2.5×10^{-12}m/s to 6.0×10^{-12}m/s. The first permeability test performed after the 56kPa and after the 111 kPa load increment is slightly higher than the second test for all the bentonite types. This pattern changes slightly for the 221kPa surcharge, but the variation in permeability is caused by the same effect. After each load increment there is a reduction in voids. Due to the sample being saturated or close to saturation, this reduction in voids causes an excess in pore pressure. Half of this pressure acts against the hydrostatic pressure from the falling head permeability test being applied to the sample and therefore slightly affecting the readings. In time, the pore pressure stabilizes and the permeability reduces and stabilizes.

8.2. The Triaxial Apparatus

8.2.1. Test method and apparatus

Permeability testing is performed using a standard triaxial cell. The advantage of testing under triaxial conditions are twofold. Firstly, a fairly realistic in-situ stress condition in created and secondly, the chance of sidewall leakage is limited. The sample is encased in a latex membrane sheath and when the cell pressure is increased to more than the pore pressure, the latex molds itself around the uneven sides of the sample creating an effective seal.

Three triaxial cells were used to perform these series of tests. The cells were mounted on a table and all connected to the same cell pressure, pore pressure and back pressure reservoirs. The pressure reservoirs were supplied from a compressed air source via manually set pressure control valves. The sample pore drain line from the top cap was connected to a measuring system with which the permeability could be calculated. The measuring system consisted of a 2.6mm Ø glass tube about 500mm long. A compressed air supply line joined into the cell end of the glass tube. This allowed a bubble of air to be inserted into the permeant water which would act as a marker to measure the flow from which the permeability could be calculated. A typical layout of one cell in the system is shown in Fig. 5.

In order to shorten the test period, a pure bentonite sample of 5mm thickness was used. The sample is prepared by lightly compacting and leveling a sample into a 5mm high ring with a diameter of about 75mm. A piece of filter paper and a porous disc are then positioned on the sample. A 30mm Ø sample cutter is then pushed over the disc into the 5mm layer of bentonite. A wooden piston is inserted into the back of the sampler and pushed down firmly onto porous disc. The sample cutter and the piston are lifted up together containing the bentonite sample and porous disc. The top filter paper, porous disc and top cap are then placed onto the bentonite sample in the cutter. The sample is slowly extruded while the latex membrane is carefully rolled over the sample. The sample is placed on the base pedestal of the triaxial cell and the top cap is connected to the pore drain (back pressure line). A latex membrane positioned over the sample either seals or is sealed with rubber bands onto the base and cap.

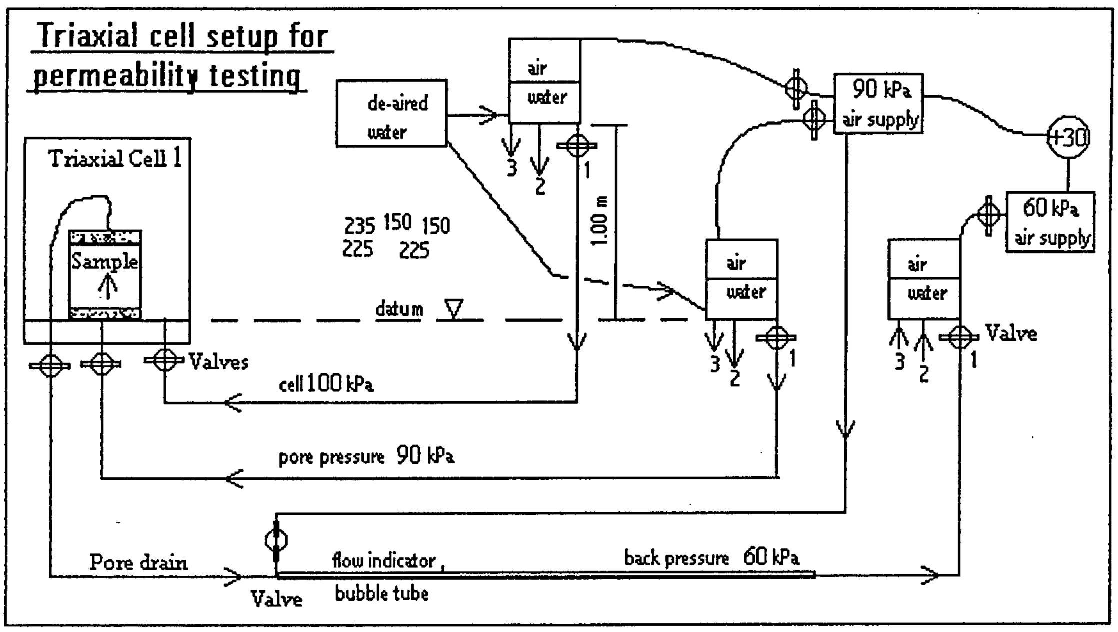


Fig. 5: A diagram layout of one of the three triaxial cells used to measure the permeability of bentonite.

The cell is first filled with de-aired water and the base pedestal drain lines are flushed and filled with de-aired water. The back pressure line is kept closed. The cell pressure and sample pore pressure are incrementally increased in approximately 50kPa steps. The pore pressure must remain at 10kPa lower than the cell pressure to insure that the latex membrane maintains its seal against the sample. This was achieved by placing the reservoir supplying the cell pressure one meter higher than the reservoir supplying the pore pressure and supplying both reservoirs with the same air pressure. Once the required cell pressure and pore pressure have been reached, the back pressure line is opened and the pressure is applied.

8.2.2. Test results

Tests were conducted on Plett, Koppies and Culbond bentonite. Culseal could not be tested in this apparatus due to its granular nature which did not allow it to be prepared in the same way as the other finer samples were. All of the tests were conducted with a pressure difference across the sample of 40kPa and a cell pressure of 10kPa higher than the pore pressure. Two series of tests were conducted. The first series was subjected to a cell pressure of 100 kPa and the second series were done under a cell pressure of 200kPa. Problems were encountered throughout the test period with the manually controlled pressure valves due to an inconsistent supply pressure. The same type of bentonite was tested in all three cells at the same time which provided comparative testing and irregularities such as sidewall leakage, membrane damage or equipment malfunction could be detected at an early stage.

The permeability is calculated by measuring the distance the permeant flows along the glass rod in a specific time. It took at least 5 days before the permeant reached the measuring tube and after that a minimum of three readings were taken at 24 to 48 hour intervals for each test. A summary of the results of the tests performed are shown in Table 8. A typical triaxial permeability test results sheet is shown in Appendix H.

	Triaxial Test Summary (m/s)					
	Coefficients of permeability					
	Cell Pressure = 100 kPa			Cell Pressure = 200 kPa		
	Back Pressure = 60 kPa			Back Pressure = 160 kPa		
Cell	Koppies	Plett	Culbond	Koppies	Plett	Culbond
1	$2.18x10^{-11}$	$4.57x10^{-11}$	$4.81x10^{-11}$	$2.25x10^{-11}$	$3.71x10^{-11}$	$4.4x10^{-11}$
2	$2.36x10^{-11}$	$4.67x10^{-11}$	$4.27x10^{-11}$	$2.55x10^{-11}$	blocked	$3.56x10^{-11}$
3	$2.8x10^{-11}$	blocked	$3.43x10^{-11}$	damaged	$2.73x10^{-11}$	$4.64x10^{-11}$
average =	$2.44x10^{-11}$	$4.62x10^{-11}$	$4.17x10^{-11}$	$2.4x10^{-11}$	$3.22x10^{-11}$	$4.2x10^{-11}$

Table 8 : Summary of the results of the permeability tests conducted on the selected South African bentonites under triaxial conditions.

8.2.3. Discussion and analysis of results

The advantage of using three cells is evident in the results in Table 9. Half of the batch tests performed had one cell failing. Although it was found in the consolidometer permeameter that an increase in vertical stress resulted in a decrease in permeability, this was not experienced in the triaxial cell where the sample is in a different state of stress. The variation in permeability between the different types of bentonite is minimal, although Koppies bentonite consistently produced permeability coefficients of below $3x10^{-11}$m/s.

Hydraulic gradients of a maximum of 800 were used in the triaxial tests in an attempt to shorten the test periods. The highest gradient mentioned in the literature at hand was 340 used by Mollin et. al. (1996) for a soil/bentonite sample. Edil and Erickson (1985) indicated that high hydraulic gradients could cause physical changes within the sample which would result in a decrease in permeability. Some irregularities were found with the tests conducted in this research with respect to variances in permeability under different hydraulic gradients. The triaxial test results are one order of magnitude lower (more permeable) than the results obtained in the consolidometer permeameter under the 221 kPa load increment where the hydraulic gradient ranged between 360 and 200 under falling head test conditions.

8.3. Standard Falling Head Permeameter

8.3.1. Test Method and Apparatus

Permeability tests are performed on thin layers of pure bentonite under controlled swelling conditions using standard permeameters modified in a simple way. These permeameters are classed as fixed-wall permeameters. Four of the eight permeameters available are referred to as small permeameters with an internal diameter of 73.8mm. The other four are large permeameters and have an internal diameter of 113.6mm (according to BS 1377).

In order to test the permeability of a material like bentonite within a reasonable period of time, a very thin layer of bentonite and a high head of water is required to obtain a high hydraulic gradient. The D/CHWSS provided a 3m head of de-aired water, therefore a layer of less than 10mm of pure bentonite was required to achieve a hydraulic gradient comparable to the consolidometer permeameter and triaxial test. Control over the swelling of the bentonite was also required in the permeability tests to be comparable to the two tests just mentioned earlier and the new Mini Permeameter apparatus. Unfortunately, the constant head standpipe outlet take-off points on the cylinder limit the depth to which the confining piston can be pushed making the standard permeameter unsuitable for thin layer testing. This problem was overcome by filling the cylinder with a coarse dry sand up to the top outlet nipple. A layer of finer sand (15mm thick) is then placed onto the coarse sand and gently compacted. A filter paper is then placed onto the sand and the bentonite was carefully poured into the permeameter. A steel plunger purpose made to just fit into the permeameter was used to level and lightly compact the bentonite. Bentonite was added and leveled until the desired thickness was obtained. A schematic of the standard permeameter with the fill layers is shown in Fig. 6.

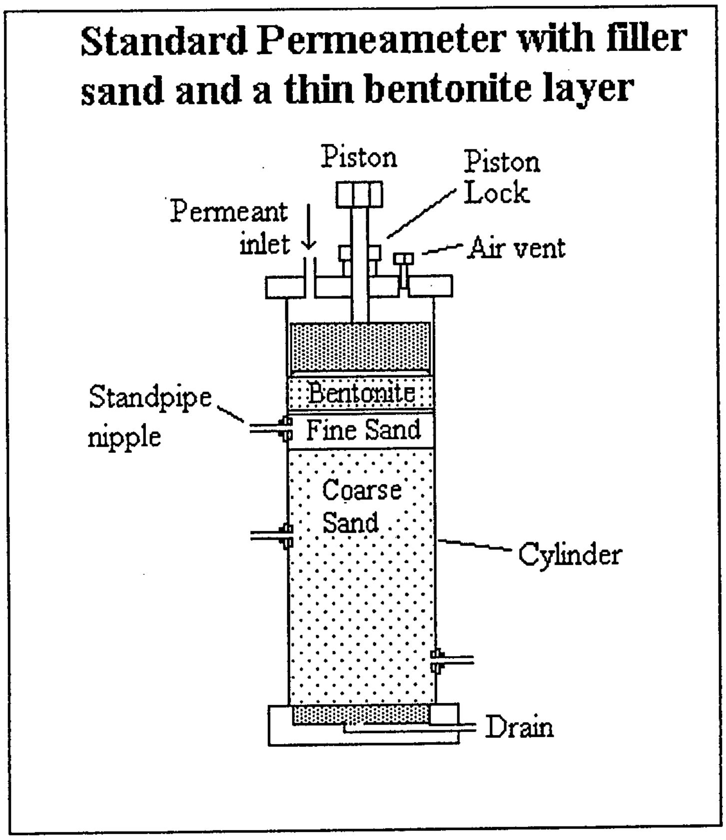
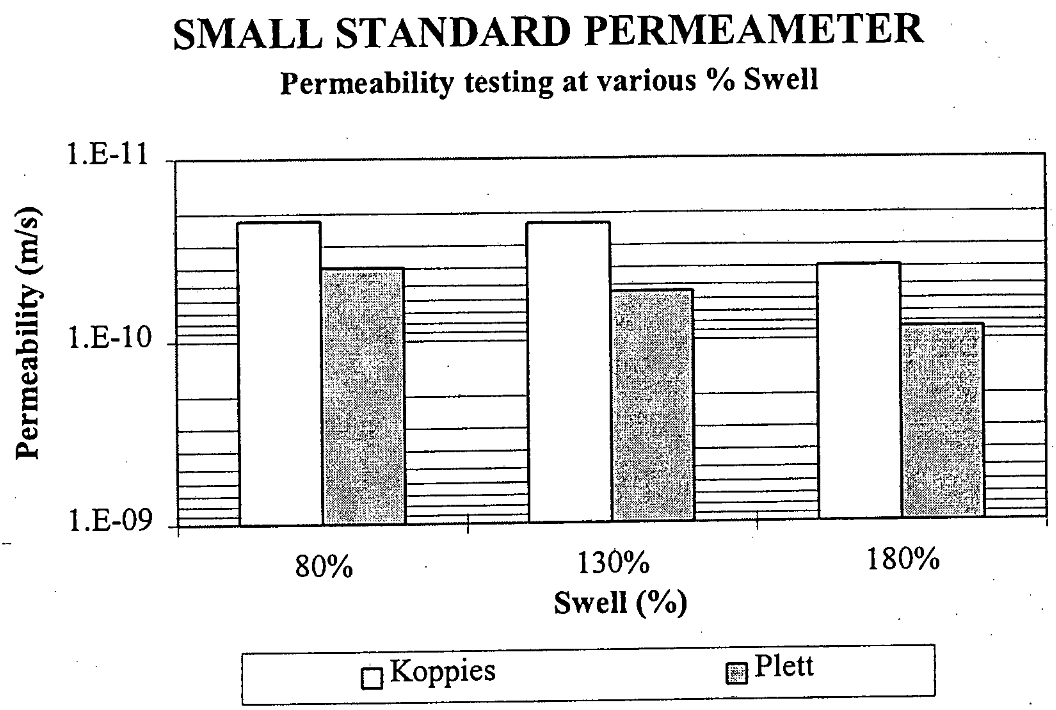


Fig. 6: Schematic of the Standard Permeameter with the thin layer of bentonite above the sand fill/drainage layers.

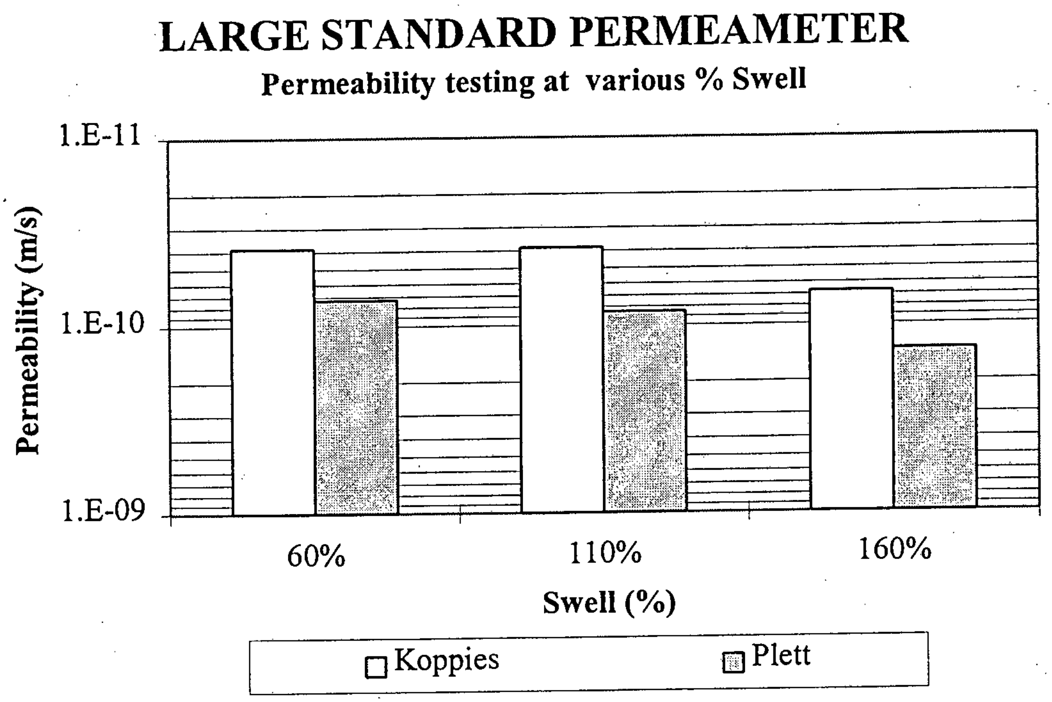
After the bentonite sample had been prepared on the sand, the top cap of the permeameter was fastened into position and the piston was pushed down so that it just touched the bentonite. The permeameter was then filled with de-aired water and allowed to stand for one hour before applying the 3m head supply from the D/CHWSS. It was found that if the head was applied immediately, sidewall leaks appeared instantaneously. If the bentonite was allowed to hydrate under 100mm of water for one hour, it sealed itself against the sidewall and the full head could then be applied. Due to the way the sample was prepared and the very low permeability of bentonite, a constant head test could not be used effectively to test the permeability. The permeameter remained connected to the 3m constant head until the sample was completely swollen, after which the water supply system was rerouted to a standpipe positioned 3m high and a falling head test was conducted to calculate the permeability. After doing at least three permeability tests on the samples, the piston was slowly lifted to the desired position to which swell was to be permitted and the system was rerouted to the 3m head supply. Once the bentonite has swollen to the new height, the inflow was again rerouted to the falling head system and permeability testing was again commenced. The sequence was repeated for the required swell stages tested.

8.3.2. Test results

Due to the results of these tests being used for comparative reasons only, and the uniqueness of this form of permeability testing related to swell which was beyond the aims of this research work, a decision was made to limit testing in this direction. For these reasons, only two bentonite samples were tested and the swell increments were limit to three. This would provide sufficient data to be comparable to the other experiments and give an idea of the effects of swell providing a foundation for future work. The samples tested were Koppies and Plett bentonite which were allowed to swell to 80%, 130% and 180% in the small permeameters and 60%, 110% and 160% in the large permeameters. The initial height of the sample was 5mm in all tests. A summary of the results of the tests done in the small permeameters is presented in Graph 17 and the results of the tests done in the large permeameters are presented in Graph 18.



Graph 17 : Permeability results of Koppies and Plett South African bentonite under three swell increments.



Graph 18 : Permeability results of Koppies and Plett South African bentonite under three swell increments.

8.3.3. Discussion and analysis of results

There is a defined increase in permeability with an increase in swell. This trend occurs with both the Koppies and the Plett bentonite in the small and large permeameters. The coefficients of permeability closely match those achieved in the triaxial apparatus and are slightly higher than the results obtained under the first load increment in the consolidometer permeameter, however they are within the same order of magnitude.

The results are not consistent between the small and the large permeameters. The permeability results obtained in the large permeameters are on average in the order of 2.9×10^{-11}m/s higher than those achieved in the small permeameters. The cause of this could be that as the sample swells, the contact pressure between the side-walls is reduced. Although the surface contact area is increased, there may be areas where the increased swollen bentonite has not sealed itself against the cylinder, and the existing seal (before swell) is reduced due to the reduction in contact pressure. This would result in an increase in permeability which in this case occurs more in the large permeameter where the area to seal is greater and a higher probability of surface contact weaknesses exists.

A number of problems occurred with the preparation and hydration of the samples. Sidewall leakage initially resulted in a number of failures but this was overcome by allowing the sample to hydrate for a longer period under a lower water head. It was also found that by cleaning the cylinders thoroughly, sidewall leakage was also reduced. This could have been related to the silicone grease used to lubricate the seals on the top piston which may have caused the reduction in surface tension and positive contact between the bentonite and perspex cylinder.

8.4. Mini Permeameter

8.4.1. Test method and apparatus

The Mini Permeameter is an adaptation of the standard permeameter and fixed-cylinder permeameter developed as an economical alternative to the generally expensive geotechnical laboratory apparatus. This permeameter has recently been developed in the Geotechnical Laboratory at UCT for testing processed bentonite (in powder form). The permeameter uses the constant head test method with down-flow conditions. Swelling of the sample in the upward vertical direction is permitted if required. The apparatus is schematically shown in Fig. 7 and consists primarily of a glass tube of a 18mm ∅ and a length of 100mm with rubber stoppers at either end. The rubber stoppers have holes in them to allow the permeant through the system. The top stopper is connected to the available constant head de-aired water supply of 2.15m. The bottom stopper hole has a piece of geotextile in it to hold back the filter sand at the base of the cell. A collecting flask is connected to the outer part of the base stopper in such a way that evaporation is limited but no pressure build-up can occur.

Advantages of this permeameter are size and simplicity making it easy to run a number of tests simultaneously and economically. Due to the glass wall, a sample can be inspected visually for irregular sidewall leakage and the rates of swell can also be monitored.

The bentonite sample is placed above a filter layer and a small surcharge load to limit swell can be applied to the sample by placing a layer of coarse sand above it. The bentonite sample is separated from the filter and surcharge layers with filter paper. The filter paper also provides a precise reference line for the purpose of measuring the swell. The collecting flask is removed periodically and weighed to ascertain the amount of water which has permeated through the sample. Readings commence when water is first noticed in the collecting flask, which could be anything from 1 day to 1 month. Although the collecting flask is connected to the stopper with as little gap as possible, evaporation still occurs during testing and also when the flask is removed for its weigh to be taken. The evaporation is monitored by partly filling one collecting flask with water, the Mini Cell is positioned with the lower rubber stopper hole sealed off. This flask must be weighed every time the other collecting flasks are weighed, and from the decrease in weight the evaporation, although very small, can be calculated and appropriate corrections made to the other flasks.

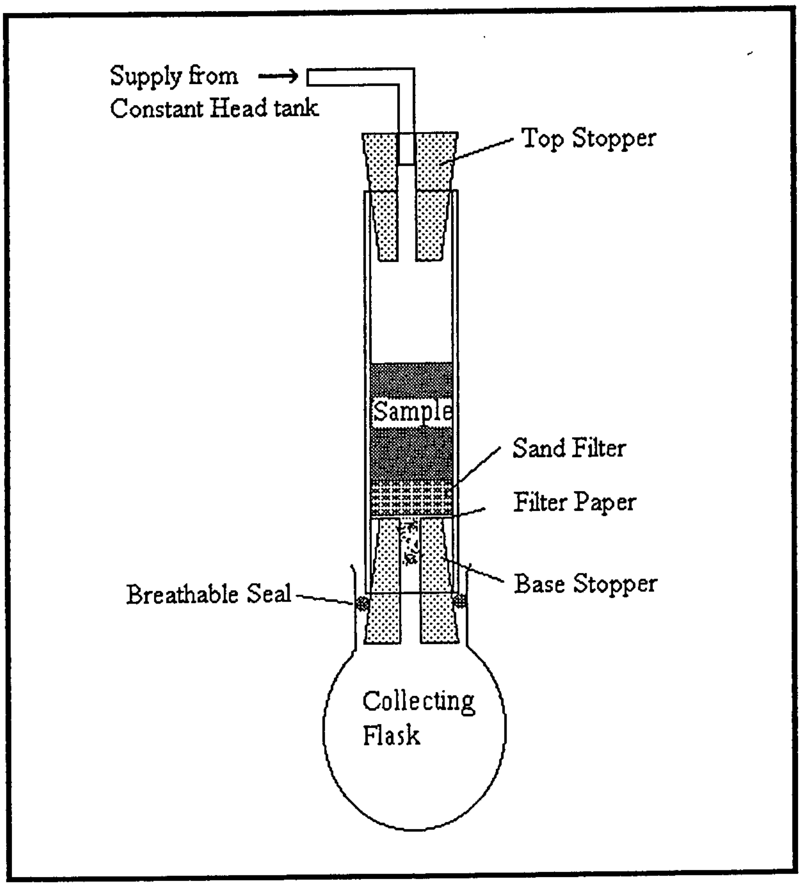
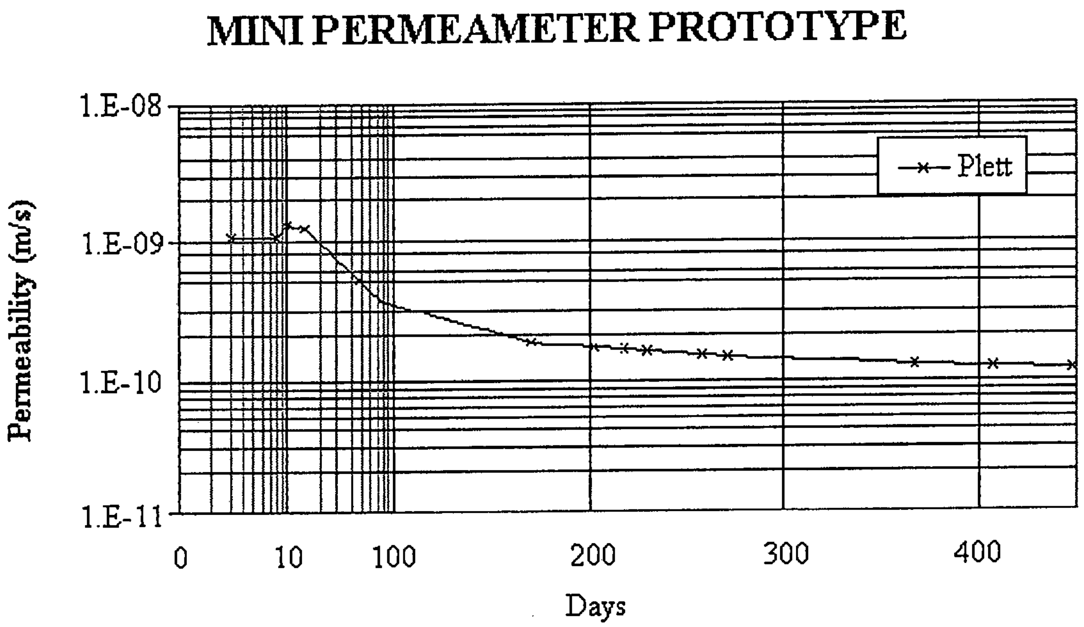


Fig. 7: Schematic of the Mini Permeameter with a sand filter layer.

8.4.2. Test results

The prototype Mini Permeameter test using Plett bentonite ran for over 500 days without any sidewall leakage or other irregularities occurring. The permeability coefficients measured as a cumulative collection of permeant are presented in Graph 19. The first 100 days are presented on a log scale and the remainder are presented normally to show the long term permeability changes more distinctively. The lowest coefficient of permeability measured in a single test period was 5.4×10^{-11}m/s.



