RELATIONSHIP BETWEEN UNDRAINED SHEAR STRENGTH AND MOISTURE CONTENT FOR RED BEREA SAND TAILINGS

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DECLARATION

I declare that this research report is my own, unaided work. It is being submitted for the Degree of Master of Science in the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other University.

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17th day of July (year) 2001

ABSTRACT

The project report deals with the relationship between the undrained shear strength and the moisture content of Red Berea sand tailings. The tailings were obtained from the Red Berea sand dunes near Richards Bay, Kwa-Zulu Natal, South Africa. The geology of the area consists of Miocene deposits of red clayey sand, classified as Berea Formation.

A method for determining stability of a tailings dam for Red Berea sand tailings, was investigated. The general method of using the degree of saturation of the tailings to specify the rate of rise, is not applicable to this type of tailings.

It was found that a relationship exists between the undrained shear strength of the tailings, and the moisture content. The moisture content can easily be measured and the undrained shear strength can then be calculated. The calculated undrained shear strength can be used in a total stress analysis to determine a factor of safety against failure.

This project report consists of a discussion of the literature, which was used as the basis for the assumptions made, as well as a description of the tests performed to prove the above-mentioned relationship. Test results are given, interpreted and used in an illustrative example of a stability analysis.

DEDICATION

To my wife, for all her patience and support.

ACKNOWLEDGEMENT

This project report would not have been possible without Iscor (Pty) Ltd. and in particular, Mr. K Goss-Ross. Thank you for giving the author the opportunity to work on this project.

Professor AB Fourie, for assisting the author in compiling this project report.

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LIST OF SYMBOLS

Allowable bearing pressure	qall
Angle of internal friction	ϕ
Coefficient of consolidation	c_v
Cohesion	c'
Density of water	γw
Diameter of shear vane	D
Effective normal stress	σ'
Effective overburden pressure	p'
Factor of safety	FOS
Height of shear vane	H
In-situ dry density	ρ_d
Liquid limit	LL
Maximum consolidation stress	σ'_m
Maximum normal stress	s1(σ1)
Maximum torque	T
Minimum normal stress	s3(σ3)
Moisture content	w
Permeability	k
Plasticity index	PI
Pore water pressure	u
Radius of shear vane	r
Saturated density	γsat
Saturation	S
Shear stress on failure plain	τ_f
Specific gravity of solids	Gs
Total normal stress	σ
Ultimate bearing pressure	qu
Undrained shear strength	c_u

1 INTRODUCTION

1.1 Background of the project

Iscor Mining's Heavy Minerals Project centers on deposits in the Northern Province, KwaZulu-Natal and the Eastern Cape. This project report focuses on the ilmenite that is found in the hills of Hillendale, near Richards Bay, Kwazulu-Natal, South Africa. This heavy mineral can be recovered from the deposits and smelted to a high-grade titania slag and high purity iron. Titania is the basis of white pigments of paint, plastics and paper, while iron is preferentially used for high quality metal castings. Rutile and zircon are also recoverable (Clarke, 1996)

The mineral has to be mined out of the hills, which consists mainly of Red Berea sands. A typical grading analysis for the Red Berea sand tailings that were used for this project report, is shown in Table 4.2. It consists mainly of clay particles (smaller than 2μm).

During the mining process the topsoil will be stripped (from land previously used for agricultural purposes) and either stockpiled in the early days of the project, or transported directly to rehabilitate the backfilled mining areas. Hydraulic monitors will be used to break and slurry the ore, and achieve a uniform feed to the primary wet plant (PWP).

The PWP has a capacity of 1 200 t an hour. The five-stage spiral plant will separate the finer fraction (-45μm) and coarser fraction (+45μm) tailings from the heavy mineral concentrate. The coarser tailings, and a portion of the finer tailings, will be pumped back to the mine for backfilling. The bulk of the finer tailings will be pumped to the sub-aerial depositioning (SAD) site. These tailings are the subject of this report.

The tailings will be deposited in thin layers, approximately 100mm thick. Drying and consolidation will be done on a controlled basis and the water will be reclaimed. It is expected that the site will rise at a rate of 1,2 to 1,5 m a year. The site will then be rehabilitated for agricultural use.

The tailings dam has to adhere to statutory requirements, of which the most relevant are the Minerals Act (Act 50 of 1991) and the Water Act (Act 36 of 1998). These two Acts cover the requirements for public safety and dam safety respectively. A complete list of the legislation applicable to Mine Residue disposal is given in Appendix A.