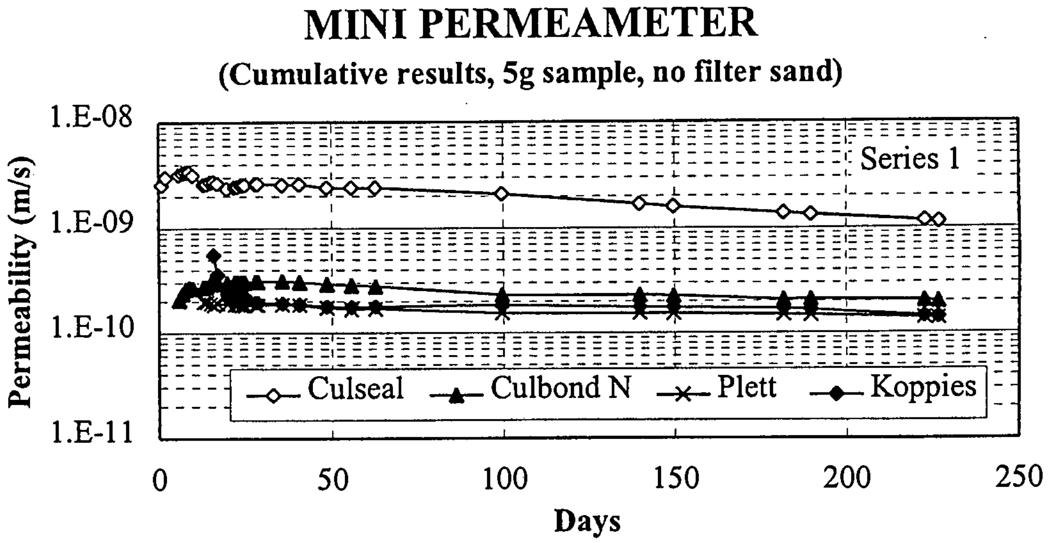
Graph 19 : Permeability results of Plett bentonite tested in the prototype Mini Permeameter for over 500 days.

The four selected samples of South African bentonites namely: Culseal, Culbond, Plett and Koppies are used in all the various Mini Permeameter tests. The first series of tests conducted in the Mini Permeameter were done using only filter paper below the bentonite sample and no surcharge load. The samples weighed 5g each which resulted in a height of approximately 25mm. De-aired water was used for the test at a constant head of 2.15m. The test ran for 227 days after which the permeability almost ceased, and the test was stopped due to the presumption that the filter paper had blocked.

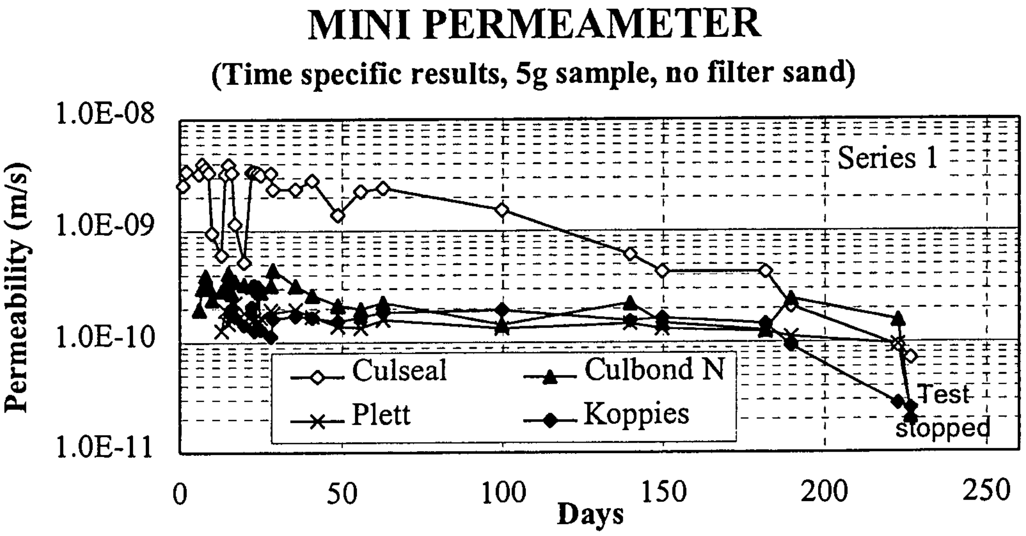
The results of the Mini Permeameter permeability tests can be presented in two ways. The first way shows the permeability calculated as an accumulation of the permeant. The results of the first series of tests is presented in Graph 20 in this way. The second way to present the results is to calculate the permeability for each time interval between readings. This is shown using the first test series in Graph 21. The first way provides a more averaged value were as the second way clearly shows the short term variations in permeability. The advantage of calculating the permeability for each time interval is shown in the first series of tests where a sharp drop in permeability occurred after 220 days. However, the average values are regarded as more realistic of in-situ conditions. The final swell thickness which give an idea of the density of the hydrated bentonites were : Culseal 180%, Koppies 152%, Plett 128%, Culbond 112%.

The second series of tests conducted on the four bentonites had a different arrangement with a 3g layer of filter sand approximately 15mm thick placed below the bentonite to insure even drainage of the bentonite layer. The bentonite samples used weighed 2g and provided a height of 10mm. A thinner layer was selected in an attempt to shorten the period of time before readings were obtainable. The cumulative permeability results of these tests are shown in Graph 22. The final swell thickness reached by the samples were as follows: Culseal 189%, Koppies 170%, Culbond 150%, Plett 120%.

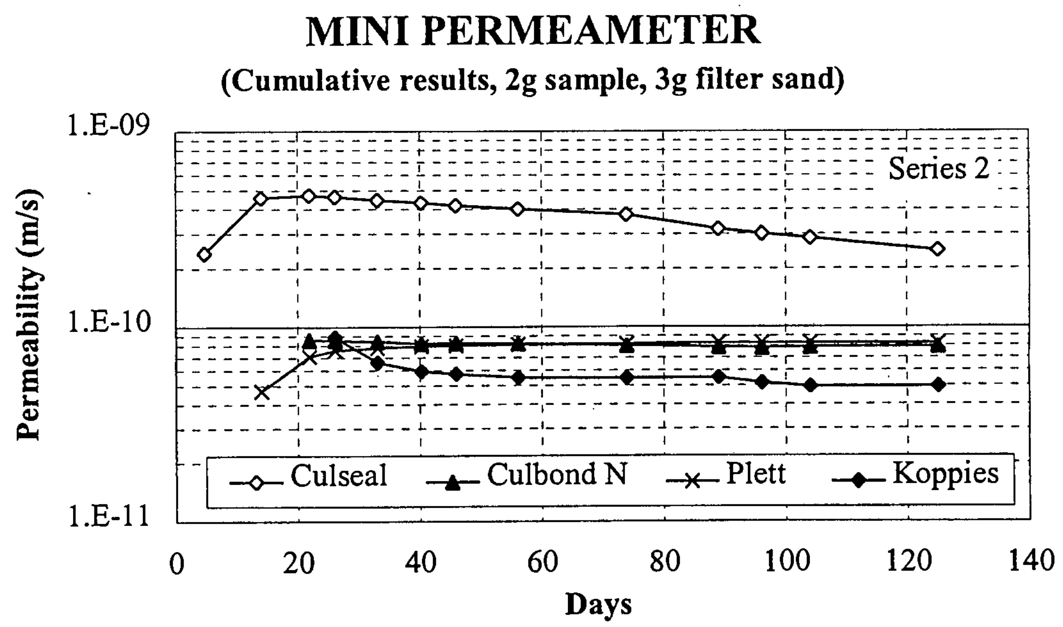
The third series was conducted using the sand filter layer again which proved effective in the second test series. A sample mass of only 1g providing a layer of approximately 5mm and was selected to further reduce the period required before readings were obtainable. The results of these tests are shown in Graph 23. The tests were terminated at 84 days after the Culseal sample developed a sidewall leak which caused irreparable damage to the whole test series. The final swell thickness reached by the third series was as follows: Culseal 300%, Koppies 240%, Culbond 220%, Plett 133%.



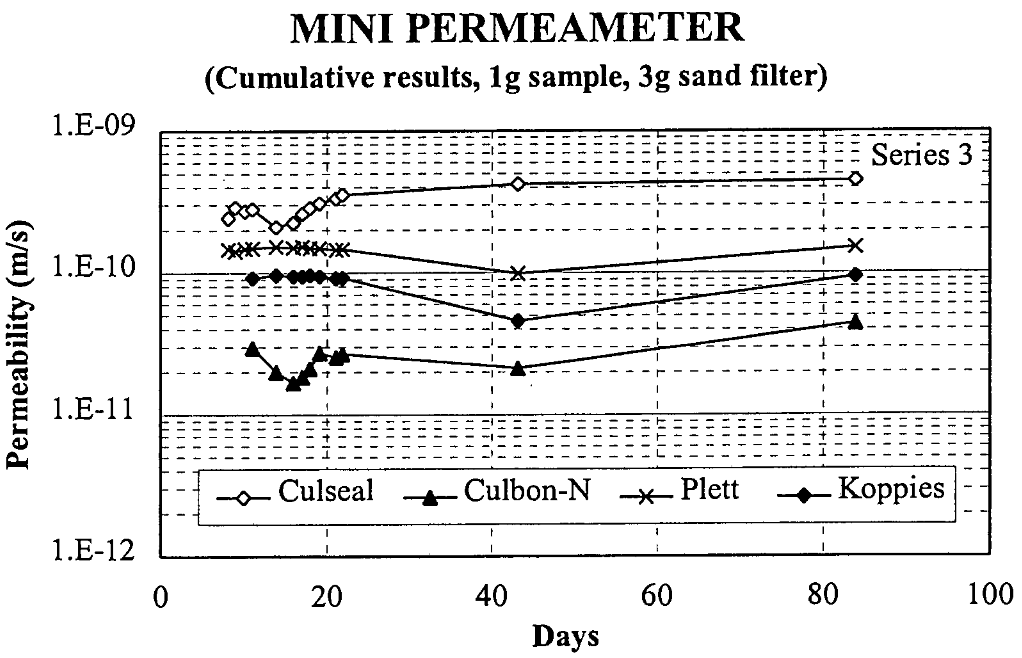
Graph 20 : Average permeability values of the first test series of the selected bentonites measured with swelling allowed.



Graph 21 : Individual test period permeability values of the first test series of the selected bentonites measured with swelling allowed.

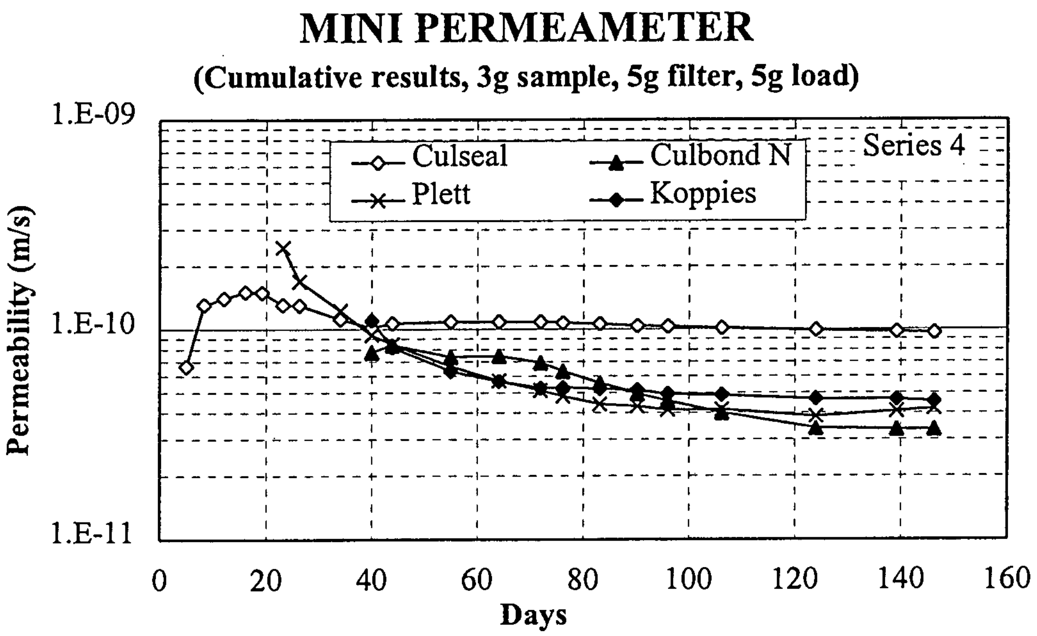


Graph 22 : Average permeability values of the second test series of the selected bentonites measured with swelling allowed.

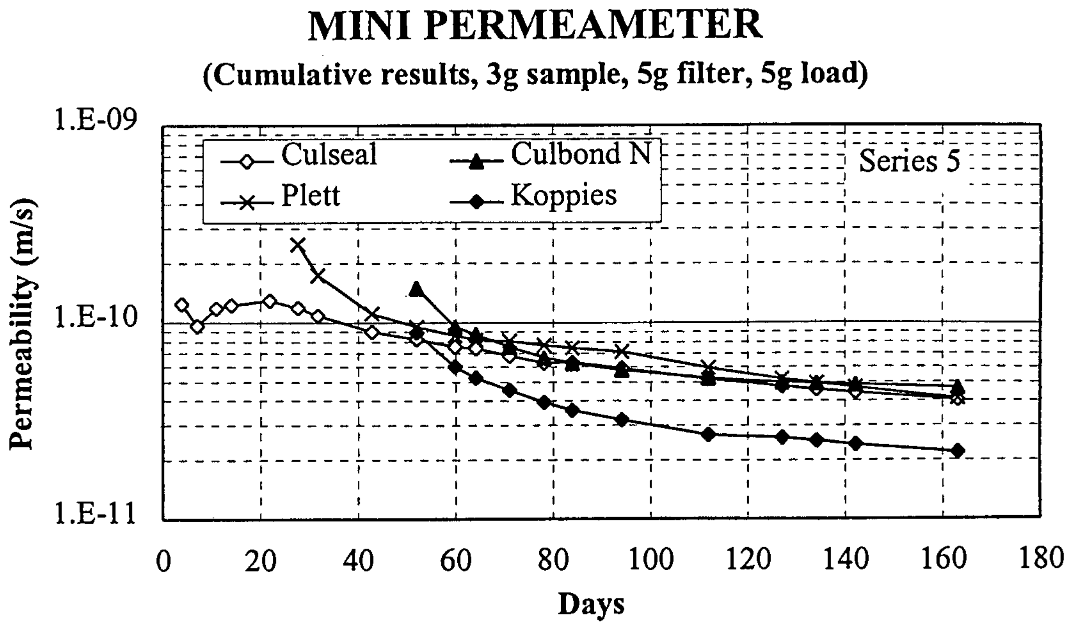


Graph 23 : Average permeability values of the third test series of the selected bentonites measured with swelling allowed.

The forth and fifth Mini Permeameter series were tested concurrently on the four selected bentonites under the same conditions. This was done to check the consistency of the apparatus. A 3g bentonite sample providing a thickness of about 15mm was used with a 5g sand filter below and a 5g sand surcharge load above the sample separated with filter paper. The surcharge load was applied to limit the swell so that the effect on the permeability could be monitored. The permeability tests ran for over 145 days and the results of the forth and fifth series are shown in Graph 24 and Graph 25. The maximum swell readings for the forth series were as follows: Koppies 129%, Culseal 117%, Culbond 65%, Plett 56%. And for series five: Culseal 117%, Koppies 94%, Plett 69%, Culbond 63%.



Graph 24: Average permeability values of the forth test series of the selected bentonites measured with a surcharge load.



Graph 25: Average permeability values of the fifth test series of the selected bentonites measured with a surcharge load.

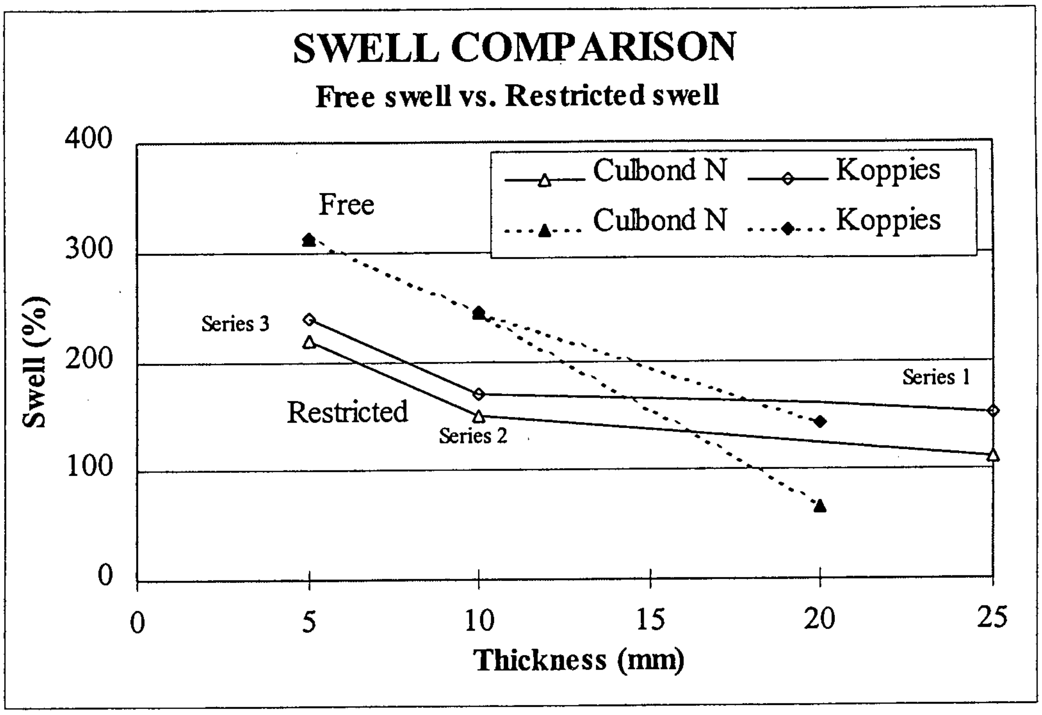
8.4.3. Discussion and analysis of results

The initial height on the prototype sample was 10mm which makes the test comparable to the Plett bentonite test conducted in the second series of Mini Permeameter which had the same initial height. The prototype permeability results are slightly higher averaging out to 1.5x10^{-10}m/s compared to 8.0x10^{-11}m/s. The difference in permeability is almost non-existent when the prototype is compared to the third series where an initial layer thickness of 5mm was used. This motivates the possibility of sidewall weak points in the prototype which reduced the effective thickness of the bentonite layer. The constant reduction in permeability with time was presumed to be caused by the filter system consisting of a piece if geotextile which was slowly getting clogged.

The first series (5g sample, no filter, no surcharge) of tests in the Mini Permeameter produced results after only a few days and these were all irregular. Only after about 30 days did the results become more regular except for Culseal whose permeability kept dropping until 140 days into the test period before stabilizing. The initial irregularities in permeability were caused by the hydrating and swelling of the bentonite which stabilized after 20 days. At approximately 180 days a marked drop in permeability was noticed with the Culseal and Koppies samples and similarly for Culbond and Plett at about 220 days. It was assumed that the filter paper had blocked, and the test was therefore terminated. The coefficients of permeability are one degree higher than those achieved in the second series of tests which consisted of a 2g sample with a sand filter layer. No evaporation monitoring was done during this period and a correction factor from a later test was used which could have affect the results, although insignificantly.

The difference in permeability results between the first and second series could also be attributed to slight errors in measuring the flow quantities in the collecting flasks and general equipment problems with the first series of tests. Most of these were eliminated and the process streamlined for the second series of tests. What could also have effected the results is that due to the first series of samples not having a filter sand layer, insufficient drainage occurred within the sample. This caused an increase in the pore water which resulted in a less dense and therefore a more pervious material. This is further motivated by comparing the final swell of the Culbond and Koppies samples of the first test series with the second and third series of tests conducted on the same products. The comparison is depicted in Graph 26.

When examining Graph 26, it is evident that the swell for the first series (represented by its initial thickness of 25mm), does not follow the same trend as the swell results of the second and third test series. Neither does it compare to the almost linear trend experienced by the free swell tests done on the same material. The difference in swell between the free swell and the restricted swell samples is almost identical at the initial heights of 5mm and 10mm but varies dramatically for the 20mm and 25mm samples. From the tests performed, it is predicted that the swell for the 25mm restricted samples should lie below the 100% swell line. If a sand filter layer was used in the first test series, the permeability readings would be similar to those recorded in test series two and three. Thus, the effectiveness of the sand filter layer providing a more dense and therefore less pervious material is evident when the final swells of the samples are compared with each other and to the results of the free swell test.



Graph 26 : Free swell test results compared to restricted swell test results for Koppies and Culbond bentonite.

The use of a thinner sample layer of 10mm in the second test series did not reduce the length of time for the first permeability readings due to the permeant having to saturate the filter sand before reaching the collecting flask. Two of the samples produced readings after 20 days and stability in the results occurred only after 40 days. The permeability of Culseal continued to drop throughout the test and it is expected that due to its granular nature, the more dense particles take longer to hydrate and therefore as hydration continues, the permeability decreases.

The time interval before permeability readings are achieved was reduced to about 10 days in the third series with the use of 1g samples providing an initial thickness of 5mm. The permeability results seemed to reach stability after 20 days, but then continued to drop for a further 20 days. A permeability test performed at 84 days showed a marked increase in permeability and while weighing the Culseal samples permeant, a severe sidewall leak occurred which resulted in the termination of the third test series. Culbond produced the lowest permeability of 2.0×10^{-11}m/s which is inconsistent to the other Mini Permeameter tests performed and should be in the order of 4.0×10^{-11}m/s. The permeability results do however match the results obtained in the consolidometer permeameter under the 55.7kPa load increment.

Although the permeability results of test series four closely match those of series five (which was anticipated due the test setups being identical), there were some irregularities. The results of Plett and Culbond are very closely matched and follow the same pattern as Koppies bentonite of a decrease in permeability over time. The largest difference occurred with Culseal where the permeability varied from 9.6×10^{-11}m/s recorded in the fourth test series to 4.0×10^{-11}m/s recorded in series five. It was also noticed that in series four, the permeability did not drop significantly with time. Koppies bentonite also produced a lower permeability reading in test series five and with Culbond, produced their first results at 50 days, 10 days later than in series four. Possible reasons for these variances could be experimental error which is unlikely due to the general consistency of results, or variations within the selected sample. The samples were all taken from the same batch but due to the small size of the sample used for testing, the effect of a lower or higher quality portion within the sample could greatly affect the results. The same could be said for most of the small variations experienced in all of the other permeability tests performed.

The effect of the light surcharge of 0.25kPa (5g) on permeability varies between the different types of bentonite. Culseal experienced the greatest reduction in permeability due to the surcharge, dropping from 2.5×10^{-10}m/s to

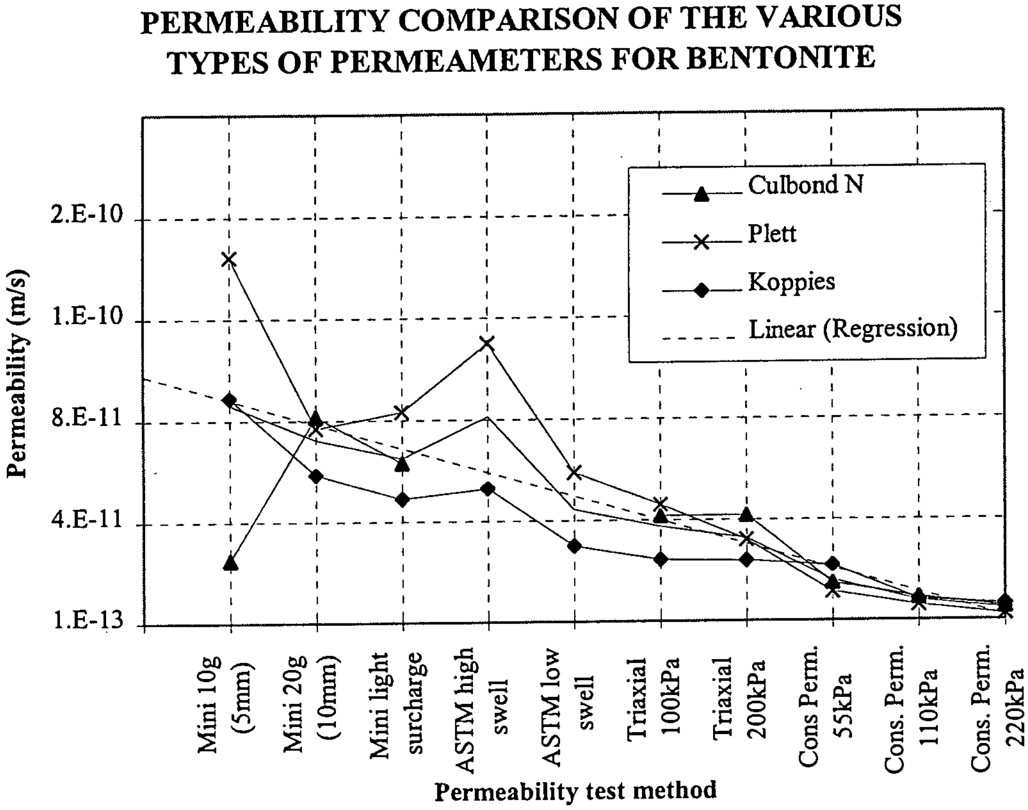
9.9×10^{-11}m/s. Culbond and Plett dropped by about 4.0×10^{-11}m/s due to the surcharge and Koppies bentonite experienced almost no change.

In general, the permeability tests conducted in the Mini Permeameter with no surcharge are consistent to within an accuracy of 5.0×10^{-11}m/s. A sample thickness of less than 10mm could experience sidewall leakage and should be treated with care. Although permeability test results are obtainable after about 20 days, at least 20 days of testing should follow once the first permeant is obtained before an accurate assessment can be made. By applying even a light surcharge to bentonite, the permeability can be expected to drop in most cases.

In all the tests conducted in the Mini Permeameter, the permeability continued to drop for 120 days or more. In order to limit the overall test period, it is suggested that the permeability is measured 20 days after the first permeant is collected and the coefficient of permeability calculated is then multiplied by a time reduction factor of 0.60 to obtain a final permeability.

8.5. Discussion on Bentonite Permeability Testing

The effect of the permeability test apparatus and test conditions greatly affect the permeability results of pure bentonite. Variations from 1.5x10^{-10}m/s to 1.0x10^{-12}m/s were experienced between the tests conducted in this research. These variations were mostly the result of changes in the confining stress which was different for each type of permeameter. A summary of permeability results obtained in the various types of permeameters used to test pure bentonite in the previous chapters are presented in Graph 27.



Graph 27 : Comparison of the permeability test results of pure bentonite obtained with the various types of permeameters.

A phenomenon previously noted by many authors including D.E Daniel (1985, 1993) and R Trauger (1992) etc., is the reduction in permeability with increased stress. This is evident in Graph 27 where the tests conducted in the Mini Permeameter are done under very low or no confining stress and the tests conducted in the consolidometer are under confining stresses of up to 220kPa.

The effect of the increased stress is threefold; firstly it creates a more dense material which effectively reduces the voids which in turn limits the flow of liquid; secondly it restricts the free swell uptake of water which causes a less dense material and thirdly, it forces the swelling bentonite into all the voids including those areas not filled by zones of inactive bentonite.

High confining stresses are not always available when using a bentonite liner for example a landfill capping which has possibly only 0.5m of soil cover. For this reason it is necessary to use the correct laboratory test apparatus whenever possible to match the lower stress conditions expected in the field so that an accurate assessment of the permeability can be made. Graph 27 provides a basis for the selection of the type of permeameter best suited for determining the permeability of a specific type of bentonite in a field application. The Mini Permeameter simulates thin layer bentonite liners with very low or no stress (free swell conditions) and also allows for high hydraulic gradients. The ASTM standard permeameter can be used for the same conditions as the Mini Permeameter but provide a more useful role in that the swell volume can de controlled and light vertical stresses can be applied. The Triaxial cell offers the most realistic simulation of field conditions but is not suited for low confining stresses and very high confining stresses. The confining stress or cell pressure is directly related to the hydraulic gradient and at low pressures the hydraulic gradient is also low resulting in long waiting periods to obtain permeability results. Specialized cells and the respective test setup are required for high pressure testing. The consolidometer permeameter is best suited for high vertical confining stresses. A high vertical stress relates to an increase in the horizontal stress which greatly reduces sidewall leakage. The hydraulic gradient is independent of the applied load except under low vertical stresses where the up-flow hydraulic force cannot exceed the applied load. The effect of this is that lower hydraulic gradients are required which result in longer test periods.

The correct laboratory equipment to simulate field conditions is not always available and certain test equipment is very costly and intricate to operate and could be uneconomical to use for long permeability tests. It may therefore be necessary to use an alternative test apparatus to obtain the required data. A comparison of the permeability results in Graph 27 provides a solution to this problem. Depending on the apparatus available, a correction factor obtainable in Table 9 can be applied for each situation to the coefficient of permeability to obtain a more realistic value. Tests resembling field conditions should be used whenever possible and large differences in confining stresses should be avoided when using comparative tests.

Test Method		Mini 10g	Mini 20g	ASTM	Mini	ASTM	Triaxial	Triaxial	Cons Perm.	Cons. Perm.	Cons. Perm.
conversion factors		(5mm)	(10mm)	high swell	surcharge	low swell	100kPa	200kPa	55kPa	110kPa	220kPa
		convert from									
Mini 10g (5mm)	c	1	1.077	1.231	1.436	1.936	2.302	2.638	5.363	11.047	18.334
Mini 20g (10mm)	o	0.928	1	1.143	1.333	1.798	2.137	2.449	4.979	10.256	17.021
ASTM high swell	n	0.812	0.875	1	1.167	1.573	1.870	2.143	4.357	8.974	14.894
Mini light surcharge	v	0.696	0.750	0.857	1	1.348	1.603	1.837	3.734	7.692	12.766
ASTM low swell	e	0.516	0.556	0.636	0.742	1	1.189	1.362	2.770	5.705	9.468
Triaxial 100kPa	r	0.434	0.468	0.535	0.624	0.841	1	1.146	2.330	4.799	7.965
Triaxial 200kPa	t	0.379	0.408	0.467	0.544	0.734	0.873	1	2.033	4.188	6.950
Cons Perm. 55kPa		0.186	0.201	0.230	0.268	0.361	0.429	0.492	1	2.060	3.418
Cons. Perm. 110kPa	t	0.091	0.098	0.111	0.130	0.175	0.208	0.239	0.485	1	1.660
Cons. Perm. 220kPa	o	0.055	0.059	0.067	0.078	0.106	0.126	0.144	0.293	0.603	1

Table 9 : Conversion factors for inter-relating coefficients of permeability of pure bentonite obtained through the various test methods.

9. Permeability Testing of Soil/Bentonite Mixes

Due to practical, structural and economical considerations, the use of bentonite mixed with soil to form an impervious liner is favoured in preference to the pure bentonite liner. The required coefficient of permeability can be achieved by altering the percentage of bentonite, and coefficients of permeability close to that of the pure bentonite are obtainable. For a liner to be effective not only requires a high degree of control during the construction process, but sufficient preliminary testing is required to establish the optimum bentonite content in a particular liner mix and to predict the effects of the permeant on the liner.

The permeability of soil/bentonite mixes can be tested in a number of ways. The most common are; the compaction mold permeameter (of which there are a number of variations), the consolidometer permeameter and the triaxial or flexible-wall permeameter. The advantage of the consolidometer and triaxial permeameters is that samples can be tested under a confining stress and undisturbed samples can be taken from site and tested in these devices. Once constructed, the permeability of the liner can be tested in-situ using an infiltrometer of which there are a number of types and test methods.

The permeability tests conducted on the selected South African bentonites were performed on samples prepared in the laboratory. Tests were conducted on mixtures ranging from 4% to 14% by mass of bentonite with sand. The samples were prepared in a compaction mold and left to hydrate before being permeated using double-ring permeameters. After testing, one sample was cut from each compaction mold and placed into a consolidometer permeameter where its permeability was re-tested under increasing loads. In both test setups the permeant liquid used was de-aired tap water the details of which are given in Sub-section 7.2.

9.1. Double-Ring Compaction Mold Permeameter

9.1.1. Test method and apparatus

The Double-Ring Compaction Mold Permeameter is an improved version of the standard compaction mold permeameter. Its main feature is that the base of the mold from where the permeant is collected is divided into an inner and outer ring of equal area. The permeant from the two ring areas are collected separately and the volumes are compared providing an indication as to any possible sidewall leakage. When a significant difference is noted, the sample is usually remade and re-tested, although a result is still obtainable by measuring the permeant collected from the inner ring only.

A constant head or falling head test method can be used to determine the permeability. For soil/bentonite mixes where a very low permeability can be expected, the constant head test could take weeks to obtain a single reading and is thus not recommended. The falling head test with a test period of only a few days is therefore preferred and used in this research work.

In order to test a number of samples in a relatively short period of time requires the use of a large number of permeameters. Due to the sample having to stand for two weeks or more to hydrate means that the number of compaction molds are approximately three times that of the double-ring base and top caps required for permeability testing. This results in a large number of apparatus required at an equivalent high cost. An economical solution was found by using a 110 mm Ø HDPVC pipe cut into sections of 150mm long to form compaction molds. The top caps and double-ring bases were made of a similar material. A diagram of the double-ring compaction mold permeameter is displayed in Appendix I. Although PVC is a cheap alternative, it must be emphasized that it may not be resistant to aggressive permeants.

The soil/bentonite samples were prepared by firstly mixing the sand with the required amount of water to achieve a moisture content of approximately 2%

higher than the optimum moisture content for the mixing soil based on recommendations by Kenny et al. (1992) and Ather (1987). The bentonite was then mixed into the soil using a mechanical mixer. For this series of tests, 10% water was added to the sand to obtain the optimum moisture content. An amount of 1100g of the mixture was then compacted in the compaction mold on top of a geofabric layer in 3 layers, 25 blows per layer to a density of about 1900kg/m^3 (±5%) providing a thickness of about 55mm. A surcharge load of 3kPa was applied and the mold was then filled with water and the sample was left to hydrate. After hydration, the compaction mold is placed onto the double-ring base and the top cap is positioned, fastened down and connected to a stand pipe ready for testing.

9.1.2. Test Results

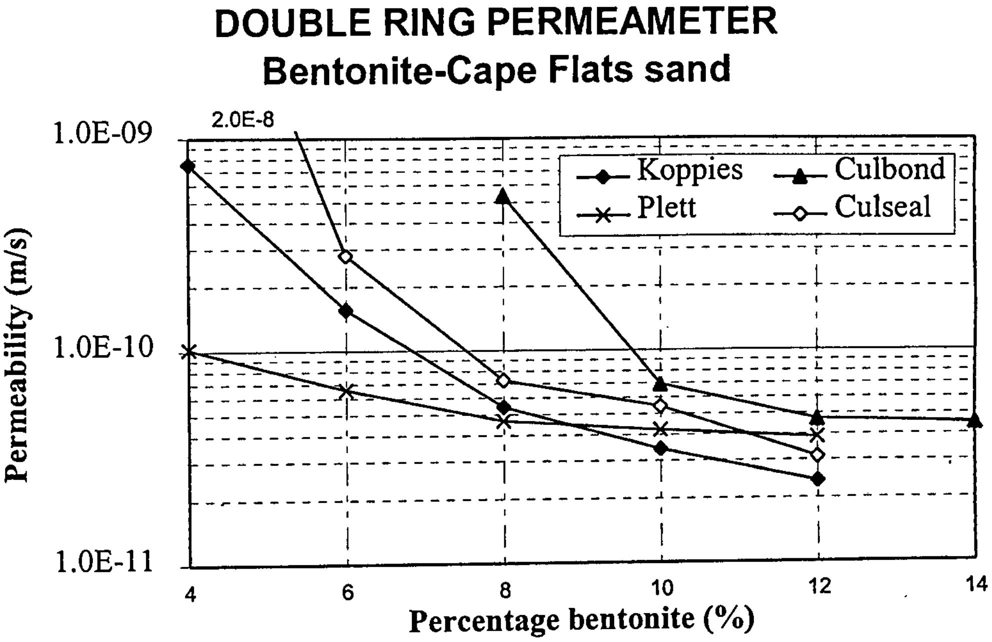
Two series of tests were conducted. The first series of tests on soil/bentonite mixes were done using Cape Flats sand, a fine grained, uniformly graded sand. The second series of tests were done with Klipheuwel sand, a well graded material. The grading curves of the two soils are presented in Appendix J.

The first series of samples were prepared using Koppies, Culseal, Culbond and Plett bentonites with Cape Flats sand. The samples were hydrated for over 180 days to insure full hydration and saturation. Each falling head test lasted between one and three days depending on the permeability. Three tests were performed on each sample and the average was taken.

The following bentonite types and mix percentages by mass were tested :

Koppies bentonite :	4%, 6%, 8%, 10%, 12 % bentonite to sand
Plett bentonite :	4%, 6%, 8%, 10%, 12 % bentonite to sand
Culseal bentonite :	4%, 6%, 8%, 10%, 12% bentonite to sand
Culbond bentonite :	8%, 10%, 12 %, 14% bentonite to sand

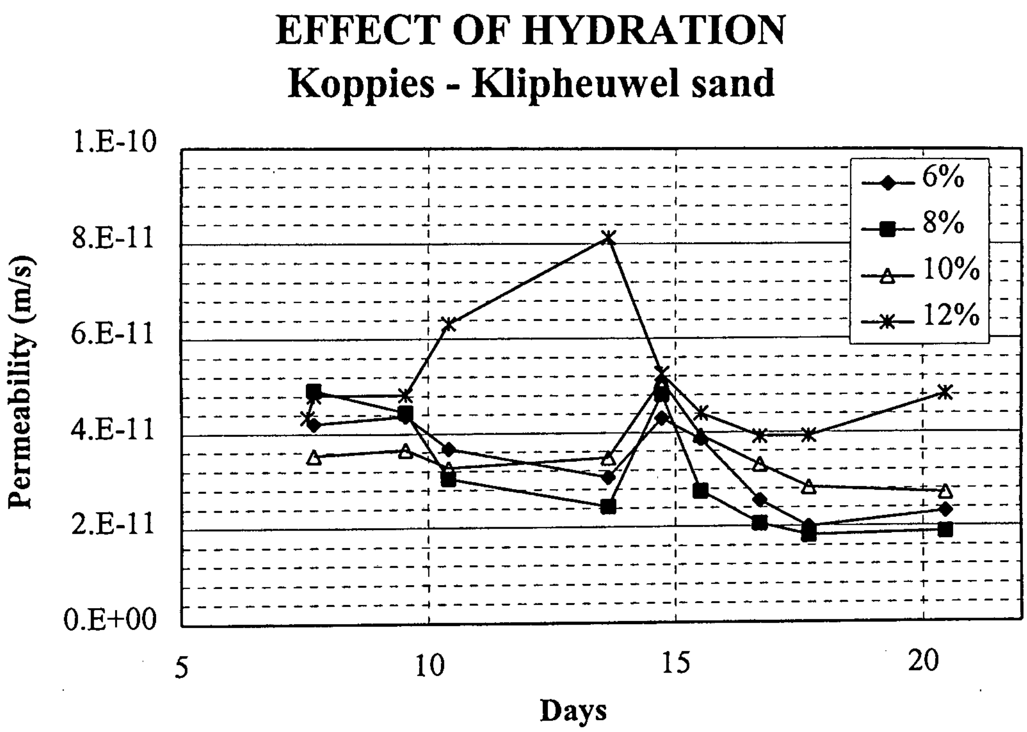
Preliminary tests indicated that Culbond showed a marked drop in permeability in the lower percentage mixes and therefore a higher percentage range was selected in an attempt to establish the optimum bentonite content. The results of the first tests series using Cape Flats sand are shown in Graph 28.



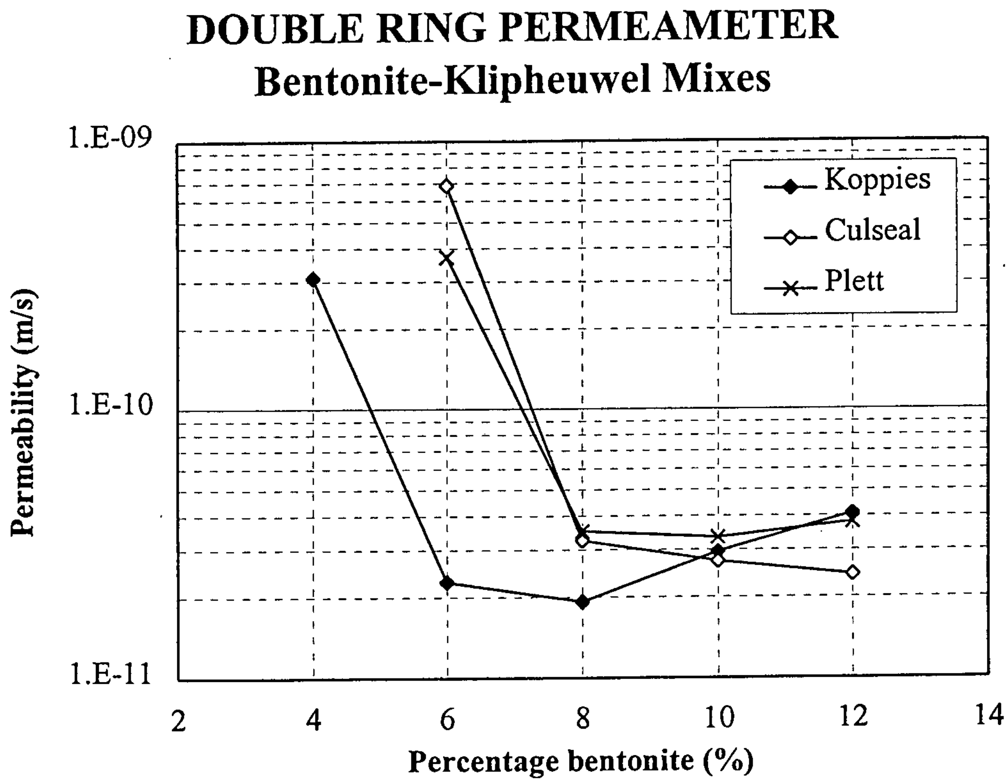
Graph 28 : Double-Ring Compaction Mold Permeameter results on the selected bentonites mixed with Cape Flats sand.

The second series of samples done with Klipheuwel sand were prepared using Koppies, Culseal and Plett bentonites. The samples were hydrated for different periods of time ranging from 7 to 14 days depending on equipment availability. This allowed for the assessment of the extent of the effect of hydration on the permeability. The first set of samples were prepared with 6, 8, 10 and 12% Koppies bentonite and were hydrated for 7 days before being tested over a period of two weeks. The effect of hydration on the Koppies-Klipheuwel mix is shown in Graph 29.

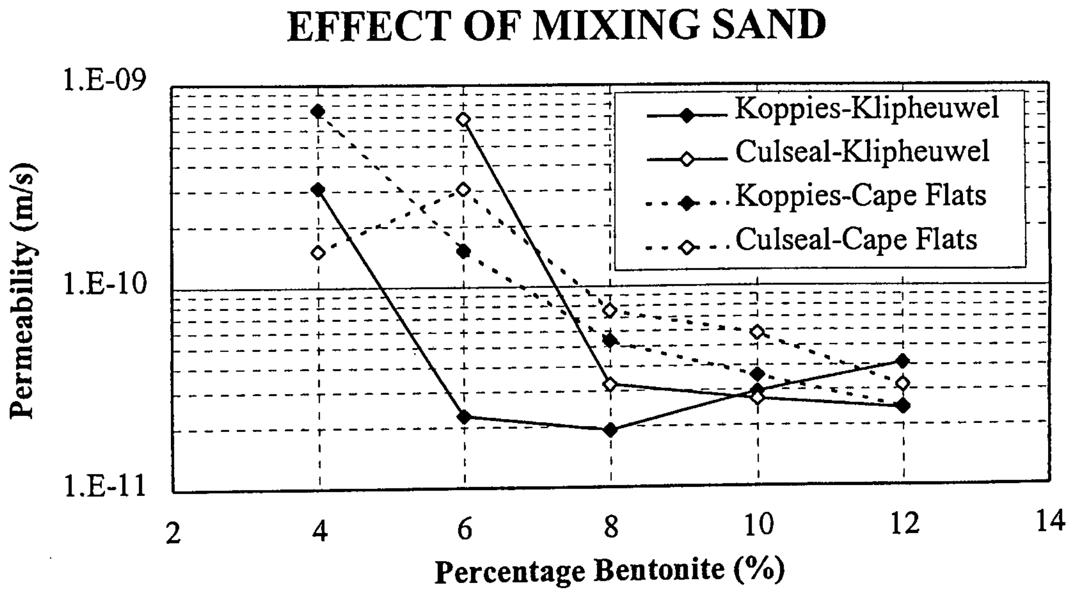
An additional sample using 4% Koppies bentonite was added to the set after noticing that the 6% mix was performing well and an even lower percentage could still be effective. The second and third sets of samples were prepared with Culseal and Plett bentonites at 6, 8, 10 and 12% contents by mass. The samples were hydrated for three weeks prior to permeability testing which lasted a further week. The permeability results taken from the last four tests once stability was reached are presented in Graph 30.



Graph 29 : The effect of the period of hydration on the permeability of various percentages of Koppies bentonite with Klipheuwel sand.



Graph 30 : Double-Ring Compaction Mold Permeameter results on the selected bentonites mixed with Klipheuwel sand.



Graph 31 : Effect of the mixing sand on the permeability of sand/bentonite mixes.

9.1.3. Discussion and analysis of results

The use of the double-ring compaction permeameter was ineffective in monitoring small sidewall leaks. The amount of permeant in most cases even after extensive test periods was almost non-existent and did not provide any control over the test. Where extensive sidewall leakage occurred, the double-ring system operated well in identifying the cause of the high permeability.

As previously noted by Chapuis (1990), Lundgren (1981), Kenny et al. (1992) and many other authors, the amount of bentonite mixed with soil had a direct effect on the permeability. In this series of tests it occurred especially in the lower percentage (4%, 6%) mixes by mass for both the Cape Flats sand and the Klipheuwel sand. As the bentonite percentage is increased in the mix, an optimum is reached which closely matches the permeability of the bentonite. For Cape Flats sand mixes, the reduction in permeability is minimal in a mix with a bentonite content beyond 10% by mass and this percentage is therefore regarded as the optimum bentonite content for all the selected bentonites (Graph 28). For the Klipheuwel sand mixes, the optimum bentonite content occurred at about 8% by mass (Graph 30). Both, Culseal and Koppies experienced increases in permeability beyond this point. The slightly higher permeability values could be attributed to sidewall leakage. Although the difference in the permeability results of approximately $2x10^{-11}$m/s is so small, it is a cause of concern and further investigation is recommended.

Washing out of the bentonite from the sand (termed 'bentonite washing') was noticed in most of the 4% and 6% mixes indicating insufficient bentonite content (Marcotte et al., 1994). If too little bentonite is used or the bentonite is insufficiently mixing with the soil, the voids within the soil structure remain open and create seepage paths, resulting in a higher in permeability.

The effect of hydration of the bentonite within the mix on the permeability was monitored on the Koppies-Klipheuwel samples. The reduction in the permeability (in the order of $2.0x10^{-11}$m/s) was minimal but noticeable in the majority of the samples over the three weeks test period, as shown in Graph 29. The 12% by mass mix produced very irregular results possibly due to a non-homogeneous bentonite distribution within the mix, however the 6%, 8% and 10% mixes followed a similar trend. The results tended to flatten out after 18 days indicating full hydration and saturation.

A further effect on the permeability of a soil/bentonite mix are the properties of the soil itself. A fine grained (clay or silt) soil and a well graded sand will require less bentonite than a course uniform sand. The effect of this is demonstrated in Graph 31. Cape Flats sand has steep grading curve with 73% of its particle size lying between 0.15mm and 0.6mm in comparison to Klipheuwel sand which is regarded as well graded with 45% over the same grain size range. At the optimum mixtures of between 8% to 10% the Klipheuwel mixes provide lower permeability coefficients, in other words a more impervious material and which requires less bentonite to achieve the same permeability coefficients resulting in a more economical mix.

In the Cape Flats sand series, Graph 28, Culbond bentonite displays in general, the highest permeability for the same percentage bentonite to soil mix compared to the other bentonite types. In the pure form this was not the case where its permeability was similar to that of Koppies. This could be due to a reaction with the sand or due to bacteria growth within the sample which was noticed in the 8% mix. Bacterial growth was also noted in the 4% and 6% Culseal mixes which showed a corresponding high permeability but the effect of the bacteria is not certain. A similar situation was documented by Edil and Erickson (1985) who reported growth of bacteria on the edges of the specimen. They noted the bacteria growth was only in a relatively small area and that the effect on the permeability was not significant. It is expected that the growth of bacteria would be minimal in-situ where a lack of oxygen would resist growth, but it further emphasizes the need to test each proposed mix properly before it is used as an impervious liner.

9.2. Consolidometer Permeameter

9.2.1. Test method and apparatus

The Consolidometer Permeameter used to test bentonite/soil mixes is the same apparatus used for testing pure bentonite. The apparatus is described in detail in Sub-section 8.1.1. The same test procedures and load increments are carried out, but the sample is prepared in a different manner. The soil/bentonite samples used in these tests were cut from the samples prepared in the compaction mold described previously. The second set of samples were first subjected to permeability testing in the double-ring permeameter before being tested in the consolidometer permeameter.

The sample was obtained by first pushing the consolidometer cutting ring approximately 35mm into the sample using a modified drill press which provided an even vertical load. The excess material was carefully trimmed from around the cutting ring and the mold was then removed. A sample was taken from the cuttings for assessing the moisture content. After final trimming of the sample, a dry powder bentonite fillet seal was placed on the base of the sample between the sample and the ring, schematically shown in Fig. 8, to limit sidewall leakage. The sample was then weighed in order to obtain the bulk density. Great care was required throughout the process to insure that the sample disturbance was kept to a minimal.

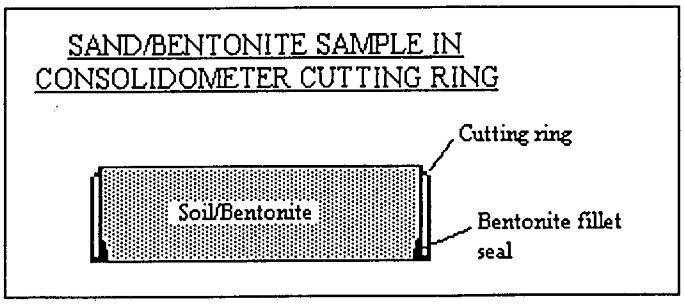


Fig 8 : Cross-section of the sand/bentonite sample prepared in the consolidometer cutting ring showing the bentonite fillet seal between the sample and the ring.

Before placing the sample into the consolidometer pot, the lower porous stone and infiltration area was filled with de-aired water so that no air would remain trapped below the sample. The cutting ring and sample was carefully positioned, fastened down and placed into the loading yolk ready for consolidation and permeability testing. The same load increments were employed in all the tests conducted in the consolidometer permeameter and the loading and permeability test sequence comprised of the following routine:

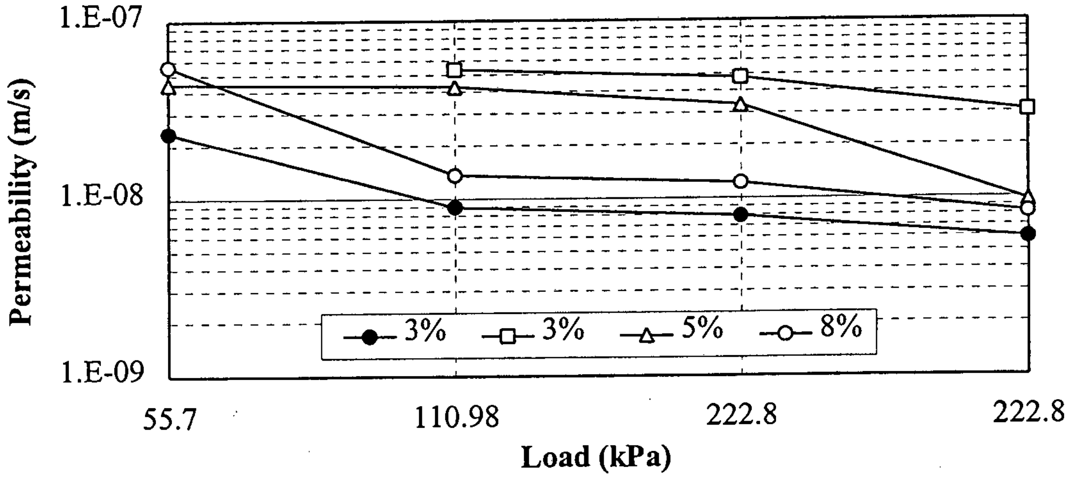
start of test
    Step 1 : Load 1 = 55.70kPa
    Step 2 : Permeability testing
    Step 3 : Load 1 = 110.98kPa
    Step 4 : Permeability testing
    Step 5 : Load 1 = 222.80kPa
    Step 6 : Permeability testing
end of test

After a period of at least 24 hours after loading, the standpipe was connected to the base of the consolidometer pot and was filled with water and allowed to permeate through the sample. No readings were taken until a stable condition was reached after which the standpipe was refilled and the falling head permeability test was started. The load was increased every three days if a satisfactory permeability test was achieved. For the samples with a higher bentonite content, about four to five days were required between the load increments to achieve at least one permeability test. Consolidation readings were taken throughout the test and at the end of the test, the moisture content was again assessed.