One of the aspects of safety of a tailings dam is stability. There are a number of factors that influence the stability of a tailings dam, such as the rate of rise, material properties and method of deposition, etc. The rate of rise of a dam is a measure of how high the dam is built in a certain time period. The rate of rise is in turn determined by the minimum strength required in a particular layer, before the next layer can be deposited.

In conventional tailings dam design it is normally assumed that successive layers of tailings can be placed on top of each other as soon as the preceding layer has reached a certain strength. Usually the degree of de-saturation is used as an indication of when this minimum strength has been reached.

The ilminite tailings, used in this project report, start to de-saturate below ±35% gravimetric moisture content (the moisture content is also referred to as the water content and is defined as the ratio of the weight of water to the weight of solids in a given vloume of soil). At 35% moisture content the tailings are very hard and the time taken to reach this point is uneconomical from an operational point of view.

Therefore another method has to be used to determine when a layer of tailings on the dam is strong enough to carry the next layer, without compromising stability.

1.2 Scope of the project report

A pilot plant for the mining process was run in Hillendale for a number of months. The tailings obtained from the pilot plant were used to build experimental tailings deposits. These experiments were done to obtain information, such as beach profiles, rate of drying etc., which could be used for the final design of the dam. These tests did not provide enough information to determine the strength of the tailings deposit, and therefore the allowable rate of rise, as discussed in the previous section.

This project report consists of an investigation into the relationship between the moisture content and undrained shear strength of Red Berea sand tailings. If a relationship exists, it could be used to predict the required drying period between successive layers on the tailings dam.

Some of the tailings from the pilot plant operation were gathered in 25l drums and transported to Pretoria, where laboratory tests could be performed. The relationship between moisture content and undrained shear strength was determined with two different methods, described in detail in later chapters, namely the shear vane apparatus and the shear box test. Geotechnical parameters, such as coefficient of consolidation and undrained shear strength, were determined from the test results.

Foundation indicator tests were done to determine other geotechnical parameters (Plasticity Index (PI) and Liquid Limit (LL) etc). These parameters can be used to compare the tailings to materials used in other studies.

Stability analyses of a typical dam section were also done to illustrate the use of the findings. The stability analyses were done for the proposed design layout of the tailings dam, with the material properties obtained from the geotechnical site investigation. Total stress analyses were done, because effective stress and effective strength parameters could not be determined without pore water pressure measurements.

1.3 Layout of the report

The report consists of this introductory chapter, which describes the background of the project, the reasons for the investigation and the methods used for the investigation .

Some factors influencing stability of tailings dams as well as previous investigations of related topics are reviewed in the next chapter.

Chapter 3 describes the principles of shear strength, as well as the test procedures, namely the shear vane and the shear box, used for determining shear strength.

Chapter 4, states the results obtained from the experimental procedures, with some conclusions.

Chapter 5 is a discussion on the results obtained from the experimental results in Chapter 4. The principles discussed in Chapter 2 and the results from Chapter 4 are combined to form a method of determining the shear strength used in the stability analyses. Some calculations of settlement, which could be used for rehabilitation purposes, are also shown.

The use of the experimental results in total stress stability analyses is illustrated in Chapter 6.

In the concluding Chapter 7, the importance of the findings will be discussed as well as how the findings can be used for other applications.

1.4 Conclusion

A methodology had had to be found to determine the strength or stability of a tailings dam built with Red Berea sand tailings.

It was expected that there is a relationship between the moisture content of the tailings and its strength. This report shows that the relationship does exist and it further illustrates how this relationship can be used to determine the stability of a tailings deposit.

2 FACTORS INFLUENCING STABILITY OF TAILINGS DEPOSITS

2.1 Introduction

There are a number of factors influencing stability of tailing deposits, such as suction and capillary stresses. Some of these factors are discussed in this chapter. Other related topics such as thickened tailings and deposition methods are also discussed.

2.2 Suction

This project report is on the relationship between undrained shear strength and moisture content and although it is not dealt with in this report, a large influence on the undrained shear strength, is the suction developed when a material dries out. Suction is briefly discussed in this section.

One explanation for the stability of slopes, which are steeper than the angle of internal friction, is that the material has an apparent cohesion. This cohesion consists of a number of factors including suction. Suction can be described as the negative pore pressure that builds up as the material dries out. A number of investigators have measured relatively large suctions in the upper 1m to 2m of a slope (Fourie, 1996). These suctions were measured with a tensiometer.