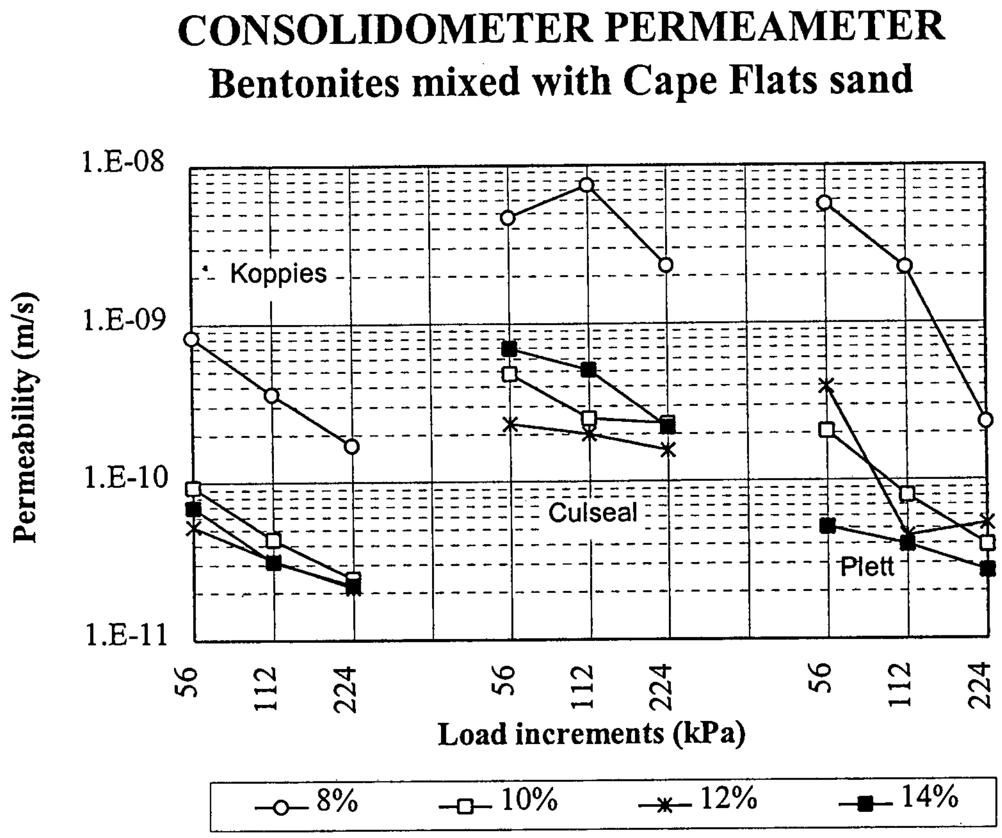
9.2.2. Test Results

The first set of samples tested in the consolidometer permeameter were prepared and tested after a relatively short period of hydration ranging from 8 to 40 days. These samples were not tested in the double-ring permeameter as this equipment was in the process of being manufactured and was not available as yet. The samples with the higher bentonite contents were tested last and therefore had the longest hydration period. The soil used in the soil/bentonite mix was Cape Flats sand at a moisture content of 10%, details of which are provided in the previous section. Koppies, Culseal and Plett bentonite were used at a concentration of 8%, 10%, 12% and 14% by mass. The bentonite fillet used to limit sidewall leakage was employed throughout this test series. The preliminary test results which motivated the idea of using a bentonite fillet for limiting sidewall leakage are shown in Graph 32. The results of the first set of soil/bentonite mixtures done with Cape Flats sand are presented in Graph 33. A typical test data spreadsheet is presented in Appendix K.

CONSOLIDOMETER PERMEAMETER
Preliminary Test Series



Graph 32 : Consolidometer Permeameter preliminary tests done without a bentonite fillet on mixes of Koppies bentonite with Cape Flats sand.

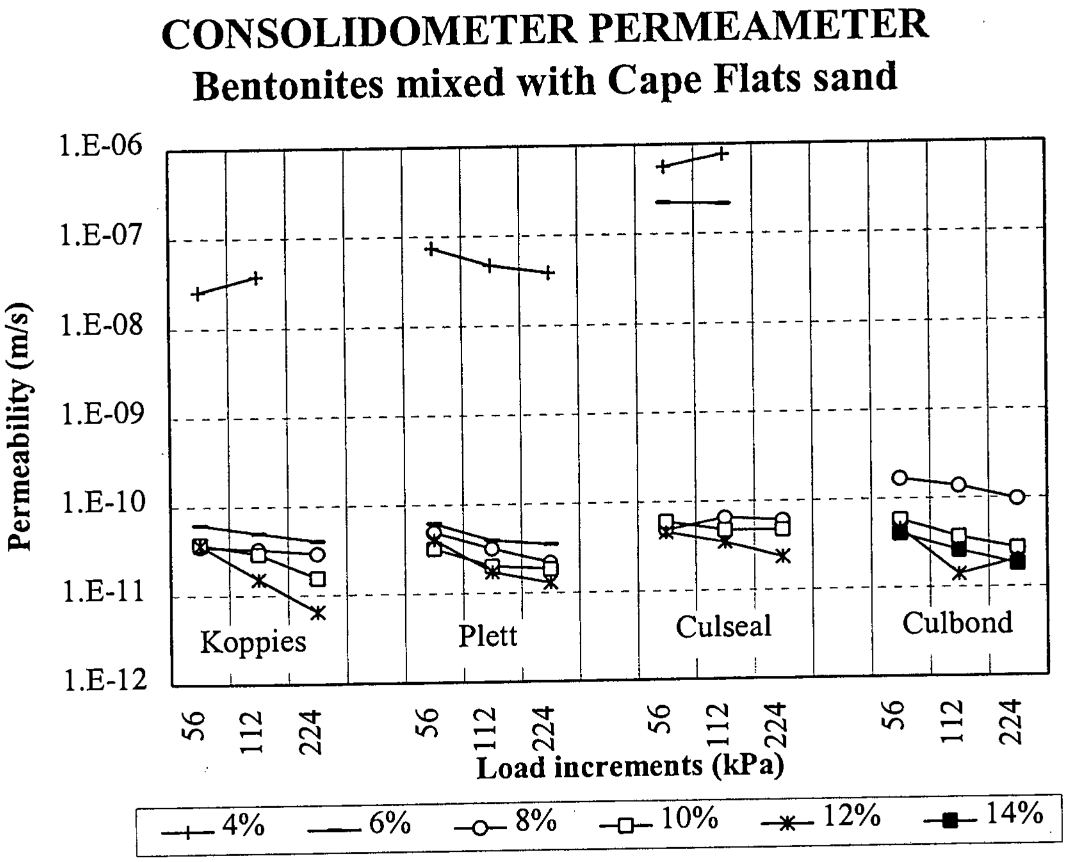


Graph 33 : Consolidometer Permeameter test results done on different percentage mixes of the selected bentonites with Cape Flats sand after relatively short hydration periods of between 8 and 40 days.

The second set of soil/bentonite mixes tested were again prepared with Cape Flats sand. The samples were allowed to hydrate for extensive periods of time ranging from 209 to 400 days. The samples were first tested in the double-ring permeameter before they were cut from the compaction mold and re-tested in the consolidometer permeameter. The bentonites used were Koppies, Plett, Culseal and Culbond and the mix percentages are shown in Table 10. The different percentage bentonite contents by mass for the various bentonite types was done in an attempt to optimize the range over which an optimum bentonite content could be expected. The results of the consolidometer permeameter permeability tests are presented in Graph 34.

Bentonite Type	Percentage bentonite mixed with Cape Flats sand					
	4%	6%	8%	10%	12%	14%
Koppies	•	•	•	•	•	
Plett	•	•	•	•	•	
Culseal	•	•	•	•	•	
Culbond			•	•	•	•

Table 10 : Summary of soil/bentonite mixes of the selected South African bentonites with Cape Flats sand.



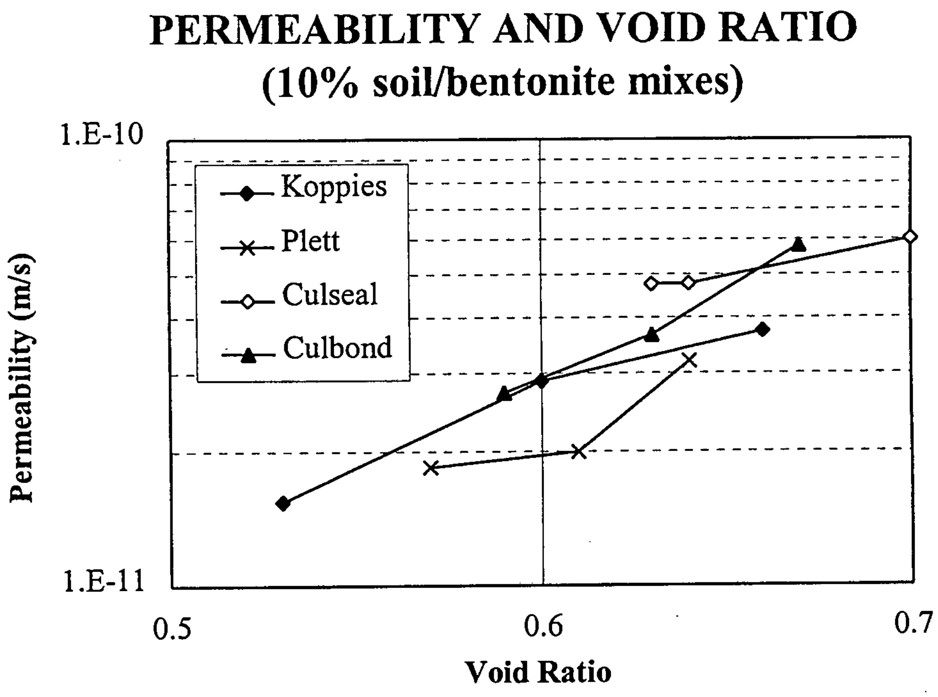
Graph 34: Consolidometer Permeameter test results done on different percentage mixes of selected South African bentonites with Cape Flats sand after hydration periods of over 200 days.

9.2.3. Discussion and analysis of Results

The conceptual understanding regarding groundwater flow in soils is a reduction in permeability due to an increase in confining stress. Soil/bentonite mixes are no exception and exhibit in most cases a marked decrease in permeability with an increase in confining stress. An increase in vertical stress results in a decrease in void ratio and authors like Mollins et al. (1996), Kenney et al. (1992) have related void ratio to hydraulic conductivity and have displayed a direct relationship between the values. The Kozeny-Carmen permeability equation stated in Section A, Chapter 2.4.1, indicates that the relationship tends to linearity at large void ratios (Mollins et al., 1996). It is noted here that the void ratio is related to the clay (bentonite) within the mix and not to the void ratio of the mix as a whole and therefore void ratios of far greater than one (e_{c}.=5) can be expected due to the high swelling ability of the clay material. Chapius (1990) correlates permeability and porosity and indicates that there is no direct relationship between the two values and only a general trend appears to exist.

Referring to Graphs 32, 33 and 34, a reduction in permeability with an increase in stress is experienced in all of the tests (except for 3 of a total of 35 tests). Two of the tests in which a reduction was not experienced were with the 4% mixes, both of which experienced sidewall leaks and washing of bentonite and are only shown for completeness. The third, non-conforming test was that of the 8% Culseal mix which produced a lower permeability under the 56kPa load than under the 112kPa load, although a reduction in permeability was again experienced under the 224kPa load increment.

A comparison between void ratio and permeability has been done on the second set of 10% soil/bentonite mixes tested. The results are shown in Graph 35. Although there may not be an exact relationship between the logarithm of permeability and the void ratio, a near logarithmic trend for all the bentonites types can be observed.



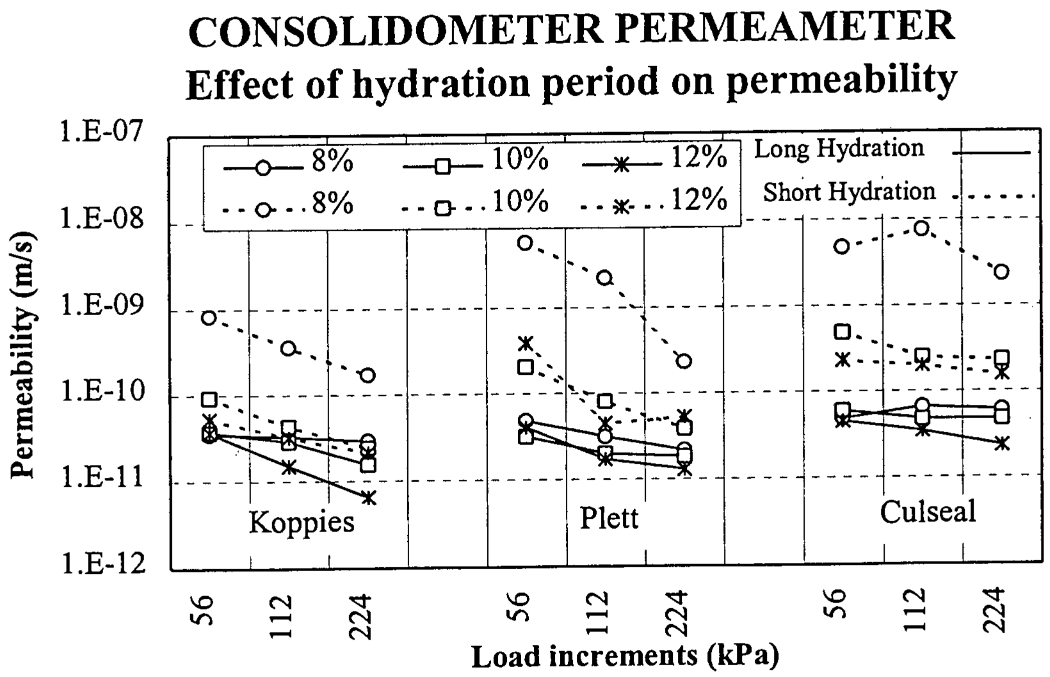
Graph 35 : Comparison between permeability and void ratio on the 10% mixes (by mass) of the selected bentonites with Cape Flats sand.

As experienced in the double-ring compaction mold permeameter, the percentage of bentonite within the mix greatly affects the permeability especially in the lower percentage mixes. Both, the first and second set of tests presented in Graph 33 and Graph 34 indicate that an optimum bentonite content is obtained at between 8% and 10% bentonite content by mass and the decrease in permeability above this percentage is minimal. At a 6% bentonite content and below, washing of the bentonite was experienced indicating insufficient bentonite which results in open voids providing drainage paths within the soil matrix. It is emphasized that in situations like these, the washed bentonite may clog the permeameter filter system and result in false permeability readings. As previously mentioned, the reduction in permeability in mixes higher than optimum, in most cases tends to be negligible. This is especially noticeable in the samples with shorter hydration periods (Graph 33) where in some cases the 12% mixes produces equal or lower permeability values than the 14% mixes.

Although the hydration periods for the first set of mixes were between 8 days and 40 days, the permeability measured in all of the samples is higher than those tested in the second set after an extensive hydration period of over 209 days. The variations in permeability are shown in Graph 36.

The Culseal bentonite mix samples showed the greatest overall improvement in permeability due to full hydration with a decrease in permeability of approximately one order of magnitude. As previously mentioned, the lower percentage mixes were tested first. In the case of the first set of samples, the lowest mix was 8% and the hydration period was between 8 and 13 days. A marked decrease in permeability occurred with all of the 8% samples in the second set.

Although the remainder of the samples also experienced a decrease, the changes were moderate in comparison. The changes could only be attributed to hydration and it appears that periods well in excess of one month are required to achieve full hydration.



CONSOLIDOMETER PERMEAMETER
Effect of hydration period on permeability

Graph 36 : Effect of Hydration on the permeability of samples tested in the consolidometer permeameter.

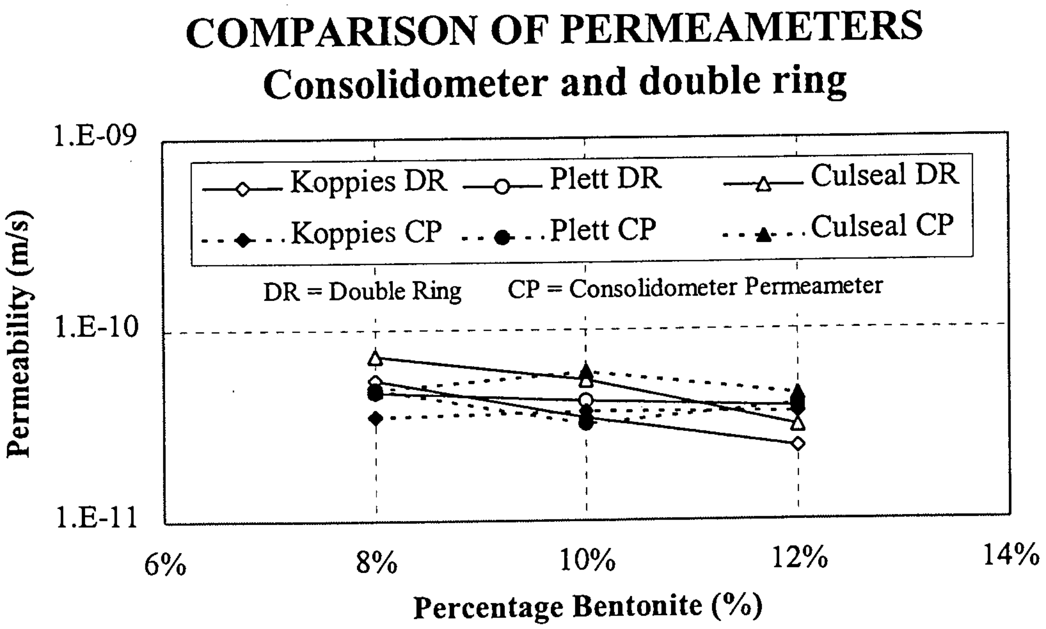
The permeability results achieved in this research study with all of the selected South African bentonites within the optimum content range closely match the hydraulic conductivity of a Black Hills Wyoming bentonite mixed with a fine sand of uniform grain size tested by Kenny et al. (1992). The 'Wyoming' 8% mix produced a hydraulic conductivity of approximately 2.1×10^{-10}m/s under a vertical stress of 60 kPa and 9.2×10^{-11}m/s under a vertical stress of 117kPa. Similarly the 12% 'Wyoming' mix produced a permeability of 6.1×10^{-11}m/s under the 117kPa vertical stress.

9.3. Discussion on Soil/Bentonite Permeability Testing

From the permeability test results of both the double-ring permeameter and the consolidometer permeameter it is clear that a decrease in permeability is experienced with an increase in the amount of bentonite added to the soil. An optimum bentonite content is reached at approximately 10% by mass for Cape Flats sand in both methods of testing and at percentages above this, little change in permeability is experienced. At bentonite contents below the optimum content permeability increases dramatically. At concentrations of 6% and below it was found that washing of bentonite occurred indicating that voids within the matrix are not filled with bentonite and flow paths exist.

Insufficient hydration was noticed in both types of permeameters. In the double-ring permeameter tests done with bentonite/Klipheuwel sand mixes showed decreases in permeability even after 18 days of hydration. In the consolidometer permeameter, the second set of tests were allowed to hydrate for over 200 days and produced marked drops in permeability (especially in the 8% mix) in comparison to the first set which were tested after short periods of hydration. It was found that periods in excess of one month are required to achieve full hydration. To obtain accurate results, a minimum of 30 days hydration should be allowed before final permeability testing. This length of time would result in excessive test periods. In the case of the double-ring permeameter, the reduction in permeability was limited, and can be expected to be approximately 2 $\times 10^{-11}$m/s lower than the 10 day permeability reading. Under larger vertical stresses in the consolidometer permeameter, a bigger drop in permeability occurred, although the drop varied considerably with the different bentonite contents. The variations are shown in Graph 36. Reductions of as much as two orders of magnitude can be expected in some cases and in others the reduction was as low as one tenth of an order of magnitude. Due to these nonlinear variations, a permeability reduction factor to limit the test duration would not be possible to calculate.

A comparison between the coefficients of permeability achieved in the double-ring permeameter and the consolidometer permeameter under the first load increment of 56 kPa are presented in Graph 37. The results used were those of the samples which had hydrated for over 200 days and the mixes selected were the 8%, 10% and 12% Koppies, Plett and Culseal bentonites mixed with Cape Flats sand. The results compare very favourably. In some cases the readings were almost identical and the largest variation in reading occurred with the 8% Culseal mix which produced a reading of only $2.4x10^{-11}$m/s lower in the consolidometer permeameter. On average the readings varied by a value of only $1.6x10^{-12}$m/s with the double-ring permeameter producing the lower readings. Although it was expected that the consolidometer permeameter would produce lower permeability results even under the lowest vertical stress of 56kPa, the double-ring permeameter produces lower permeability readings under a marginal vertical stress of 3kPa. The likely reason for this is that the samples are slightly disturbed when cutting them into the consolidometer loading ring. Slight cracks may occur within the sample and along the sidewalls and with an application of higher stresses, these are forced closed and this would in-turn result in lower values of permeability.



Graph 37 : Comparison of permeabilities of sand/bentonite mixes tested in the Consolidometer Permeameter and the Double-Ring Compaction Mold Permeameter.

10. Consolidation

During all of the consolidometer permeameter tests, changes in volume of the samples were measured firstly to establish the exact height of the sample for permeability calculations, and secondly to provide information on the effects of the bentonite content on consolidation. Volume measurements were undertaken on both the pure bentonite samples and on the soil/bentonite mixes.

10.1. Test method and apparatus

The apparatus used for testing volume change in soils is known as the consolidometer and is the same equipment used to test permeability under a vertical confining stress. The test method and apparatus used are described in detail in Section 8.1. The changes in volume were monitored with a dial gauge and readings were taken just before and just after each load increment and thereafter on a daily basis. Readings were also taken at the start and at the end of each permeability test.

The soil/bentonite samples were all pre-hydrated and were saturated almost immediately after applying the first load increment. The pure bentonite samples were prepared in the dry state, and therefore reacted differently than the soil/bentonite mixes under initial saturation. The pure bentonite sample preparation is explained in detail in Section 8.1.1. In briefly, it consisted of a 5mm layer of bentonite placed above 15mm of Cape Flats sand. The samples were allowed to settle for 15 minutes under the first load increment of 55.7kPa before being saturated. Due to the modifications of the consolidometer for permeability testing, saturation was done via the bottom porous stone in the upwards direction. This allowed the sand to saturate first before the upper layer of bentonite got wet and therefore no air would be trapped in the sand below the bentonite layer. The same consolidometer test was also performed on the Cape Flats sand in order to ascertain its contribution to the consolidation. The results of the consolidation of the sand under the three load increments were used as a correction factor to calculate the consolidation of the pure bentonite.

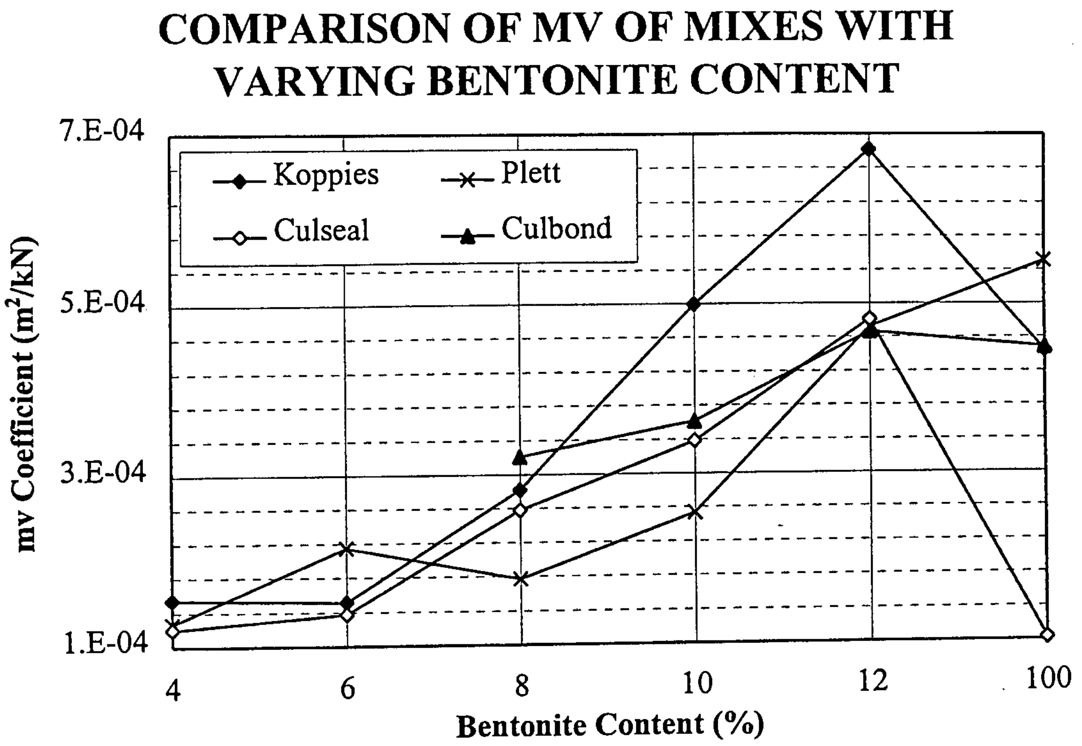
10.2. Test results

The samples tested were firstly; the selected South African bentonite mixed with Cape Flats sand (the mixes are summarized in Table 10, Section 9.2.2), and secondly; the selected South African bentonites in their pure form. Tests were also conducted on the sand aggregate namely Cape Flats sand.

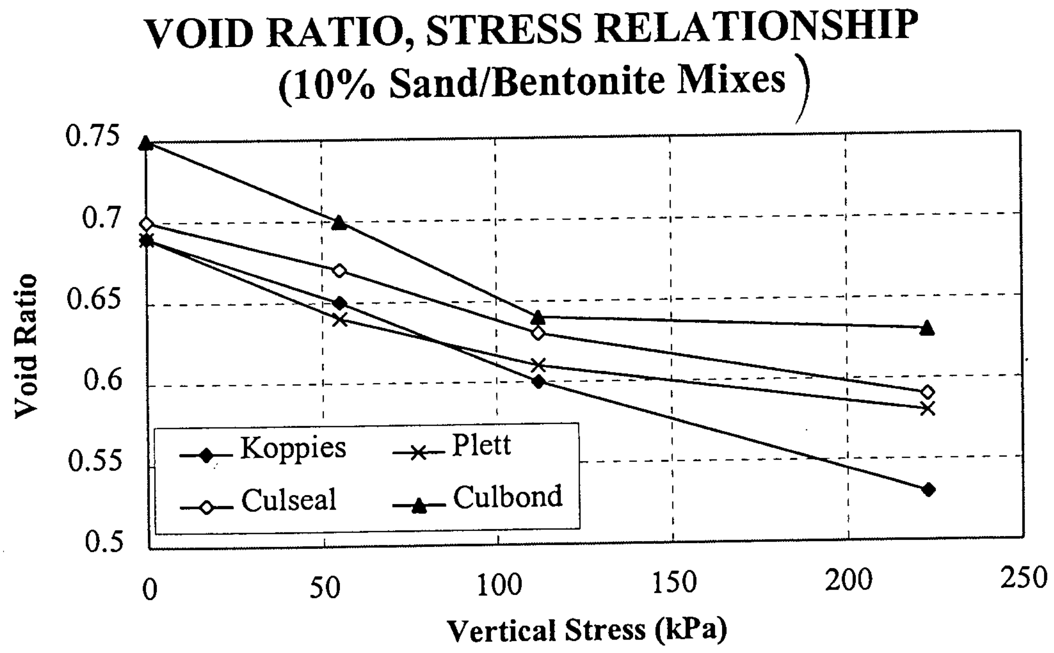
The consolidation of the samples tested was converted to a void ratio for each load increment. The gauge readings used to calculate the void ratios were the readings taken at the end of the load increment period, i.e. just before reloading. The final void ratios at each load increment were used to calculate the coefficient of volume compressibility (mv) for the load increment from 56kPa to 112kPa and then from 112kPa to 223kPa. The average of these values for the selected bentonites at different mix percentages with Cape Flats sand is presented in Graph 38. The results of the mv values for the selected pure bentonites are also included in Graph 38 (see 100% bentonite content).

Typical void ratios of sand bentonite mixes at the applied vertical loads used to calculate the mv values shown in Graph 38 are presented in Graph 39.

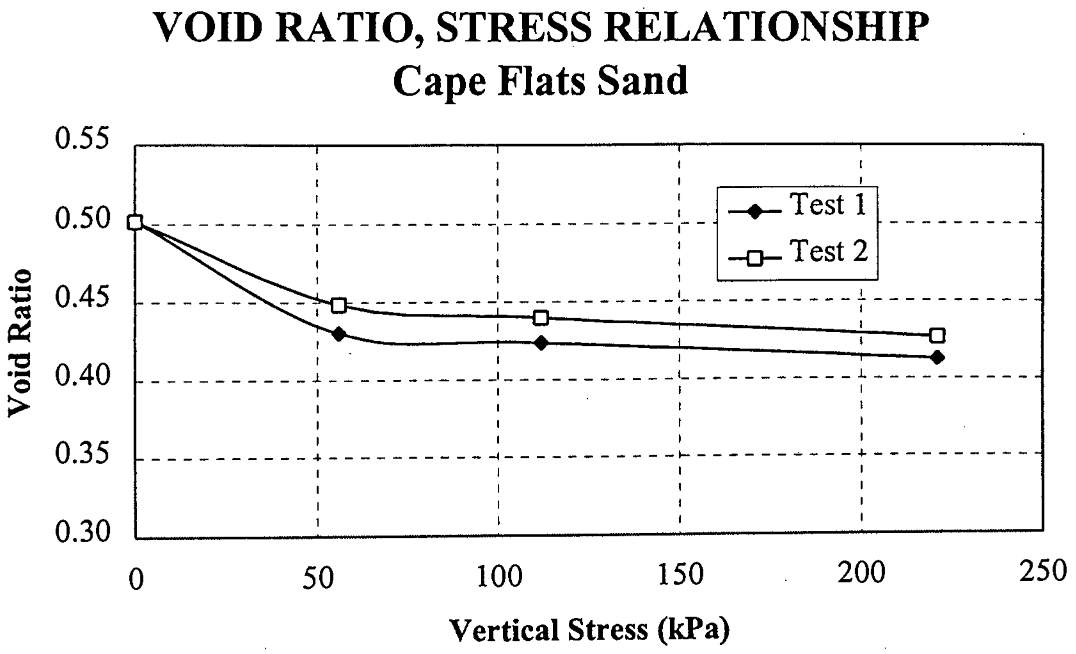
Two tests were performed on Cape Flats sand in order to obtain layer coefficients for the calculation of the consolidation of the thin bentonite layer in the pure bentonite tests. A summary of the results of these tests are shown in Graph 40. The average results of the void ratios at each load increment for the pure bentonite samples is shown in Graph 41.



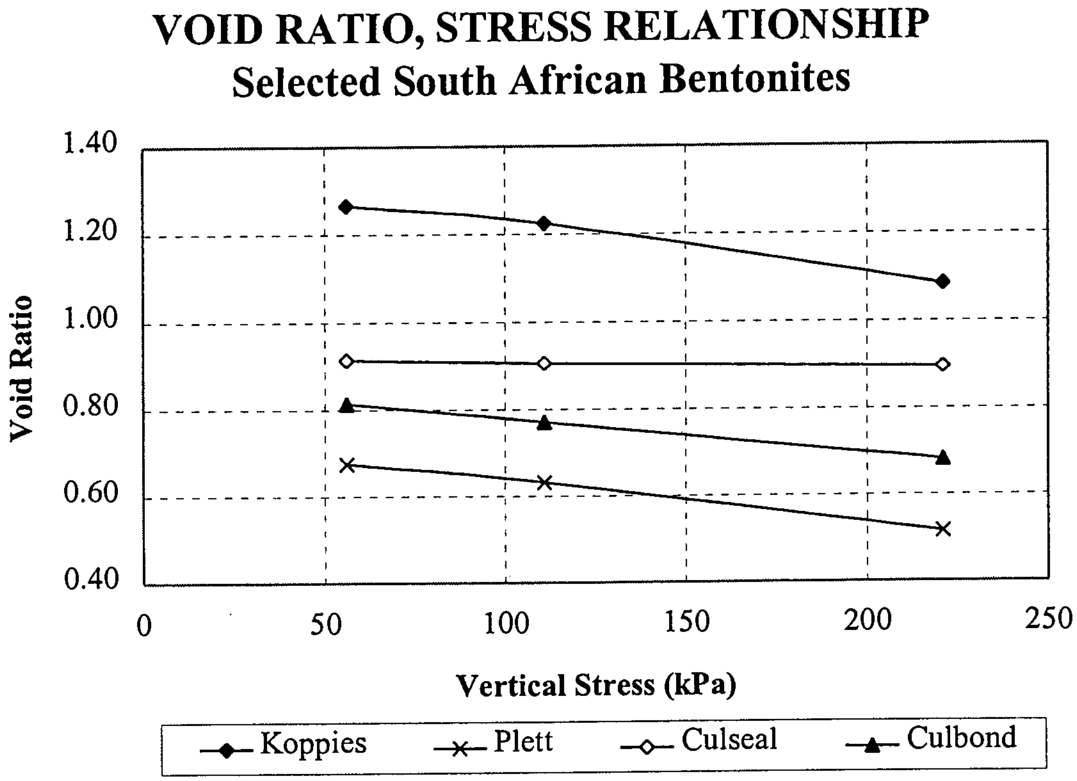
Graph 38 : Comparison of the Coefficient of Volume Compressibility (mv) with varying contents of the selected bentonites mixed with Cape Flats sand.



Graph 39 : Typical void ratios at the applied vertical load increments for the selected bentonites mixed at a 10% concentration with Cape Flats sand.



Graph 40.: Void Ratio - effective stress diagram of Cape Flats sand.



Graph 41 : Void ratio - effective stress diagram of the selected bentonites.

10.3. Discussion and analysis of results

It was found that in the sand/bentonite mixes in general the void ratio reached stability after 48 hours and would not drop further than 0.02mm within the following 96 hours in the consolidometer. For the pure bentonite samples this was not the case, no fixed condition common to all the types of bentonite tested was encountered due to the varying effects of hydration and swell of the bentonite on the layer thickness.

A clear relationship was found to exist between the percentage of bentonite in a mix and the coefficient of volume compressibility (mv). As displayed in Graph 38, the mixes with the higher bentonite content provided a higher mv coefficient, although the differences in values are small. The 6% by mass sand/bentonite mixes provided an approximate mv coefficient of $1.5x10^{-4}$ m^2/kPa in comparison to the 12% mixes which prvided an approximate mv coefficient of $4.5x10^{-4}$ m^2/kPa. These coefficients are related to a constraint modulus (Es) of 6MPa for the 6% mixes and 2MPa for the 12% mixes. These are both typical values of a 'very soft' clay. Under the load conditions applied, the change in void ratio was approximately linear in most of the mixes. This is demonstrated in Graph 39 for the 10% bentonite content mixes. The pure bentonites performed quite differently.

The void ratios of the pure bentonite did not follow a typical response of reduction of void ratio with time after the application of a vertical confining stress. An initial consolidation occurred with the first load increment of 56kPa before saturation. After saturation, all of the samples produced negative consolidation i.e. swell. On applying the next load of 111kPa, the samples showed signs of only slight settlement and in some cases almost no change in volume neither positive nor negative occurred. In these cases, the settlement was equal to the swell and if an mv value was calculated, it would be close to zero, a typical value for rock. Although for pure bentonite, the value would be highly sensitive to changes in stress and moisture content. When subjected to the third load increment of 223kPa most of the bentonites showed clear signs of consolidation. A great deal more testing is required to establish these trends and to predict the behavior of pure bentonite under various stresses.

In comparison to the consolidation of the soil/bentonite mixes and the pure bentonite samples, the Cape Flats sand reacted quite differently. As shown in Graph 40, about 75% of the settlement occurred during the first load increment, the sand material obviously being close to its densest state of packing.

11. Conclusions

Based on the available literature and the results of the tests conducted on the South African bentonites from the deposits at Koppies, Plettenberg Bay and Heidelberg, the following conclusions have been drawn :

11.1. Chemical Analysis

Although differing chemical analysis of the selected bentonites were presented by Jones (1993), Grim and Güven (1978), SAMREC and by tests performed within this research project, the factors of importance to engineering quality (permeability and swell) namely the MgO content and the SiO_2:Al_2O_3 ratio did not vary considerable between the products and the following was determined :

a) The selected bentonites all posses MgO contents representative of a good quality bentonite. Koppies posses the highest content, Culbond the second highest and Plett the lowest of the three selected bentonites.

b) The SiO_2:Al_2O_3 ratio indicates that Culbond and Culseal are regarded as medium quality bentonite (Reschke and Haug, 1991).

c) All of the bentonites tested were found to posses either feldspar or quartz as an impurity but the amounts were very small.

11.2. Particle analysis

Specific surface tests and a particle size analyses were conducted on dry Koppies, Plett and Culbond using the nitrogen adsorption method, a Sedigraph and an electron microscope analysis. The following was noted:

a) The specific surface area of the bentonites tested in the unhydrated state were: Koppies 38.9 m^2/g, Plett 24.7m^2/g, Culbond 29.7m^2/g.

b) The nitrogen adsorption method provided an indication of the dry particle size which is related to the processing of the bentonite. However, the method does not give a clear indication as to the total surface area available for hydration which is in the order of 800m^2/g (Komine and Ogata, 1996)

c) The Sedigraph analysis provided a particle grading from 100μm down to 20μm and it was found that 87% of Koppies,73% of Plett and 48% of Culbond particles are smaller than 20μm.

d) An analysis at a magnification of 40,000 showed that all of the structures of the selected bentonites are of the compact lamellar type. Koppies displayed a clear planar habit indicative of a low crystallization rate. Areas of the Plett bentonite particles appeared to be highly foliated indicating loss of surface charge which would result in minimal hydration and low swell in these areas. The compact lamellar structure of Culbond was not as clear as in the other samples and small granular particles were present.

11.3. Specific Gravity

Performing standard specific gravity tests on pure bentonite was found to be an almost impossible task due to the high cohesiveness and low permeability making dispersion almost impossible. After extensive saturation under vacuum, results were obtained for the selected bentonites. The S.G. tests on soil/bentonite mixes were easier to disperse. The results were as follows:

a) An average of three tests on each sample of the pure bentonites resulted in an S.G. for Koppies of 2.83, Plett 2.58, Culseal 2.58 and Culbond 2.61.

b) The specific gravity tests conducted on soil/bentonite mixes varied from 2.61 to 2.74 and appeared to decrease with an increase in bentonite content which was expected for Plett and Culseal due to the sand possessing a higher S.G. The opposite was expected for Koppies due its S.G. being higher than that of the sand, but the results were erratic and no clear trend could be established.

11.4. Atterberg Limits

Atterberg limit tests were conducted on Koppies, Plett and Culbond. The liquid limit determination done using both the Casagrande device and the 'fall cone' method was done on Koppies and Plett to correlate the differences. The plastic limit was carried out in the standard way (BS 1377). The following results were obtained:

a) The plastic index using the Casagrande method for Koppies was 407%, Plett 320% and Culbond 100%.

b) The plastic index using the 'fall head' method for Koppies was 406% and Plett 164%. The difference between the Plett results is uncertain but it is expected that the thixotropic properties of bentonite would adversely affect the results of the 'fall cone' method.

c) On Casagrande's Plasticity Chart, Koppies, Plett and Culbond fell well within the highly plastic clay zone although Culbond did not plot close to the 'B-line' which is not typical of a montmorillonite clay.

11.5. Swell Testing

A number of undisturbed swell tests were conducted on samples of the selected bentonites under various conditions. Tests were first conducted under no confining pressure and are referred to as 'free swell' tests and then the samples were tested under varying confining pressures. The following was observed :

a) Depending on the thickness of the sample tested, large variations in free swell were obtained. It was found that as the layer was increased from ±4mm up to ± 20mm the final swell varied from about 600% to 100 %. A swell index was therefore developed which eliminates the effect of thickness and is termed the 'Zero Thickness Free Swell Index'. This index is also useful for dispersive materials. Like some bentonites, dispersive soils stay in suspension after mixing and cannot provide a result for a normal swell index test which requires full settlement of the material.

b) The results of the Zero Thickness Free Swell Index for the bentonites were: Culseal 681%, Culbond and Koppies 378% and Plett 273%.

c) The effect of only a minor surcharge of 25Pa on the bentonite samples caused a reduction in swell of almost a third, and the effect of a hydraulic head of around 2m reduced the swell by almost half of the free swell values.

d) Under larger vertical confining stresses of 111kPa in the consolidometer, the final swell of Koppies was 37%, Culseal 20% and Culbond 10%. Plett experienced no swell at all. These values decreased even further under increase loading.

11.6. Permeability Testing of Pure Bentonite

Testing the permeability of pure bentonite requires time and a number of test apparatus to achieve a sufficient number of results. Tests were conducted in the consolidometer permeameter, Triaxial cell, ASTM standard permeameter and a newly developed cost effective permeameter called the Mini Permeameter. The observations made and results achieved were as follows :

a) Test periods in the consolidometer permeameter were dramatically reduced by testing only a 5mm thick layer of bentonite. This was achieved by filling the remainder of the cutting ring with porous sand incorporating a bentonite layer around the edge to prevent sidewall leakage.

b) Reduction in permeability due to an increase in confining pressure was noticed in all of the tests especially in the consolidometer permeameter where coefficients of permeability dropped by about one order with an increase in vertical stress from 56kPa to 223kPa.

c) Under high confining pressures the coefficients of permeability were very similar for the bentonites tested. The coefficients obtained under a confining stress of 221kPa were; Plett $2.5x10^{-12}$m/s, Culbond $3.5x10^{-12}$m/s, Koppies $5.0x10^{-12}$m/s and Culbond $6.0x10^{-12}$m/s.

d) Samples of 5mm thick were permeated in the triaxial apparatus. It took about 5 days for the sample to saturate and for the permeant to reach the flow measuring apparatus. Only small variations in permeability occurred between the different types of bentonite and almost no reduction in permeability was noticed with an increase in the swell pressure from 100 kPa to 200 kPa. The coefficients of permeability achieved under both pressures for Koppies was $2.44x10^{-11}$m/s, Culbond $4.17x10^{-11}$m/s and Plett $4.62x10^{-11}$m/s under 100 kPa and $3.22x10^{-11}$m/s under 200 kPa.

e) A Hydraulic gradient of 800 was used in the triaxial apparatus and ranged between 360 and 200 in the consolidometer permeameter. The permeability results in the triaxial apparatus are one order lower than those in the consolidometer. The discrepancies are due to the large differences in the hydraulic gradient.

f) The ASTM standard permeameters were used to test the effect of swell on permeability. The test was limited to Plett and Koppies bentonite and the swell increments were limited to three. Both samples showed a defined increase in permeability with an increase in swell of 100%. On average Koppies produced coefficients of permeability of $3.0x10^{-11}$m/s rising to $6.0x10^{-11}$m/s after swell and Plett registered $5x10^{-11}$m/s before and $1x10^{-10}$ m/s after swell.

g) The Mini Permeameter is an adaptation of the standard permeameter developed as an economical and practical alternative for testing pure powered bentonite. Its advantages are size and simplicity making it easy to construct and operate for extensive periods of time. Permeability tests under free swell conditions especially with thin layers of bentonite produced irregular results. The permeability coefficients achieved with thicker layers of between 10mm and 25mm of bentonite and with the addition of a light surcharge load produced more consistent results. Typical permeability results achieved 20 days after the first permeant was noted under these conditions are ; Koppies $6.0x10^{-11}$m/s, Culseal $7.5x10^{-11}$m/s, Plett $8.0x10^{-11}$m/s, and Culbond $9.0x10^{-11}$m/s. It was found that the permeability continues to drop for at least a further 120 days and it is predicted that the final value will be about 60% of the value achieved at the 20 days period.

h) An insufficient filter layer was initially used in the Mini Permeameter and an increase in permeability was noticed. It is suggested that with insufficient drainage, the pore water pressure within the bentonite increases resulting in greater swell which in turn creates a less dense material of higher permeability.

i) The effect of the test apparatus and test conditions greatly affect the permeability results of pure bentonite. The variations are mostly the result of vertical stress differences within the apparatus. Although the results of the different methods are comparable, tests which simulate in-situ conditions best, should be used wherever possible.

11.7. Permeability Testing of Soil/bentonite Mixes

Due to practical, structural and economical considerations, the use of bentonite mixed with soil to form an impervious layer is favoured to a pure bentonite layer. Mix proportions are critical to achieve a low permeability and extensive testing was done in this direction using a double-ring compaction mold permeameter and a consolidometer permeameter. The observations and test results were as follows :

a) The double-ring base system of the compaction mold permeameter did not provide any significant monitoring of minor side wall leaks due the small amount of permeant passing through the sample, but clearly showed large variations caused by extensive leakage.

b) The amount of bentonite mixed into a sand has a direct effect on the permeability up to an optimum whereupon an increase in bentonite content does little to improve permeability. The grading of the sand also has a direct effect on the optimum bentonite content and the type of bentonite effects the permeability of the mix.

c) Tests conducted in the double-ring compaction mold permeameter showed that for Cape Flats sand the optimum bentonite content was in the order of 10% by mass providing a coefficient of permeability of between $3.5\text{x}10^{-11}$ m/s and $7.0\text{x}10^{-11}$m/s depending on the bentonite type. For Klipheuwel sand the optimum bentonite content was about 8% by mass and permeability coefficients ranged from $2.0\text{x}10^{-11}$m/s to $3.5\text{x}10^{-11}$m/s depending on the bentonite type.

d) Permeability coefficients increased dramatically below 6% by mass with visible washing of bentonite occurring indicating that the voids within the mixing soil were not filled sufficiently with bentonite.

e) As the bentonite hydrates the permeability decreases. It was noticed that even after 18 days of testing the permeability values continued to drop and it is suggested that a period of at least 30 days hydration should be allowed before an accurate permeability coefficient measurements can be taken.

f) The coefficients of permeability achieved in the double-ring compaction mold permeameter were almost identical to those achieved in the consolidometer permeameter under the first load increment of 56 kPa.

g) Typical permeability coefficients for the selected bentonites mixed at a concentration of 10% by mass with Cape Flats sand after complete hydration and under a 56 kPa confining stress were : Plett 3.2×10^{-11}m/s, Koppies 3.7×10^{-11}m/s, Culbond 5.8×10^{-11}m/s and Culseal 6.0×10^{-11}m/s.

h) Permeability tests conducted in the consolidometer permeameter showed that in general the permeability decreased with an increase in stress. In the case of Koppies, Plett and Culbond bentonites the permeability dropped by approximately half an order of magnitude with a stress increase from 56kPa to 224kPa. Culseal, however, showed no significant decrease.

11.8. Consolidation

Changes in volume due to an increase in the vertical stress were monitored during permeability testing of pure bentonite and soil/bentonite mixes in the consolidometer permeameter. The following was noted :

a) On application of a load, the soil/bentonite mixes generally reached stability after 48 hrs whereas the pure bentonite samples showed erratic movements in terms of compression and swell, and no pattern could be established.

b) A clear relationship exists between the coefficient of volume compressibility (mv) and the percentage (by mass) of bentonite in a mix. It was found that the higher percentage mixes provided higher mv coefficients although the difference was small. Typical mv values for a 6% mix were in the order of 1.5×10^{-4}m/s and for a 12% mix in the order of 4.5×10^{-4}m/s.

A wide variety and variation of geotechnical laboratory tests have been conducted on a few selected South African bentonites. From the results it is clear that only general trends and typical behavioral patterns could be established, which satisfied the aims of this research. A great deal more research of the Engineering Properties of South African Bentonites is needed to achieve a proper understanding of these materials, and it is hoped that this thesis will form the base of the future research.

Section C Guidelines for designing Bentonite Liners

12. Introduction

Bentonite is a natural clay material which possesses very low permeability. Once extracted from the ground, it is dried and ground into a fine powder. When exposed to water the dry bentonite hydrates and swells forming a viscous gel with a coefficient of permeability of $2.5\text{x}10^{-12}$m/s and lower under certain circumstances. This makes the product a viable option as a natural impervious liner for a number of applications in the field of Civil Engineering.

Presented here are a number of guidelines which were based on this research work to assist in the selection, design and construction of an impervious liner using bentonite as the primary material to limit seepage.

13. Selection

There are a number of options available to the designer when choosing an impervious liner. Underlying the choice are the permeability requirements for the particular site which can range from zero to a semi-permeable material of around $1.0\text{x}10^{-7}$m/s. In addition to the primary liner, a secondary liner may also required as a fail safe mechanism on more hazardous waste containment applications. The typical liner options range from concrete, HDPE (or geomembrane), asphalt, normal clay and a variety of bentonite liner systems (Breitenbach, 1994).

Bentonite is a natural material and although it does allow liquid to pass through it, the volumes are almost undetectable. It is therefore ideal as a primary liner material in dams, ponds, landfill caps, low hazardous applications and as a secondary liner in high hazard containment applications. It becomes a viable option when clay is not found close to the site and/or the permeability of the local material is too high. Bentonite liner systems are divided into three main categories namely : pure bentonite liners, geosynthetic clay liners and soil/bentonite mix liners.

13.1. Pure Bentonite Liners

This liner system consists of a layer of pure powder (or granular) bentonite ranging in thickness depending on the containment requirements, but is usually about 50mm to 100mm thick. It offers the advantage of very low permeability and the layer remains highly plastic when wet allowing for movement without loss of integrity. The main problem with pure liners is the application. The material has very low shear resistance (internal angle of friction = 9°) and is therefore limited to shallow slopes (Daniel, 1993). Laying of the material in a uniform manner on site is almost impossible due to the fine powdery consistency of the processed product. The cost of the material does not make this application viable unless it is used as a thin layer in non-critical situations.

13.2. Geosynthetic Clay Liners

This system is used extensively in Europe, Canada and the United States of America as a secondary liner material and for capping landfill sites. It comprises of a layer of bentonite approximately 5mm thick either held between two layers of geotextile or glued to a geomembrane (Trauger, 1992). There are various products available internationally and no geosynthetic clay liners are presently manufactured in South Africa, making the imported product uncompetitive to other liner systems.

The geosynthetic clay liner offers a continuous layer of bentonite and the additional strength of the geotextile or the additional sealing of a geomembrane depending on the type. They all offer their individual advantages, but a point of contention are the products where the outer sandwich layers of geotextile are not bonded together through the bentonite resulting in the same problem of low shear resistance experienced with the pure bentonite liner.

13.3. Soil/Bentonite Mix Liners

In many containment situations the in-situ material possesses inadequate impermeability to pass as a liner material. By mixing only a small percentage of bentonite into the soil, the permeability characteristics can be greatly altered and the material can function successfully as a primary or secondary liner.

The soil/bentonite mixture acts as a composite material if designed correctly. The bentonite fills the voids which limits seepage while the structure of the soil stays intact providing load-bearing and shear strength capacities. The soil/bentonite mixture can be handled on site with conventional construction equipment and can be placed and compacted with relative ease.

Depending on the soil with which the bentonite will me mixed and the permeability requirements of the liner, the percentage of bentonite will vary from about 4% to about 15% by mass and in many applications soil bentonite becomes an economically viable alternative liner system with the advantage of being made of natural materials.

14. Design

The basis of liner design is to create a layer which posses the required coefficient of permeability for the specified application. These requirements could be laid down by a number of bodies for example in the case of landfill sites in South Africa, the Department of Water Affairs provides the requirements. In addition to permeability, the long term durability needs to be assessed and in the case of all liners, internal and overall stability requires assessment.

These guidelines are directed primarily at designing bentonite liners with regard to permeability but it must be stressed that the general geotechnical aspects must not be ignored.

14.1. Permeant

In all liners, the effect of the permeant liquid needs to be fully assessed. Natural liner materials which include bentonite are particularly prone to degradation once in contact with certain chemicals.

A saline solution for instance balances the exchangeable cations on the bentonite platelets which results in a far reduced swell and therefore higher permeability. Acidic solutions may cause breakdown of the liner material which may have a negative effect on permeability, however, if the concentrations are low, the acid may aid the sealing process due to the precipitation of small particles out of solution which could block the pores in the soil (Daniel, 1985).

All liner materials considered should be tested with the expected permeant well before installation in order to assess the full chemical interaction of the materials.

14.2. Permeability Testing

Permeability testing of the proposed liner is usually undertaken in the laboratory. Depending on the conditions under which the liner will operate, an appropriate permeability test method will be required to simulate the in-situ conditions. This is not always possible especially due to either the particle boundary conditions in the field or the extensive periods required to obtain readings which could occupy the respective equipment for weeks at a time.