In general the suctions are higher at shallow depths. It also shows that suction decreases during the wet season and this decrease is less for greater depths. The suction in a slope is also reduced by prolonged periods of rainfall,
(Fourie, 1996)

The influence of suction on stability is positive, but the magnitude is difficult to quantify. There are a number of thoughts on how suction should be incorporated into the stability calculations. The question of what value of cohesion and friction angle should be used is still a matter of debate (Fourie, 1996). The important fact to note is that the effects of suction increases the factor of safety dramatically, particularly for depths less than about 1m.

The commonly observed strength-gain by desiccation (drying) of soils (or tailings) indicates the presence of high tensile stresses in the fluid between the solid particles as evaporation proceeds from the surface. These stresses, or negative pore pressures, draw the particles closer together into a denser packing, thereby increasing the mass and strength.

Osmotic suction is believed to play a role in only the finest clay-like tailings. Matric suction is by far the most important component in the study of desiccation of tailings. The total theoretical suction can amount to 10 or more MPa, but in practice such high values are seldom encountered.

2.3 Capillary Stresses

Robinsky (1999) noted that a simplified approach to evaluate negative pore pressures is to measure the capillary rise of a liquid in a mass of tailings. Capillary stresses result from unsatisfied molecular attractions at an atomic level, but more practically, it is a result of surface tension. On the tailing deposit, as evaporation of the process water commences, surface particles begin to dry and immediately menisci form in the pore spaces between the particles. As menisci form, surface tension of the fluid is mobilized and tensile stresses develop in the liquid. The developed tension pulls more liquid to the surface and at the same time pulls solid tailing particles together. The latter action compresses the bulk mass and reduces the inter-particle pore sizes. More liquid is extruded, which then evaporates. As pore sizes decrease, the tensile stresses increase. The intensity of the stresses is governed by the fineness and continuity of the capillary passages or pores, as well as the nature of the fluid (usually water). With finer particles and tighter packing of the particles, the capillaries become finer and the tensile stress is greater. Tensile stresses in the fluids result in compressive stresses in the tailings. Very high compressive stresses are produced during drying. Water content decreases and percent solids, strength and density increases. A well-known example of strength gain by desiccation is the increase in strength of potter's clay upon natural drying.

2.4 Consolidation and overconsolidation

Consolidation is when soil particles move into a closer spacing, either by gravity or load, and the voids between the particles are reduced. The voids can either be filled with air or water or a combination of air and water. When fully saturated (S=100%), the voids are completely filled with water.

Been and Sills (1981) describe an experimental and theoretical study of the self-weight consolidation of soils, in which the magnitudes and rates of settlement were attempted to be measured for the transition phase between sediment in suspension and soil with properties described by traditional parameters. A model was developed with the ability to describe consolidation due solely to self-weight of soil, but they assumed that the coefficient of consolidation is constant. This assumption results in linear void ratio/effective stress and permeability/void ratio relationships, which is not found in practice. This illustrates the difficulty in determining the exact magnitude and rate of settlement of tailings in a tailings-dam.

Bjerrum (1954) showed that the normally consolidated Norwegian clays show a linear increase in undrained shear strength with depth, which can be expressed by a constant ratio of undrained shear strength, c_u, to effective overburden pressure, p'. They determined this ratio for various clays and found a close correlation between c_u/p' and the plasticity index (PI). Bjerrum (1954) states that even for normally consolidated clays, it is known that the constant increase in strength with depth does not hold good in the upper crust. This so-called "drying crust" is a well-marked feature of Norwegian clay beds, being characterized by a high shear strength and often with a system of fissures.

Parry's paper on overconsolidation in soft clays (1970), states that most natural deposits of soft clays are lightly overconsolidated as a result of one or more of the following causes:

- Desiccation stresses due to moisture extraction by plant roots, surface evaporation etc.
- Secondary or delayed consolidation
- Changes in static groundwater level.

He further points out that even a small degree of overconsolidation has a significant effect on the settlement of the material. In particular settlements are small if the equivalent critical pressure (i.e. the pre-consolidation pressure) is not exceeded, but may be very large if this pressure is exceeded.

Parry (1970) further states that the consolidated undrained shear strength of a lightly overconsolidated clay is almost constant and independent of consolidation pressure until the consolidation pressure reaches a value equal to the past maximum consolidation stress σ'_m. Once the value of σ'_m is reached, the soil becomes normally consolidated and the undrained shear strength increases linearly with consolidation pressure (similar to Bjerrum, 1954).