14.2.1.Pure bentonite

The coefficient of permeability under the various test conditions can vary from 1.5×10^{-10}m/s to 2.0×10^{-12}m/s. For pure bentonites, a solution to permeability tests not matching in-situ conditions is that by using some other available permeability apparatus and by applying the conversion factors given in Table 1, a more realistic in-situ value can be obtained.

Table 1 : Conversion factors for various permeameters

Test Method		Mini 10g	Mini 20g	ASTM	Mini light	ASTM	Triaxial	Triaxial	Cons Perm.	Cons. Perm.	Cons. Perm.
conversion multipliers		(5mm)	(10mm)	high swell	surcharge	low swell	100kPa	200kPa	55kPa	110kPa	220kPa
		(convert from)									
Mini 10g (5mm)	c	1	1.077	1.231	1.436	1.936	2.302	2.638	5.363	11.047	18.334
Mini 20g (10mm)	o	0.928	1	1.143	1.333	1.798	2.137	2.449	4.979	10.256	17.021
ASTM high swell	n	0.812	0.875	1	1.167	1.573	1.870	2.143	4.357	8.974	14.894
Mini light surcharge	v	0.696	0.750	0.857	1	1.348	1.603	1.837	3.734	7.692	12.766
ASTM low swell	e	0.516	0.556	0.636	0.742	1	1.189	1.362	2.770	5.705	9.468
Triaxial 100kPa	r	0.434	0.468	0.535	0.624	0.841	1	1.146	2.330	4.799	7.965
Triaxial 200kPa	t	0.379	0.408	0.467	0.544	0.734	0.873	1	2.033	4.188	6.950
Cons Perm. 55kPa		0.186	0.201	0.230	0.268	0.361	0.429	0.492	1	2.060	3.418
Cons. Perm. 110kPa	t	0.091	0.098	0.111	0.130	0.175	0.208	0.239	0.485	1	1.660
Cons. Perm. 220kPa	o	0.055	0.059	0.067	0.078	0.106	0.126	0.144	0.293	0.603	1

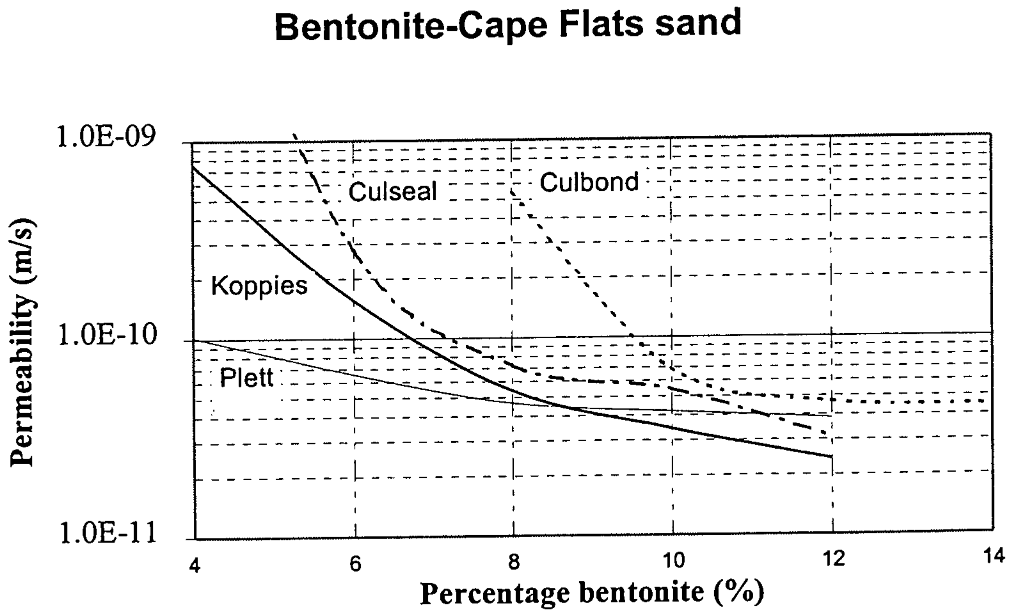
Testing of pure bentonite and soil bentonite mixes takes weeks if not months. Research has shown that under certain testing conditions of pure bentonite, realistic readings are only achieved 20 days after the first permeant appears which could take several weeks. The permeability continues to slowly decrease for a further 4 months to a value of about 60% of the permeability measured at 20 days.

Solutions to reduce testing periods are not always advantageous to the results achieved. The only way the testing period can be reduced is by increasing the hydraulic gradient across the sample. This is either done by using a higher liquid pressure or by using a thinner sample. Both are problematic, a thinner sample means a less representative sample and the effect of a weak spot creating a flow path is exasperated. A very high hydraulic gradient may result in physical changes within the sample and may cause unrealistic forcing of flow paths. Gradients of between 200 and 300 showed no adverse effects when testing bentonite in the consolidometer permeameter, but when subjected to gradients of up to 800 in the triaxial cell, the permeability results increase by approximately one order of magnitude.

14.2.2.Soil/Bentonite

The coefficients of permeability of a soil/bentonite mix are typically less than one order of magnitude higher than those of pure bentonite. The success of a soil bentonite mix is in the percentage (by mass) of bentonite. Too little bentonite will result in open voids within the soil and too much bentonite will force the soil particles apart and create a weak structure. The less the bentonite is allowed to swell the lower the permeability will be.

An optimum bentonite content is achieved when without compromising the possibility of insufficient mixing creating weak spots, a near maximum permeability is achieved. This is ascertained by testing a range of different bentonite concentrations and finding the percentage at which the permeability becomes constant. A typical set of tests ranging from 4% to 14% using different types of bentonite is shown in Graph 1.



Graph 1 : Typical test results for obtaining the optimum bentonite content of a soil/bentonite mix. (% by mass)

The percentage may change when the mix is subjected to the permeant liquid it will be retaining in its final liner form. It is therefore essential that permeability tests are conducted wherever possible with the actual permeant.

It is recommended that the soil/bentonite mix should contain a percentage of bentonite slightly higher than the optimum mix under laboratory conditions to compensate for wastage and insufficient mixing on site. For example, in Graph 1 the optimum would be between 8% and 10% for Koppies, Culseal and Plett, but the recommended mix on site would be the upper value of 10% to allow for wastage etc.

As noted previously, the permeability test should match the in-situ conditions and variations in results can be expected through different test methods. One difference here is that there are limited ways to test soil/bentonite mixes. A compacted sample is always required which limits permeameters to the triaxial apparatus, consolidometer permeameter and compaction mould permeameter. Each test apparatus has its own unique advantages and in-situ correlation.

Research has shown that for soil bentonite permeability testing under low confining stresses, a double-ring compaction mould permeameter would produce results almost identical to those obtained in a consolidometer permeameter under a vertical load of 56 kPa. Under greater loads the reduction in permeability varied considerably for the different mix percentages and bentonite types which emphasizes the need for tests that represent in-situ conditions.

14.3. Theoretical Design

Deciding on the optimum bentonite content range over which to test could be a hit and miss operation. A possible solution to this can be achieved by an initial assessment of the material to be used and the in-situ operating conditions.

Different bentonites have different swelling abilities and permeability under different conditions. For example; one bentonite may have a free swell of over 400% and a swell under a 221kPa confining stress of only 25%, whereas another could have a free swell of 300% and produce no swell under a confining stress. The coefficient of permeability in the free swollen state can be as high as 5.0×10^{-10}m/s and under confinement as low as 5.0×10^{-12}m/s.

The aim of bentonite is to completely fill the voids within the soil to create an impervious layer. To maintain the integrity of the soil, the swell pressure of the bentonite may not exceed the confining stress keeping the soil particles together. To satisfy both of these, the swelling ability of the bentonite to be used in the mix needs to be known (using the relevant permeant).

From the tests conducted on the typical South African bentonites, the swells under high confining stresses did not exceed 38%, therefore if the volume of swollen bentonite within a mix does not exceed the void space of the mix, separation of the soil particles should not occur. On the other hand, the minimum bentonite required to just fill the voids is the volume of bentonite in its free swollen state. This is demonstrated in the following example :

INFORMATION REQUIRED :

<u>Mixing Soil</u>

e.g. Cape Flats Sand

Void Ratio at expected confining stress e = 0.42 (see Appendix E)

Bulk density = 1850kg/ m^3

Bentonite

e.g. Koppies

Swell under expected confining stress = 35% (see Graph 15 sect. B 6.2.2.2)

Free swell index of bentonite = 378% (See Table 6 sect B 6.1.4)

Bulk density = 997kg/m^3

CALCULATIONS

1. The volume of voids in the soil is required to know how much bentonite is needed to fill the voids. It is convenient to work in 1m^3 units and therefore the volume of voids in 1m^3 of soil

$$= 1\text{m}^3 \text{ x } e/(1+e) = 1 \text{ x } 0.42 / (1+0.42) = \mathbf{0.30m^3}$$

A maximum swell of 10% greater than the swell under the expected in-situ confining stress is used to guarantee that no separation of the soil particles occurs. In other words, the fully swollen bentonite will not built up a sufficient swell force to push the soil particles apart.

∴ Corrected maximum swell under expected confining stress

= swell under expected confining stress x 1.1

= 35% x 1.1 = **38.5%**

The percentage of bentonite in the swollen state is converted to a percentage of bentonite in the unswollen state.

∴ Percentage unswollen bentonite

= 100% / (100% + corrected maximum swell)

= 100% / (100%+38.5%) = **72%**

DESIGN LIMITS

The **MAXIMUM** volume of bentonite allowed in the mix so as to avoid the separation of the soil particles is calculated as follows :

(volume of voids in soil) x (percent unswollen bentonite) = max. volume

= 0.30 X 72% = 0.22m^3 of bentonite per 1m^3 of soil

This is represented as a percentage by volume to the soil as :

0.22 / 1 x 100 = **22% by volume**

or as a percentage of mass as :

0.22 x 997 / 1800 = **12% by mass**

The **MINIMUM** volume of bentonite required to just fill the voids of the soil is obtained by :

(volume of voids in soil) / (free swell index of bentonite) = min. volume

= 0.30 / 378% = 0.08m^3 of bentonite per 1m^3 of soil

This is represented as a percentage by volume to the soil as :

0.08 / 1 x 100 = **8% by volume**

or as a percentage of mass as :

0.08 x 997 / 1800 = **4% by mass**

Note: 22% by volume will provide the lowest permeability, 8% the highest.

If the same sand was used with for example with Culseal bentonite, the design would be as follows:

INFORMATION REQUIRED :

Bentonite

Culseal

Swell under expected confining stress = 20% (see Graph 15 sect. B 6.2.2.2)

Free swell index of bentonite = 681% (See Table 6 sect B 6.1.4)

Bulk density = 1290kg/m^3

CALCULATIONS

1. The volume of voids in $1m^3$ of soil

$= 1m^3 \times e/(1+e) = 1 \times 0.42 / (1+0.42) = \mathbf{0.30m^3}$

Maximum swell of 10% greater than the swell under the expected in-situ confining stress

= swell under expected confining stress x 1.1

= 20% x 1.1 = **22%**

∴ Percentage unswollen bentonite

= 100% / (100% + corrected maximum swell)

= 100% / (100%+20%) = **83%**

DESIGN LIMITS

The **MAXIMUM** volume of bentonite allowed in the mix so as to avoid the separation of the soil particles is calculated as follows :

(volume of voids in soil) x (percent unswollen bentonite) = max. volume

$= 0.30 \times 83\% = 0.25m^3$ of bentonite per $1m^3$ of soil

This is represented as a percentage by volume to the soil as :

0.22 / 1 x 100 = **25% by volume**

or as a percentage of mass as :

0.22 x 1290 / 1800 = **16% by mass**

The **MINIMUN** volume of bentonite required to just fill the voids of the soil is obtained by :

(volume of voids in soil) / (free swell index of bentonite) = min. volume

$= 0.30 / 681\% = 0.04\ m^3$ of bentonite per $1\ m^3$ of soil

This is represented as a percentage by volume to the soil as :

0.04 / 1 x 100 = **4% by volume**

or as a percentage of mass as :

0.04 x 1290 / 1800 = **3% by mass**

14.4. Consolidation

On application of a load, a soil/bentonite mix reaches stability after a period of time whereas pure bentonite samples will initially show erratic movement and will eventually reach stability but this will change if the conditions change.

A relationship exists between the coefficient of volume compressibility (mv) and the percentage of bentonite in a mix. The higher percentage mixes will provided slightly higher mv coefficients. Typical mv values for a 6% mix by mass with sand are in the order of $1.5x10^{-4}m^2/kN$ and for a 12% mix in the order of $4.5x10^{-4}m^2/kN$.

Under certain situations, the pure bentonite may produce mv coefficients close to zero, i.e. typical of rock, but what is actually happening is that the swell is equal to the settlement and no change in the void ratio is experienced.

15. Construction

Regardless of how well a liner is designed and tested in the laboratory, the final success depends extensively on the physical construction of the entire liner system on site. Stringent control throughout the process is required and an experienced contractor who understands the workings of the system and the materials is strongly recommended.

Constructing liners using pure bentonite is problematic especially when only a thin layer (about 10mm) is specified. Achieving a constant thickness is critical yet almost impossible. While placing the protective layer over the bentonite, the fine bentonite powder is unevenly dispersed. This can be overcome by placing a geotextile over the bentonite before placing the next layer. Additionally, the bentonite should not get wet until it is covered with at least 300mm of soil to insure that an even layer is achieved and any small holes are self sealed when the bentonite hydrates and swells.

Soil/bentonite liners are less troublesome to construct. Three critical checks are required to insure the final product. There are :

1. Percentage content of bentonite in the liner.
2. Degree of compaction of the liner.
3. Actual thickness of liner.

The percentage of bentonite can be checked using the Methylene Blue Test preferably with tetrasodium pyrophosphate as a dispersant (Alther, 1983). The degree of compaction, the moisture content, and the thickness can be controlled with the standard geotechnical methods. Although not critical, the moisture content should be about 1% to 2% above optimum to aid the initial hydration process and compacted density. The moisture content should however not deviate beyond this. The thickness of the liner is directly related to the permeability, thus strict tolerances should be kept. Checking the thickness requires damaging the liner for a visual inspection (a situation also caused by compaction control) and special care must be taken to repair and seal the holes again.

A protective layer of about 300mm above the liner is recommended for most applications especially where the liner is exposed and could dry out or be damaged by physical means.

The initial filling of the containment area should be done as slowly as possible to give the bentonite time to hydrate before excessive hydraulic gradients are applied. This would not be a problem on landfill sites or cappings where the in-fill rates are relatively slow and the hydraulic gradients low. There exists a certain amount of debate of whether to pre-hydrate the liner with fresh water before exposing it to the containment liquid or simply to expose it immediately to the containment liquid. The reasoning behind the first option is that the bentonite is allows to build up its full potential, although the cost of the water to fill the area makes this option costly. The second perception is that if the liner hydrates in the containment liquid, the maximum potential under that liquid is obtained and no chemical changes can be expected with time.

Correct preparation of the soil layers below the liner which may also incorporate a drainage system require similar control as the liner itself. Research has shown that if the filter system is not operating effectively, a pore water pressure buildup within the liner occurs. This results in additional swelling of the bentonite which makes the bentonite less dense and therefore more permeable.

To insure that weak spots are not created where the continuity of the liner is affected, for example at inlet and outlet pipes, at collecting sumps, against walls and support plinths, etc., it is recommended that pure bentonite is liberally applied in these areas. It is preferable to use granular bentonite for ease of handling.

Although bentonite has certain properties of great value to the Civil and Environmental Engineer, without adequate knowledge of not only the construction techniques but also the design and general application, failures are inevitable. Continued research work is critical to further the knowledge and understanding of the South African Bentonites to make them a safe and viable alternative liner material.

16. Acknowledgments

The following persons and companies contributed to this research :

Dr. F. Scheele	Supervised the research and was a constant source of motivation and assistance throughout the duration of this thesis.
Mr E Von Guerard	With his staff at the UCT Civil Engineering Dept. Workshop, built most of the testing apparatus to near perfection and was always ready to help in any way.
Boland Base Minerals	Supplied bentonite for the tests conducted in this research.
Cape Bentonite	Supplied bentonite for the tests conducted in this research and information on bentonite.

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Appendix

Appendix A

Analysis of bentonites done by the Finnish Environmental Agency

**TAMPEREEN
TEKNILLINEN
KORKEAKOULU**
Rakennusgeologian laitos

Tampere University of Technology

Laboratory of Engineering Geology

Tampere 29.4.1996

Testauselostus nro RAKG 132/96

BENTONIITTINÄYTTEIDEN KARAKTERISOINTI

Tilaaja: Minna Leppänen, Syke/Laboratorio Hakuninmaantie 4-6 B, 00430 Helsinki

5 bentonite samples

Näytteet: Viisi bentoniittinäytettä (TTKK:n työnumero 36)

Determination of specific surface and mineral content

Tehtävä: Ominaispinta-alan ja mineraalikoostumuksen määrittäminen em. tilaajan toimittamista näytteistä.

Specific surface

Tulokset: Määritykset on suoritettu Rakennusgeologian laboratoriossa. Ominaispinta-alat määritettiin Flowsorb 2300 typpiadsorptiolaitteella. Mineraalikoostumukset määritettiin röntgendiffraktometrilla suunnatuista näytteistä. Tuloset on esitetty oheisessa taulukossa.

mineral content

Tulokset pätevät ainoastaan testatuille näytteille.

Specific surface   Minerals

Näyte Sample	Ominaispinta-ala (m^2/g)	Mineraalit
AC 1000 96-28	20,0	Bt, Klo/Kaol, Kv, Plg, Kms
AC 200 96-29	30,2	Na-Bent, (epäpuhtautena) Kms as impurity little
Calbund 96-30	29,7	Ca-Bent, vähän Na-Bent, (epäpuhtautena) Kv, Kms as impurity
Koppies 96-31	38,9	Na-Bent, (epäpuhtautena) Kms as impurity
Oceanclean 96-32	24,7	Ca-Bent, (epäpuhtautena) Kv as impurity

Bt: biotite (black mica)
Klo: Chlorite
Kv: quartz
Kaol: kaolinite
Plg: plagioclase
Kms: potash feldspar
Na-bent: ~~ca~~ sodium bentonite
Ca-bent. calcium bentonite

Bt = Biotiitti
Klo = Kloriitti
Kv = Kvartsi
Kaol = Kaoliniitti
Plg = Plagioklaasi
Kms = Kalimaasälpä
Na-Bent = Natriumbentoniitti
Ca-Bent = Kalsium bentoniitti

Amounts of impurities are really small, but detectable.

Pirjo Kuula-Väisänen
Tutkija

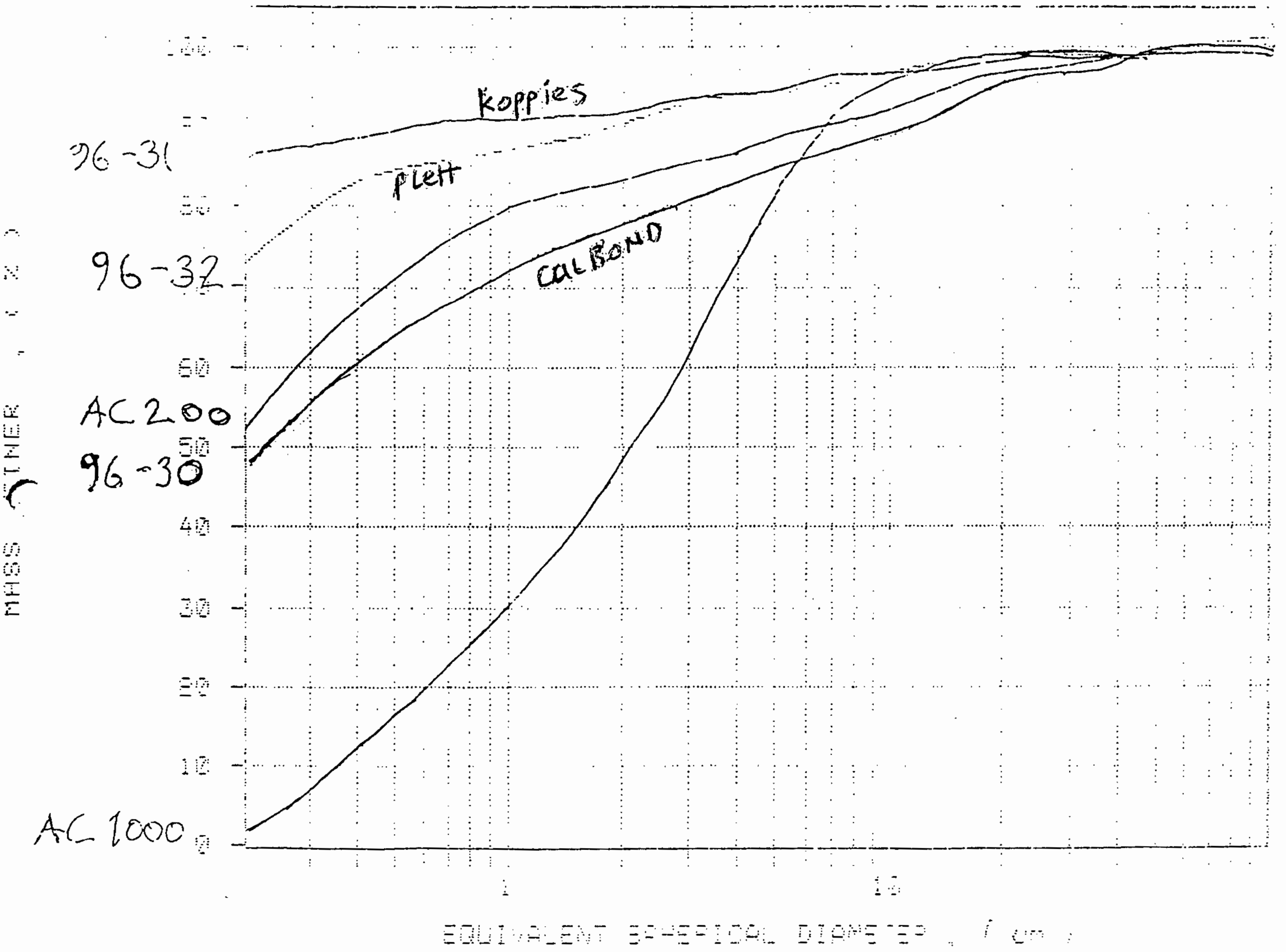
Pertti Nieminen
Laboratorioinsinööri

Sample AC1000 is bare clay.

```
SAMPLE DIRECTORY/NUMBER: BENTONIT/5              UNIT NUMBER: 1
SAMPLE ID: 96-32                                 START 16:25:22 03/14/96
SUBMITTER: Minna Leppanen, VYR                   REPRT 16:53:52 03/14/96
OPERATOR: RV                                     TOT RUN TIME    0:28:26
SAMPLE TYPE: LIHAVASAVI                          SAM DENS:   2.7000  g/cc
LIQUID TYPE: Water                               LIQ DENS:   0.9956  g/cc
ANALYSIS TEMP: 30.2 deg C    RUN TYPE: High Speed LIQ VISC:   0.7980  cp
```

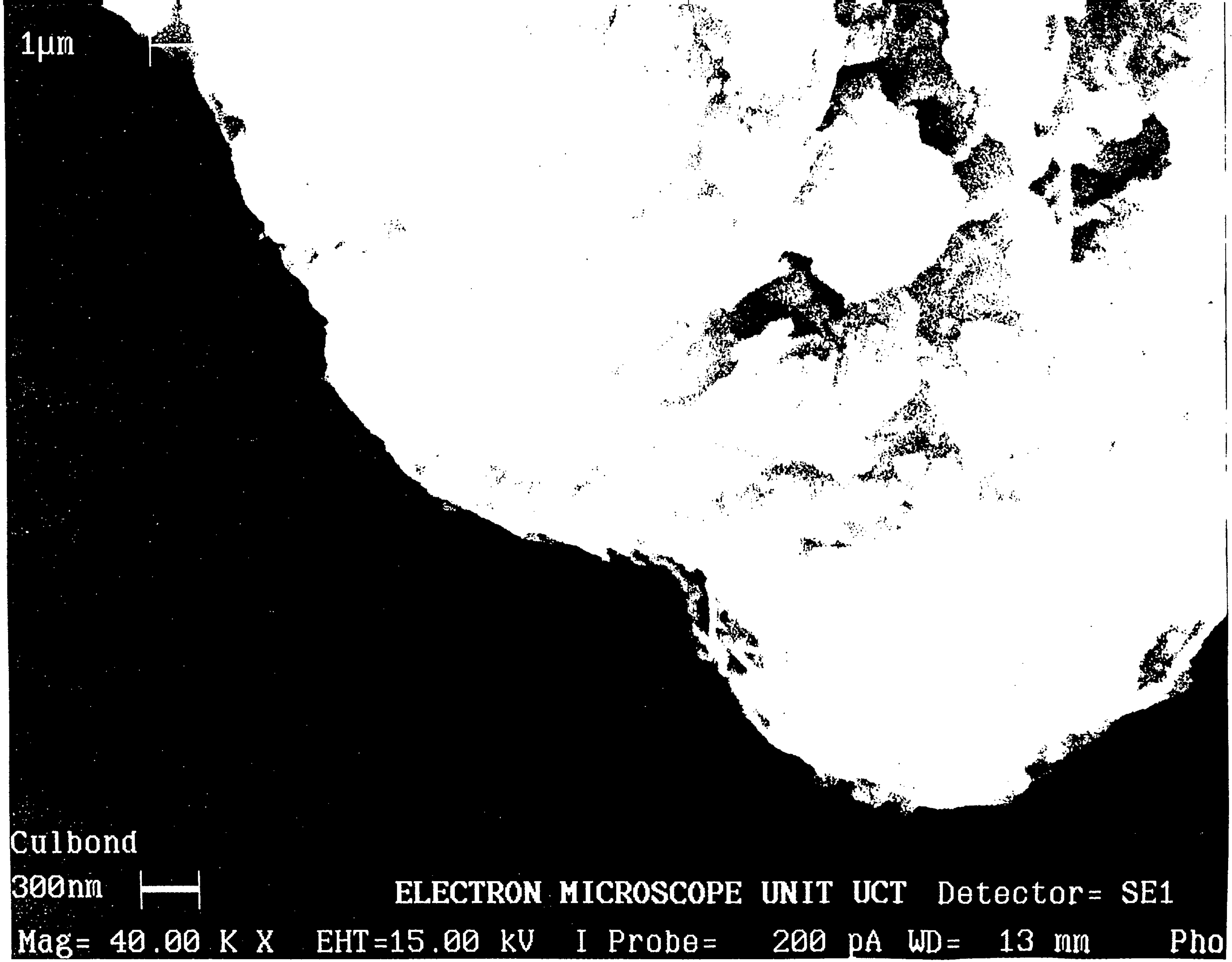
CUMULATIVE MASS PERCENT FINER VS DIAMETER