2.5 Moisture interaction with clays and clay minerals.

Gillott (1987) notes that the properties of clays are affected by the total water content and by the energy with which moisture is held. The moisture content affects the consistency, strength and density, which can be achieved by a given compactive effort. The energy with which moisture is held influences volume change characteristics, such as are shown on consolidation, drying, moisture uptake, freezing and other properties such as hydraulic conductivity, etc. which are affected by moisture migration and permeability. Moisture migration may be directly responsible for changes in the volumetric or linear dimensions of clays. The magnitude of the dimensional change depends upon the interrelated variables of geological history, fabric, mineralogical composition and nature of exchange ions. Moisture movements occur because internal gradients generate forces within the clay-water system. Gradients arise from variations in the temperature, extent of saturation, magnitude of desiccation, chemical composition and concentration of pore solutions.

The water in soils is retained in very irregular and tortuous channels and pore spaces. The fabric of the soil and the particle size of the minerals exert a large influence on the nature of the forces.

Robinsky (1999) showed that for every tailings disposal operation, there is a unique relationship between consistency and deposition slope. From experiments, it was generally found that the finer the tailings-particles, the lower the consistency (percent solids by mass) required to attain a particular deposition slope. Thus the finer the particles, the steeper the deposition slope. This is explained as follows. The finer the gradation of tailings, the greater specific surface of the particles. The greater the surface to be wetted, the more liquid is molecularly adsorbed or tied-up with the particles and which thus cannot serve to dilute the slurry.

2.6 Conventional Disposal and Thickened Tailings Disposal

The following section is a short summary on thickened tailings from the book by Robinsky, (1999). It will be shown that the behavior of the Red Berea sand tailings, as obtained from the Richards Bay area, behaves very similarly to a thickened tailings deposit. The profile used in the stability analysis in Chapter 6, is also based on the method used for construction of tailings dams with thickened tailings.

Before the process of thickened tailings disposal (TTD) is discussed, it is necessary to briefly describe the typical conventional method of tailings disposal.

Conventional tailings disposal developed from a need to dispose of an unwanted product at the least cost. The simplest method consists of one or more man-made perimeter dam walls, in which tailings is deposited with a large quantity of process water. Generally the tailings is spigotted from the perimeter walls, forming a slightly concave deposit that fills with liquid and forms a settling pond. The clarified water is reclaimed and put back into the process or treated and released into the environment. As the pond fills, the perimeter walls are raised higher above the original ground surface. Sometimes the wall is raised using the coarse fraction of the tailings.

During the mining operation phase, the dam is maintained and inspected and seepage water is reclaimed, although, not all seepage can be stopped. When the operation is completed, the side surfaces of the dam are rehabilitated by vegetation, but the top surface is usually difficult to drain and reclaim and is often a seasonally wet surface. The seepage will continue indefinitely.

The following is a list of goals and critical observations for land based tailings disposal operations:

- All structures connected with the system must be built to last in perpetuity.
- Maintenance or treatment of the effluent in perpetuity is not acceptable – the site must be left in a "walk-away" condition.
- No standing water should be permitted on an elevated tailings deposit.
- Tailings should be deposited to a self-draining slope to increase the density and strength of the deposit.
- Seepage below the deposit must be reduced below tolerable limits.

If the volume of water used with conventional tailing disposal can be reduced, the last two of the above-mentioned aspects will fall away. The slope can be increased with a lower water content, resulting in a more economical deposit. The seepage will also reduce due a lower volume of water seeping out of the deposit.

In conventional disposal, only a minor amount of thickening of tailings is done at the plant. In most base metal mines, the concentration, or percent solids at discharge (mass of solids/total mass) x100, would be 30 to 40 percent. With extremely fine tailings, the percent solids may be as low as 8 to 15 percent. This implies that on average, the volume of solids may take only 3 to 6 percent of the volume in the tailings discharge line. Therefore an enormous amount of water is pumped to the tailings dam, only to be pumped back to the plant, or to be lost due to seepage and evaporation.

There are several reasons for the large quantity of liquid; some of them are discussed here. Firstly, the ore separation process requires a large volume of liquid. Secondly, to prevent segregation and settlement of solids in the discharge line, large volumes and high velocities are required. To maintain high velocities, centrifugal pumps are preferred; they are less expensive than positive displacements pumps, which are used in TTD. Finally, the removal of some of the process water before disposal would result in extra costs.

Conventional tailings dam failures are environmentally disastrous, not because of the dam itself, but due to the large volume of very loose unconsolidated material and a great deal of water. When the dam fails, the contents may liquefy and flow through the breach. If the dam itself is also built of the tailings material, it too can liquefy and fail. This liquefied mass of material can flow long distances and cause major damage, both financially and environmentally.