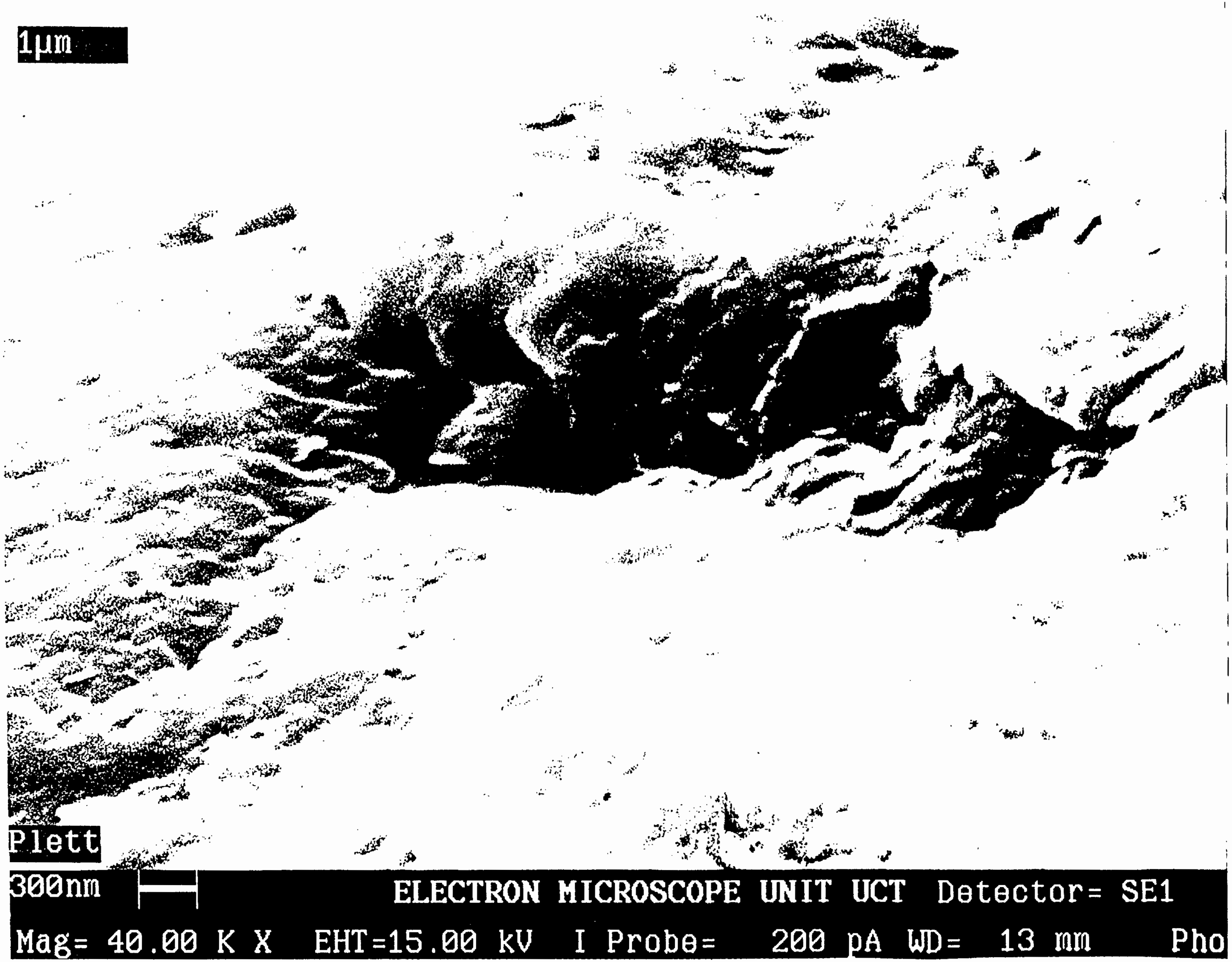
Compination of sedigraph-results

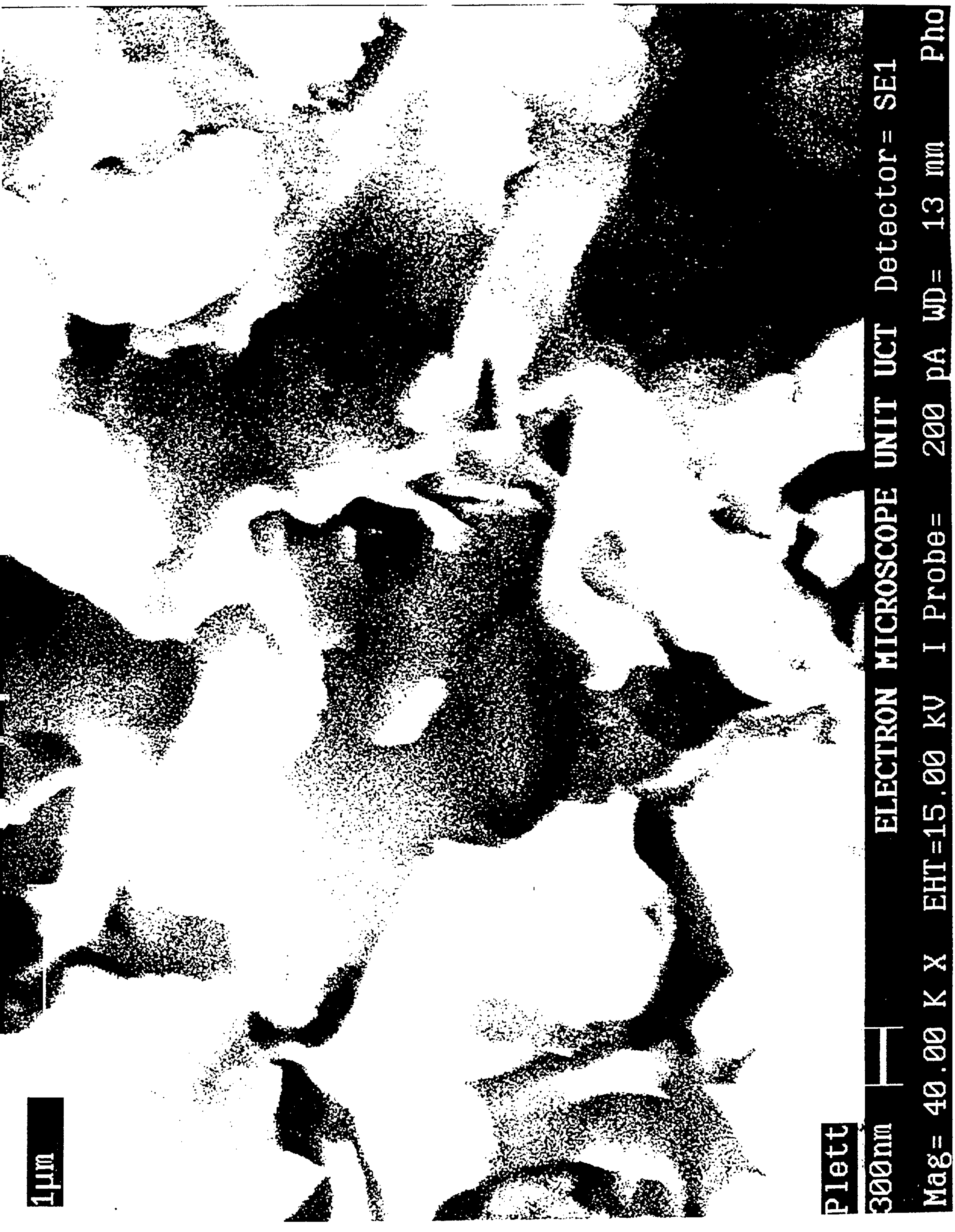


Appendix B

Electron Microscope Photographs


1µm
Culbond
300nm
ELECTRON MICROSCOPE UNIT UCT Detector= SE1
Mag= 40.00 K X EHT=15.00 kV I Probe= 200 pA WD= 13 mm Pho


1µm
Plett
300nm
ELECTRON MICROSCOPE UNIT UCT Detector= SE1
Mag= 40.00 K X EHT=15.00 kV I Probe= 200 pA WD= 13 mm Pho


1µm
Plett
300nm
ELECTRON MICROSCOPE UNIT UCT
Detector= SE1
Mag= 40.00 K X
EHT=15.00 kV
I Probe= 200 pA
WD= 13 mm
Pho

Appendix C

Water sample analysis

Test by: The Geology Department, UCT

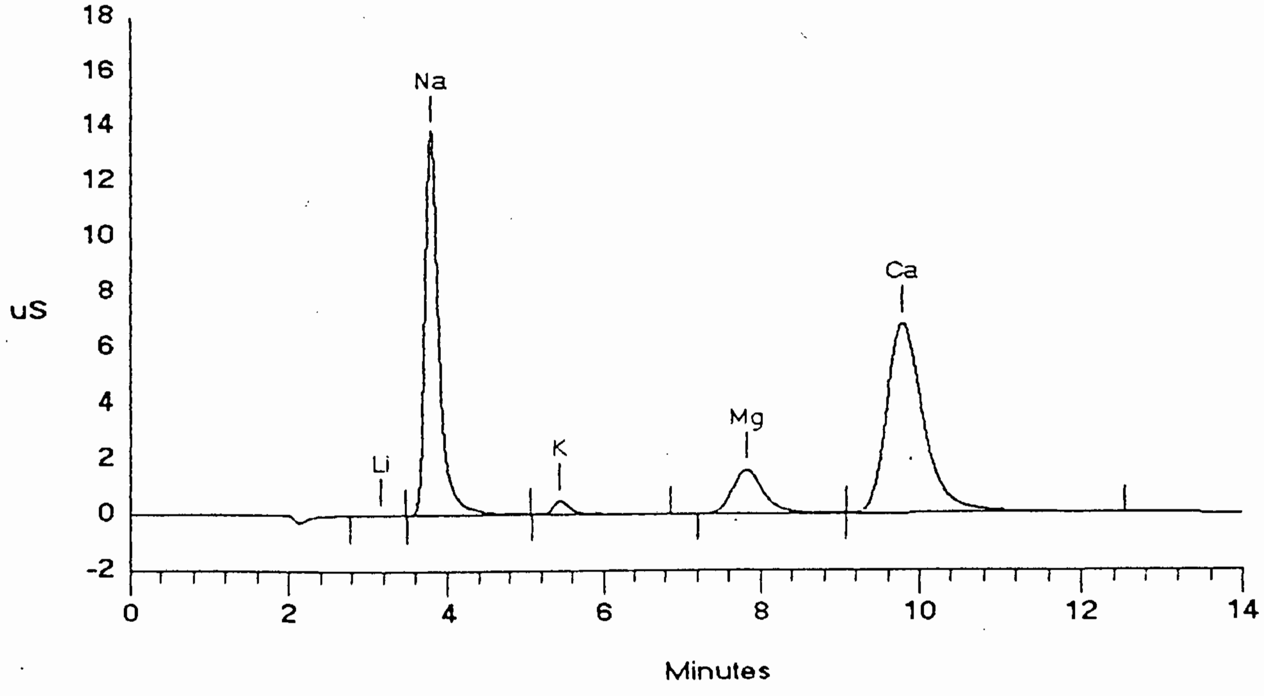
```
                    Data Reprocessed On 10/08/1996 14:30:07
===========================================================================
 Sample Name: A                               Date: 10/07/1996 11:36:39   :
 Data File   : C:\DX\DATA\FSIEAS\07109771.D02                             :
 Method      : C:\DX\METHOD\AK071096.MET                                  :
 ACI Address: 1  System: 1  Inject#: 2                    Detector:CDM-2  :
 Analyst     :                Column:                                     :
===========================================================================

alibration  Volume   Dilution Points Rate  Start   Stop Area Reject
------------------------------------------------------------------
External          1          1    4200  5Hz   0.00  14.00         1000

********************* Component Report: All Components *********************

Pk.     Ret Component             Concentration     Height        Area   Bl.  %Delta
Num    Time Name                            ppm                          Code
-----------------------------------------------------------------------------------
  1    3.17 Li                            0.000       4559       97514     2    0.53
  2    3.78 Na                            9.552   13834948   165771117     3    0.00
  0    0.00 NH4                           0.000          0           0     0    0.00
  3    5.43 K                             0.597     487085     7999917     4    0.00
  4    7.82 Mg                            1.960    1577322    43918659     2    1.12
  -    9.77 Ca                           10.511    6837220   224692368     2    2.59
                            -------------------------------------------
                      Totals             22.620   22741134   442479575
```

File: 07109771.D02 Sample: A DE-AIR DIRECT OUT OF SYSTEM pH 7·53

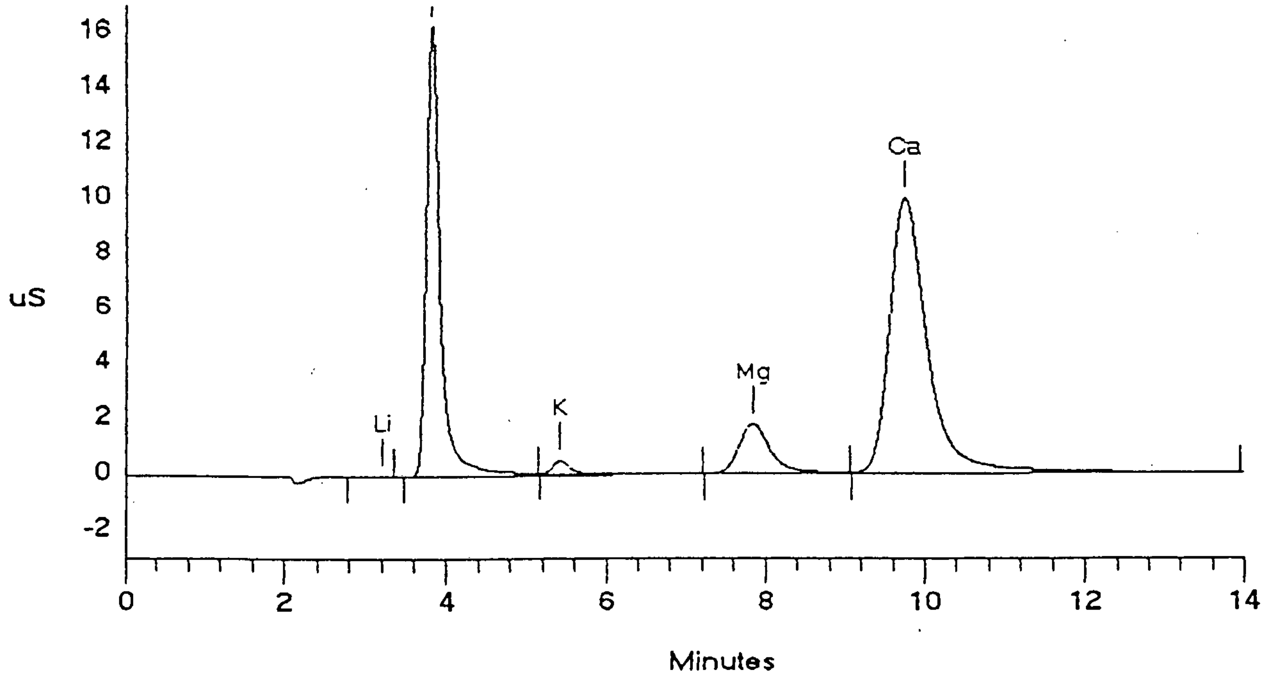


Test by: The Geology Department, UCT

```
==============================================================================
 Sample Name: B                               Date: 10/08/1996 11:57:12     :
 Data File   : C:\DX\DATA\PSIEAS\08109771.D06                                :
 Method      : C:\DX\METHOD\AK081096.MET                                     :
 ACI Address: 1  System: 1  Inject#: 6                   Detector:CDM-2     :
 Analyst     :                 Column:                                       :
==============================================================================

alibration  Volume   Dilution Points Rate  Start   Stop Area Reject
-------------------------------------------------------------------
xternal           1          1   4200  5Hz   0.00  14.00        1000

********************* Component Report: All Components **********************
Pk.     Ret Component            Concentration     Height       Area  Bl. %Delta
Nı     Time Name                           ppm                          Code
-------------------------------------------------------------------------------
  1    3.20 Li                           0.000       4297       72506    1   1.59
  2    3.82 Na                          10.721   16272277   192924574    3   0.00
  0    0.00 NH4                          0.000          0           0    0   0.00
  3    5.42 K                            0.590     493101    10265578    4   0.00
  4    7.83 Mg                           2.166    1805024    51358933    2   1.34
  5    9.73 Ca                          15.211    9961551   334767905    2   2.24
                              ------------------------------------------
                     Totals             28.689   28536250   589389497
```


File: 08109771.D06 Sample: B DE-AIR VIA SAMPLE PH 7.68

Test by: The Geology Department, UCT

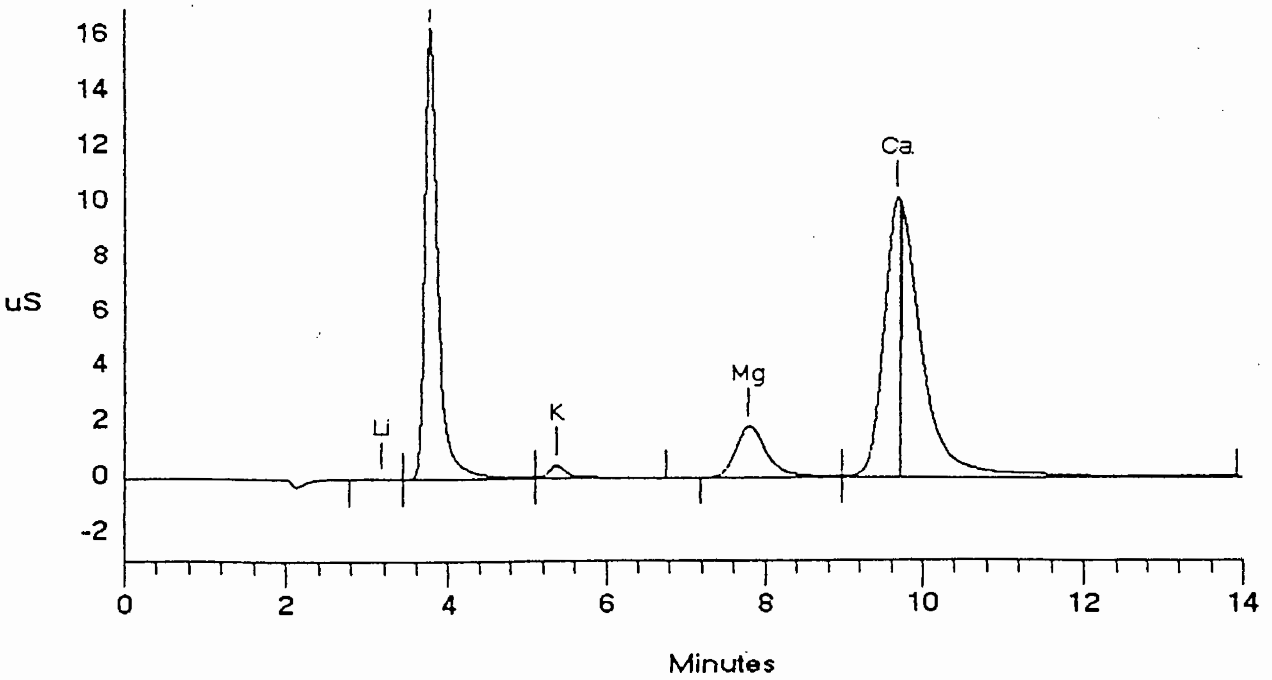
```
=============================================================================
 Sample Name: C                                 Date: 10/08/1996 12:13:19   :
 Data File   : C:\DX\DATA\FSIEAS\08109771.D07                               :
 Method      : C:\DX\METHOD\AK081096.MET                                    :
 ACI Address: 1  System: 1   Inject#: 7                   Detector:CDM-2    :
 Analyst     :                Column:                                       :
=============================================================================

alibration  Volume   Dilution Points Rate  Start   Stop Area Reject
--------------------------------------------------------------------
ternal             1          1   4200  5Hz   0.00  14.00          1000

********************* Component Report: All Components **********************

k      Ret Component                   Concentration     Height        Area  Bl. %Delta
um    Time Name                                  ppm                        Code
----------------------------------------------------------------------------------------
 1    3.17 Li                                  0.000       5523      102803    2    0.53
 2    3.77 Na                                 10.669   16193989   187448182    3    0.00
 0    0.00 NH4                                 0.000          0           0    0    0.00
 3    5.37 K                                   0.516     436455     6870928    4    0.00
 4    7.78 Mg                                  2.178    1814352    49935873    2    0.69
 5    9.68 Ca                                 15.420   10093499   330952598    2    1.72
                            ------------------------------------------------------
                      Totals                  28.783   28543819   575310385
```

File: 08109771.D07 Sample: C WATER FROM TAP AT 7AM. PH 7·64



Appendix D

0.5g Free Swell test results

Free Swell Test

1/2 g of Culseal Bentonite

Water added on 21/10/96 @ 12h15

Sand up to 10mm mark

		1		2		3		4		5		
	Time [Days]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Ave [%]
21 Oct 12:15	0	12	0	13	0	12	0	12	0	12	0	0
21 Oct 15:15	0.1	20	400	21	267	19	350	20	400	22	500	383
21 Oct 17:05	0.2	21	450	22	300	20	400	21	450	23	550	430
22 Oct 12:10	1.0	24	600	25	400	22	500	22	500	25	650	530
24 Oct 13:55	3.1	25	650	26	433	24	600	24	600	26	700	597
25 Oct 10:17	3.9	26	700	26	433	24	600	25	650	26	700	617
28 Oct 10:15	6.9	26	700	27	467	25	650	25	650	26	700	633
29 Oct 17:00	8.2	26	700	27	467	25	650	25	650	26	700	633
31 Oct 16:45	10.2	26	700	27	467	25	650	25	650	26	700	633

Free Swell Test

1/2 g of Calbon-N Bentonite

Water added on 21/10/96 @ 12h15

Sand up to 10mm mark

		6		7		8		9		10		
	Time [Days]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Ave [%]
21 Oct 12:15	0	13	0	14	0	14	0	14	0	15	0	0
21 Oct 15:15	0.1	15	67	15	25	16	50	16	50	17	40	46
21 Oct 17:05	0.2	16	100	16	50	17	75	16	50	17	40	63
22 Oct 12:10	1.0	23	333	21	175	24	250	24	250	20	100	222
24 Oct 13:55	3.1	25	400	25	275	25	275	25	275	24	180	281
25 Oct 10:17	3.9	25	400	25	275	27	325	25	275	24	180	291
28 Oct 10:15	6.9	27	467	26	300	28	350	26	300	25	200	323
29 Oct 17:00	8.2	27	467	26	300	29	375	26	300	25	200	328
31 Oct 16:45	10.2	27	467	26	300	29	375	26	300	25	200	328

Free Swell Test 1/2 g of Plett Bentonite Water added on 21/10/96 @ 12h15 Sand up to 10mm mark

		11		12		13		14		15		
	Time [Days]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Ave [%]
21 Oct 12:15	0	14	0	14	0	14	0	14	0	14	0	0
21 Oct 15:15	0.1	16	100	16	50	16	50	16	50	16	20	54
21 Oct 17:05	0.2	17	133	17	75	17	75	17	75	17	40	80
22 Oct 12:10	1.0	20	233	21	175	20	150	21	175	20	100	167
24 Oct 13:55	3.1	22	300	23	225	22	200	22	200	22	140	213
25 Oct 10:17	3.9	22	300	23	225	22	200	22	200	22	140	213
28 Oct 10:15	6.9	22	300	24	250	23	225	22	200	23	160	227
29 Oct 17:00	8.2	22	300	24	250	23	225	23	225	23	160	232
31 Oct 16:45	10.2	22	300	24	250	23	225	23	225	23	160	232

Free Swell Test 1/2 g of Koppies Bentonite Water added on 21/10/96 @ 12h15 Sand up to 10mm mark

		16		17		18		19		20		
	Time [Days]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Height [mm]	Swell [%]	Ave [%]
21 Oct 12:15	0	14	0	14	0	14	0	14	0	15	0	0
21 Oct 15:15	0.1	15	25	15	25	15	25	15	25	15	0	20
21 Oct 17:05	0.2	16	50	16	50	16	50	16	50	17	40	48
22 Oct 12:10	1.0	20	150	22	200	21	175	21	175	25	200	180
24 Oct 13:55	3.1	23	225	25	275	25	275	25	275	28	260	262
25 Oct 10:17	3.9	25	275	27	325	26	300	27	325	29	280	301
28 Oct 10:15	6.9	26	300	27	325	27	325	28	350	30	300	320
29 Oct 17:00	8.2	27	325	27	325	27	325	28	350	30	300	325
31 Oct 16:45	10.2	27	325	28	350	28	350	29	375	31	320	344

Appendix E

Consolidometer swell test results on sand with bentonite ring

CONSOLIDOMETER PERMEAMETER TEST
(FOR PURE THIN-LAYER TEST)

Test number : S1 & S2

Sample type : Cape Flats Sand with Culbond ring
Date prep : 21/05/96

Test date 21/05/96
Hydration 0 days

	Dish wt	tot wet	tot dry	Mc (%)
Mc before				5%
Mc after				

Sample mass 186 g
Bulk Density 1860 kg/m3

Ring area 4989 mm^2
Ring height 20 mm
Sample height 20 mm
Sample dia 69 mm
Sample area 3739 mm^2
Vol of solids 66447.48 mm^3

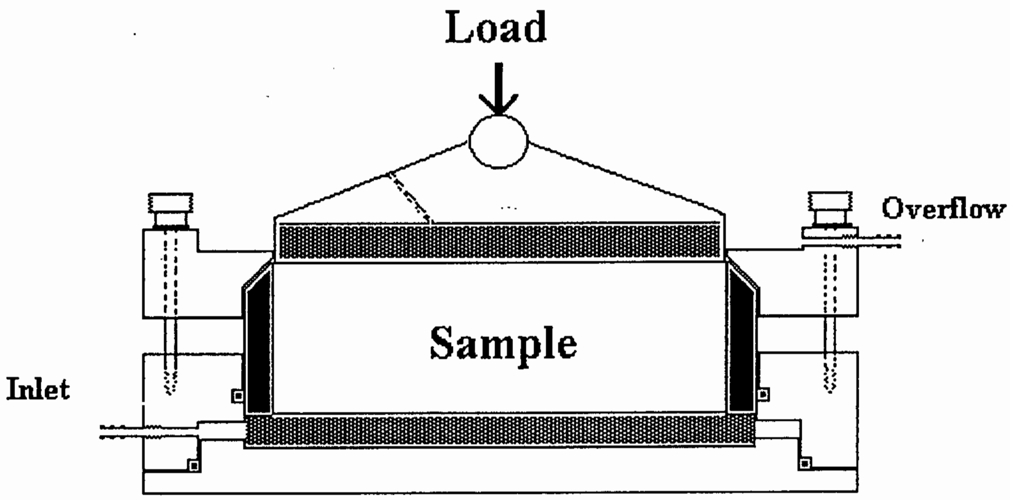
Date TEST S1	Applied load (kg)	Applied pressure (kPa)	Time	Gauge reading	Settle- ment (mm)	Total settlement (mm)	Samlpe thickness (mm)	void ratio	(Relative settlement)
21/05/96	2.564	55.830	00:00:00	12.87	0.000	0.000	20.0	0.50	15.000
			00:00:30	9.00	0.774	0.774	19.23	0.44	14.420
			00:01:00	8.98	0.004	0.778	19.23	0.44	14.420
water added			00:15:00	8.84	0.028	0.806	19.20	0.44	14.399
			00:16:30	8.75	0.018	0.824	19.18	0.44	14.385
			00:17:30	8.50	0.050	0.874	19.13	0.44	14.348
			00:19:00	8.20	0.060	0.934	19.07	0.43	14.303
			00:23:00	8.12	0.016	0.950	19.05	0.43	14.291
			00:45:00	8.08	0.008	0.958	19.05	0.43	14.285
			20:44:00	8.09	-0.002	0.956	19.05	0.43	14.286
22/05/96	5.12	111.49	00:00:00	8.09	0.000	0.956	19.05	0.43	14.286
			00:01:00	7.85	0.048	1.004	19.00	0.43	14.250
			03:20:00	7.71	0.028	1.032	18.97	0.42	14.229
			23:59:00	7.66	0.010	1.042	18.96	0.42	14.222
23/05/96	10.157	221.16	00:01:00	7.30	0.072	1.114	18.89	0.42	14.168
			23:59:00	7.20	0.020	1.134	18.87	0.42	14.153
26/05/96			18:29:00	6.93	0.054	1.188	18.82	0.41	14.112
30/05/96			20:00:00	6.90	0.006	1.194	18.81	0.41	14.108
TEST S2									
21/05/96	2.564		00:00:00	14.40	0.000	0.000	20.0	0.50	0.000
			00:00:30	11.83	0.514	0.514	19.49	0.46	0.774
			00:01:00	11.80	0.006	0.520	19.49	0.46	0.004
water added			00:15:00	11.66	0.028	0.548	19.46	0.46	0.028
			00:16:30	11.77	-0.022	0.526	19.48	0.46	0.018
			00:17:30	11.67	0.020	0.546	19.46	0.46	0.050
			00:19:00	11.44	0.046	0.592	19.41	0.46	0.060
			00:23:00	11.03	0.082	0.674	19.33	0.45	0.016
			00:45:00	10.87	0.032	0.706	19.30	0.45	0.008
			20:44:00	10.85	0.004	0.710	19.30	0.45	-0.002
22/05/96	5.12		00:00:00	10.85	0.000	0.710	19.30	0.45	0.000
			00:01:00	10.58	0.054	0.764	19.24	0.44	0.048
			03:20:00	10.42	0.032	0.796	19.21	0.44	0.028
			23:59:00	10.25	0.034	0.830	19.18	0.44	0.010
23/05/96	10.157		00:01:00	9.75	0.100	0.930	19.08	0.43	0.072
			23:59:00	9.50	0.050	0.980	19.03	0.43	0.020
26/05/96			18:29:00	9.50	0.000	0.980	19.03	0.43	0.054
30/05/96			20:00:00	9.35	0.030	1.010	19.00	0.43	0.006

Appendix F

Diagram of the modified consolidometer cell

Consolidometer Cell for Falling Head Permeability Test

Cross-section: (schematic)



Dimensions:			
	Cutting Ring		
		Diameter (outside)	85 mm
		Height	20 mm
		Thickness	2.5 mm
	Porous Disk		
		Diameter	85 mm
		Thickness	6.5 mm

Appendix G

Consolidometer permeameter pure bentonite test results

CONSOLIDOMETER PERMEAMETER TEST						PURE BENTONITE SUMMARY								
Surcharge		test number	1	2		3	4		5	6		7	8	
pressure		bentonite type	Plett	Plett		Koppies	Koppies		Calbond N	Calbond N		Culseal	Culseal	
					AVE. K(m/s)			AVE. K(m/s)			AVE. K(m/s)			AVE. K(m/s)
55.69913187			1.53E-11	1.22E-11		3.5E-11	3.2E-11		1.9E-11	2.3E-11		4.39E-11	1.29E-11	
			1.25E-11	1.03E-11		2.6E-11	1.8E-11		1.4E-11	1.7E-11		4.13E-11	3.69E-11	
		AVE. K(m/s)	1.39E-11	1.12E-11	1.26E-11	3.0E-11	2.5E-11	2.77E-11	1.6E-11	2.0E-11	1.82E-11	4.26E-11	2.49E-11	3.37E-11
110.9845485									8.5E-12	1.0E-11		1.24E-11	1.38E-11	
			8.27E-12	8.38E-12		9.3E-12	1.1E-11		9.1E-12	9.0E-12		1.22E-11	1.17E-11	
		AVE. K(m/s)	8.27E-12	8.38E-12	8.32E-12	9.3E-12	1.1E-11	1.03E-11	8.8E-12	9.7E-12	9.23E-12	1.23E-11	1.28E-11	1.26E-11
221.1634411						5.8E-12	6.8E-12		5.3E-12	3.9E-12		6.38E-12	7.34E-12	
			3.34E-12	3.99E-12		6.3E-12	6.3E-12		7.1E-12	3.7E-12		5.17E-12	4.39E-12	
			2.87E-12	1.92E-12		6.7E-12	6.2E-12		6.7E-12	4.5E-12		5.17E-12	5.51E-12	
		AVE. K(m/s)	3.10E-12	2.96E-12	3.03E-12	6.3E-12	6.4E-12	6.35E-12	6.1E-12	4.1E-12		3.25E-12	4.47E-12	
									5.3E-12	3.5E-12		4.99E-12	5.43E-12	5.21E-12
									6.1E-12	3.9E-12	5.02E-12	8.59E-12	8.94E-12	
												9.07E-12	9.05E-12	
												8.83E-12	9.00E-12	8.91E-12
Moisture	before		16.46%	16.46%		15.30%	15.30%		12.10%	12.10%		11.90%	11.90%	
content	after		81.19%	81.02%		104.37%	104.37%		74.27%	72.25%		62.62%	64.52%	

WYATT ORSMOND

Appendix H

Typical triaxial cell permeability test data sheet

TRIAXIAL CELL PERMEABILITY

SAMPLE : Koppies

Cell Pressure	110	Kpa
Pore Pressure	100	Kpa
Back Pressure	60	Kpa
Sample Thickness	7	mm (+ -)
Vol. indicator Dia.	2.59	mm (+ -)
Sample Area	706.8583471	

	Date/Time Start	End	Measure (mm)	Volume ml	Time hrs	Perm m/s
Cell 1	22/10/96 12:00	22/10/96 16:15	26	0.14	4.25	2.22E-11
	22/10/96 16:15	24/10/96 13:15	265	1.40	45.00	2.13E-11
	24/10/96 13:15	25/10/96 10:12	126	0.66	20.95	2.18E-11
					AVE =	2.18E-11
Cell 2	21/10/96 13:35	22/10/96 12:00	155	0.82	22.42	2.51E-11
	22/10/96 12:00	22/10/96 16:15	29	0.15	4.25	2.47E-11
	22/10/96 16:15	24/10/96 13:15	270	1.42	45.00	2.17E-11
	24/10/96 13:15	25/10/96 10:12	129	0.68	20.95	2.23E-11
	25/10/96 10:12	28/10/96 10:15	476	2.51	72.05	2.39E-11
					AVE =	2.36E-11
Cell 3	21/10/96 13:35	22/10/96 12:00	181	0.95	22.42	2.93E-11
	22/10/96 12:00	22/10/96 16:15	32	0.17	4.25	2.73E-11
	22/10/96 16:15	24/10/96 13:15	343	1.81	45.00	2.76E-11
	24/10/96 13:15	25/10/96 10:12	161	0.85	20.95	2.78E-11
					AVE =	2.8E-11

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Appendix I

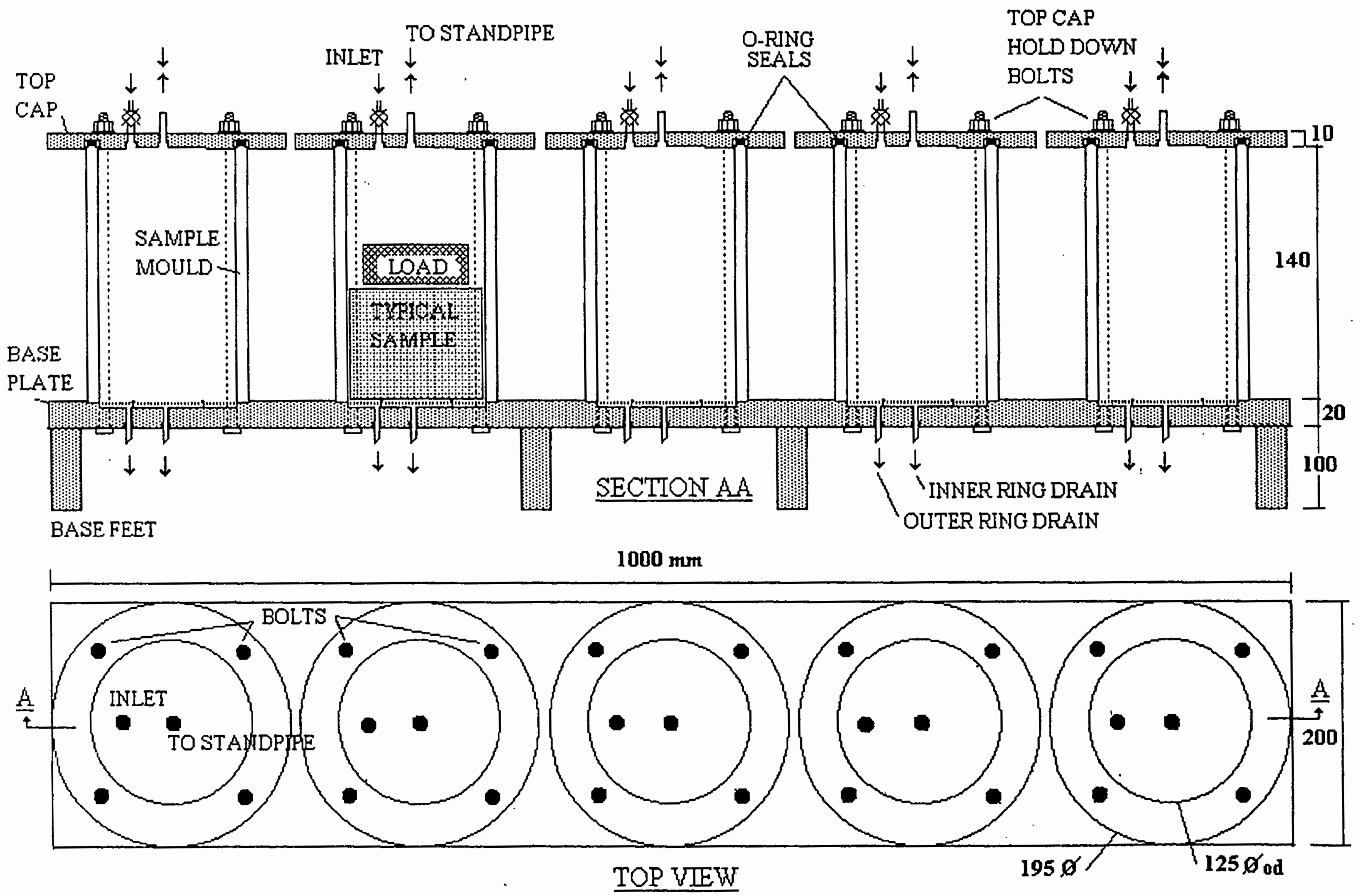
Diagram of the Double-Ring Compaction Mold permeameter

WYATT ORSMOND

COMPACTION MOULD PERMEAMETER

GENERAL LAYOUT

22/03/95

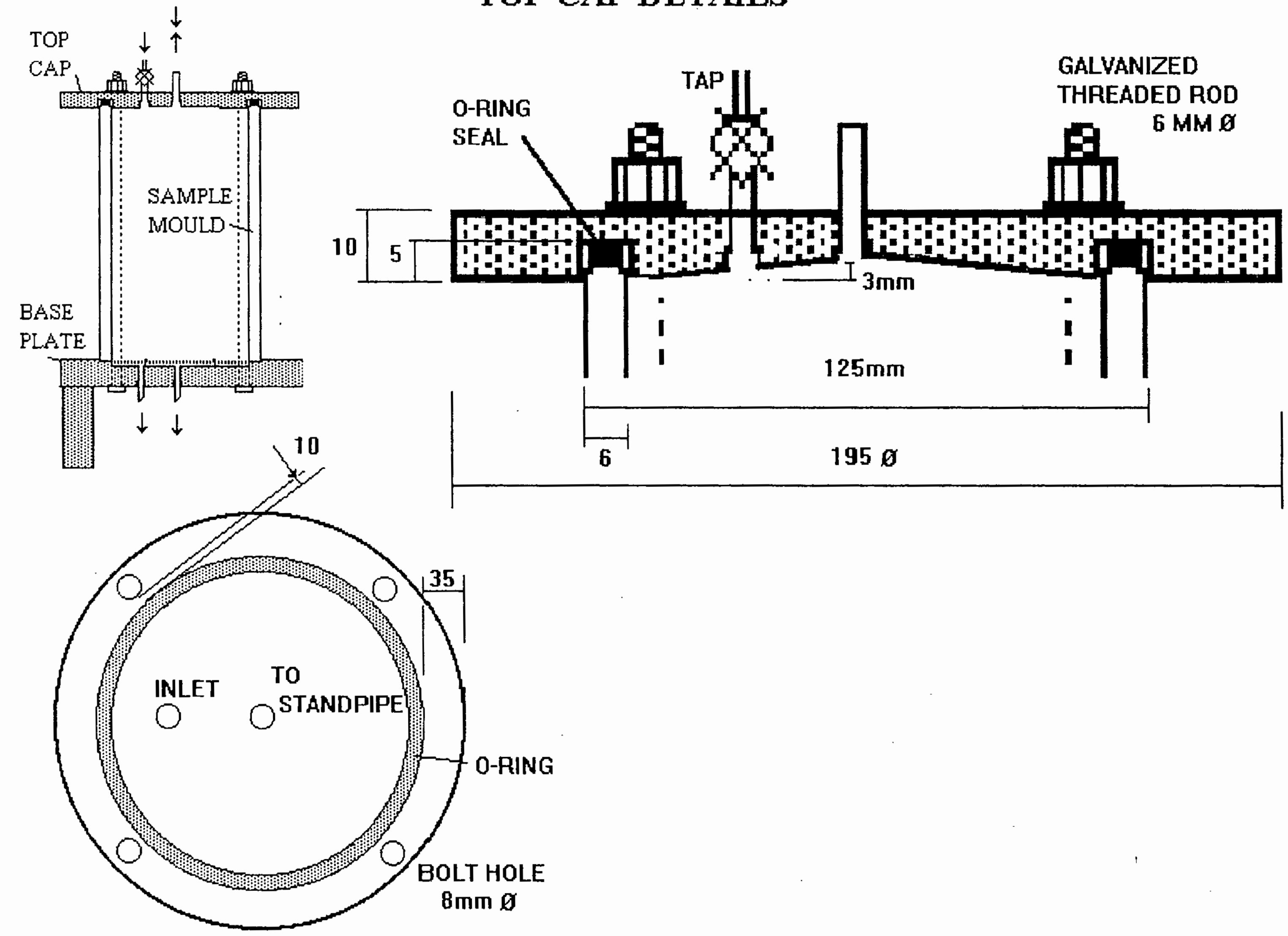


WYATT ORSMOND

COMPACTION MOULD PERMEAMETER

TOP CAP DETAILS

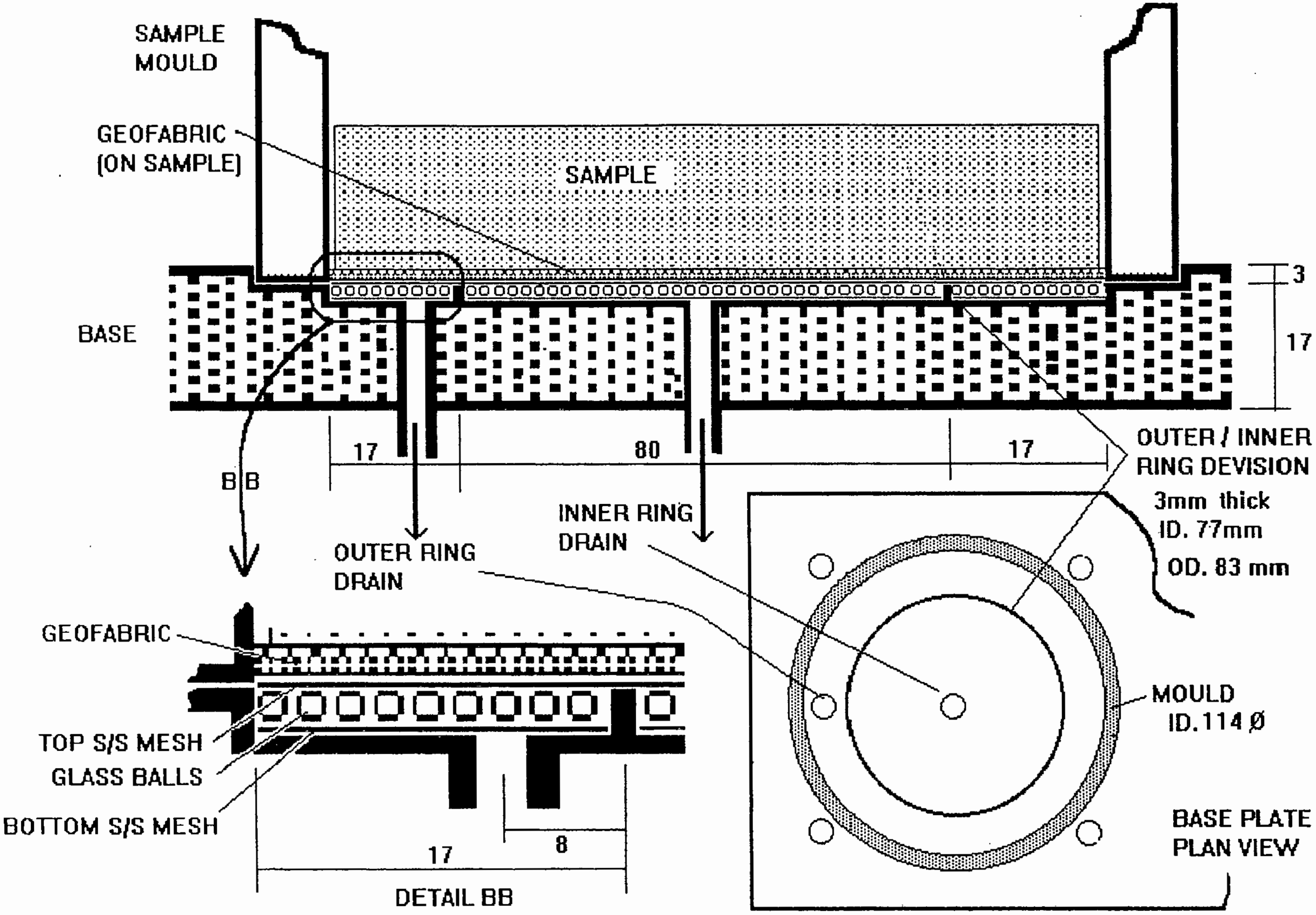
22/03/95



WYATT ORSMOND

COMPACTION MOULD PERMEAMETER
BASE DETAILS

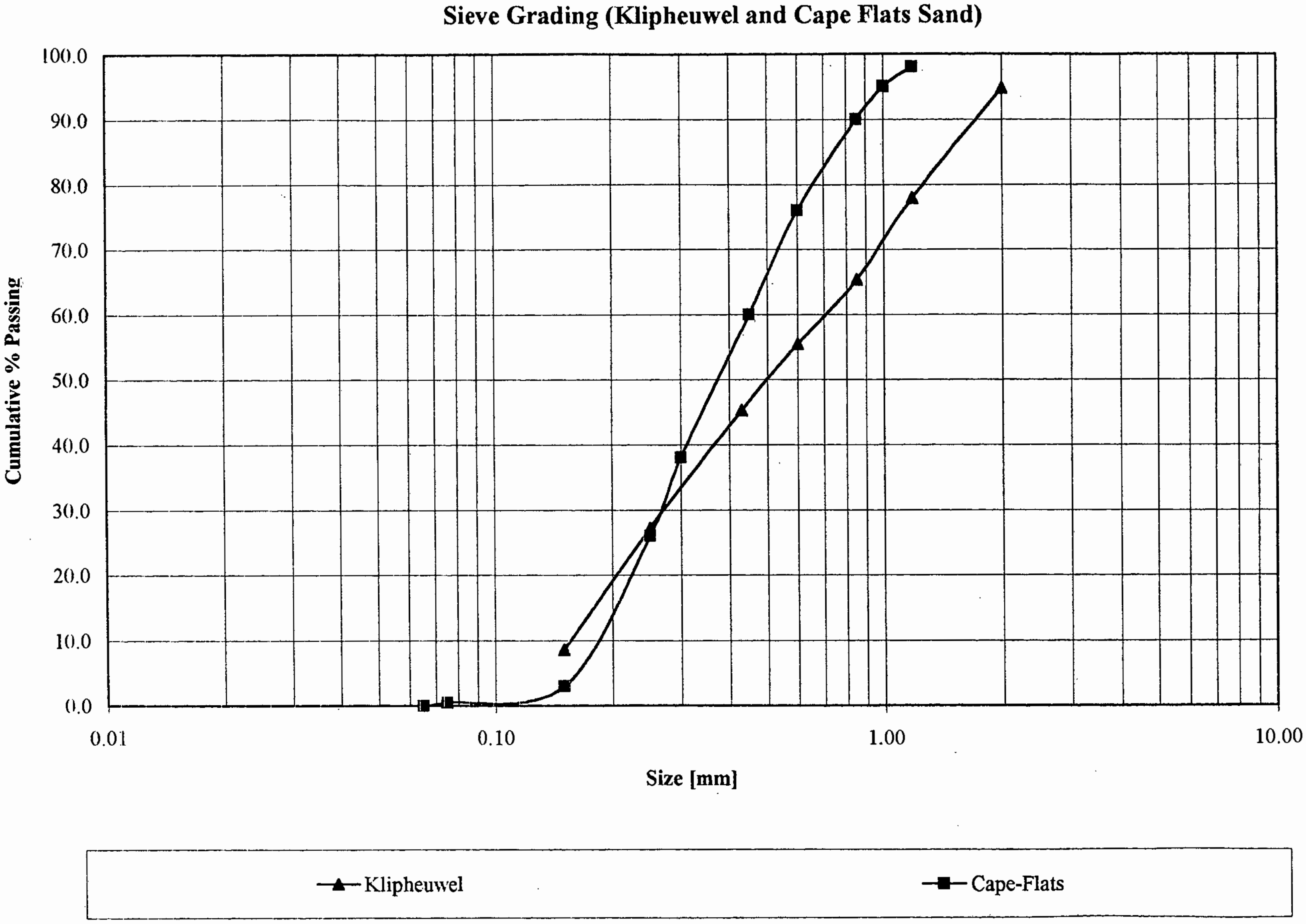
22/03/95



Appendix J

Soil grading curves

Test by: Klipheuwel : P. Comarmond
Cape Flats : J. Lai Sang



Appendix K

Typical consolidometer permeameter soil/bentonite test data sheet

CONSOLIDOMETER PERMEAMETER TEST

Test number : B/S 1

Sample type : 12% Culseal with Cape Flats sand
Date prep : 11/12/95

Test date 14/01/97
Hydration 400 days

	Dish wt	tot wet	tot dry	Mc (%)
Mc before	2.115	25.99	20.69	28.53%
Mc after	2.12	19.40	16.207	22.67%
Side of sample	2.10	19.94	16.52	23.72%

Ring area 4989 mm^2
Ring height 20 mm
Sample height 20 mm
Sample dia 79.7 mm
Sample area 4989 mm^2

Sample mass 191 g
Bulk Density 1913 kg/m3

	56-111kPa	111-223kPa
Coef. of Vol. Compress. (Mv)	6.2E-04	3.4E-04

Recorded Data

Computed

Surcharge pressure (kpa)	S - P diameter (mm)	Level in stand pipe Initial H1 (mm)	Final H2 (mm)	Time interval Date / time (t1)	Date / time (t2)	t2 -t1	Samlpe thickness	Permeability k (m/s)
55.83	3.4	1750	1555	15/01/97 16:30	16/01/97 16:43	1 00:13	19.6	4.833E-11
		1555	1422	16/01/97 16:43	17/01/97 13:45	0 21:02	19.6	4.211E-11
							ave. =	4.522E-11
111.49	3.4	1750	1605	20/01/97 17:11	21/01/97 16:35	0 23:24	18.9	3.535E-11
		1605	1474	21/01/97 16:35	22/01/97 16:28	0 23:53	18.9	3.410E-11
							ave. =	3.473E-11
223.49	3.4	1750	1345	23/01/97 13:23	27/01/97 16:00	4 02:37	18.2	2.463E-11
		1750	1648	27/01/97 16:00	28/01/97 17:00	1 01:00	18.2	2.208E-11
							ave. =	2.336E-11

Date	Applied load (kg)	Applied pressure (kPa)	Time	Gauge reading	Settle-ment (mm)	Total settlement (mm)	Samlpe thickness (mm)	void ratio
14/01/97	2.564	55.830	12:20:00	7.89	0.000	0.000	20.0	0.814
14/01/97			12:23:00	5.75	0.428	0.414	19.59	0.776
15/01/97			16:28:00	4.11	0.328	0.742	19.59	0.747
16/01/97			16:43:00	4.11	0.000	0.742	19.59	0.747
17/01/97			13:45:00	4.06	0.010	0.752	19.58	0.746
17/01/97	5.12	111.485	13:47:00	3.50	0.112	0.864	19.46	0.736
20/01/97			17:10:00	0.72	0.556	1.420	18.91	0.685
21/01/97			16:35:00	0.73	-0.002	1.418	18.91	0.685
22/01/97			16:28:00	0.68	0.010	1.428	18.90	0.684
22/01/97	10.264	223.493	16:29:00	0.45	0.046	1.474	18.85	0.680
23/01/97			13:22:00	-2.62	0.614	2.088	18.24	0.625
27/01/97			16:00:00	-2.94	0.064	2.152	18.18	0.619
28/01/97			17:00:00	-2.93	-0.002	2.150	18.18	0.619