Two examples of disastrous failures were at Bafokeng Platinum mine near Rustenburg, South Africa, in November 1974 and at Merriespruit near Virginia, South Africa, in February 1994, (Blight 1997).

The reason why the tailings are so loose, is because they were deposited with a large volume of water, which forms a pond. The tailings settle slowly through the water to form a loose structure. It does not take a large force to liquefy this loose structure. The tailings preferably need a drained area, where they can dry out and gain some strength. If the process water was stored somewhere else, only a local collapse of a dam can occur.

Most conventional tailing dams are capped with a permanent and elevated settling pond. The purpose of the pond is to allow the settling of the tailings out of the water, so that it can be decanted and put back into the process. The presence of the settling pond is the reason for most failures of the conventional disposal method (Robinsky, 1999)

The primary difference between thickened and conventional disposal systems is that in conventional systems, the only strengthening mechanism is by self-weight consolidation of buoyant-weight tailings beneath the tailings pond, which is a very inefficient method of consolidation. The material is actually continually loose and the danger of failure is always present.

Stability of a tailings-deposit itself depends on the strength or resistance to shear of the deposited material. A *thickened tailings* deposit is usually a dense, self-supporting, stable deposit, which reduces or eliminates the need for perimeter dams. It is probably resistant to liquefaction and environmentally superior to conventional systems. Large superimposed liquid ponds are also eliminated.

With thickened tailings, the tailings typically develop higher shear strengths than with conventional operations. Having been designed to desiccate, layer upon layer, liquefaction becomes improbable. The stability analyses in Chapter 6, were done for a section of a typical thickened tailings profile.

There are three mechanisms that act to consolidate and strengthen a thickened tailings deposit. All three occur simultaneously at the disposal site, but with a difference in magnitude, depending on the time after deposition. They are:

- Initial short term separation-consolidation
- Strength gain by desiccation
- Long-term consolidation by self-weight.

Before desiccation can commence, a small amount of process liquid is extruded from the freshly placed slurry as the particles settle. This is termed the 'initial short-term separation-consolidation'. The second mechanism, desiccation, is often the most important. The drying process (desiccation) was explained in detail in Section 2.3. The long-term consolidation is caused by the pore liquid being slowly squeezed out as the soil particles move closer together under the increasing self-weight.

When tailings dry and shrink, the surface layers crack. Finer particles and lower slurry concentration results in greater volume changes, and larger shrinkage cracks. Sometimes the blocks that form are curled slightly upward, breaking away from the underlying previously placed layer of tailings. This happens when surface evaporation is very rapid, tailings are extremely fine and hydraulic segregation has occurred after deposition (Robinsky, 1999).

2.7. Validity of Stability Analysis for Tailings Embankments

As described by Kealy and Soderberg (1969), stability analyses have traditionally been conducted for tailings embankments using techniques originally developed for analysis of natural slopes and conventional water retention dam slopes. For downstream-type tailings embankments, the analytical procedures differ little in validity from those used in conventional applications. However, where hydraulically placed tailings form part of the embankment, (as with upstream and, to a lesser extent, centerline embankments), classical analytical procedures, while still routinely applied, must be used with more caution (Vick, 1950).

Johnson (1975) notes that the application of analytical techniques to stability of even conventional water retention dams must be regarded as at least partially empirical. The link between analytical models and actual slope behavior may be even more tenuous for tailings dams than for conventional water dams if the origins and assumptions of the classical procedures are not thoroughly evaluated.

2.8 Conclusion

The related topics given in Chapter 2 give a background of the principles that relate to drying and therefore strength gain of tailings. The reason for the strength gain and the magnitude of strength gain depends on a number of properties such as the type of tailings, the particle size, the process liquid as well as the conditions (temperature, weather, etc.) surrounding the tailings, etc.

This chapter shows that tailings desiccate and consolidate and this results in strength and stability increase. Suction, capillary action and moisture interaction of clays and minerals contribute to this in different ways and magnitudes.

For this project report it will be assumed that the tailings forms a "drying crust" with a consolidated base layer with a constant ratio of undrained shear strength to effective overburden pressure. This assumption is valid because the phenomenon was also observed at the Pilot Plant in Richards Bay.

3 SHEAR STRENGTH AND METHODS OF DETERMINING SHEAR STRENGTH

3.1 SHEAR STRENGTH

3.1.1 Introduction

This section describes the term shear strength and explains the Mohr Coulomb failure criteria. It shows how this criterion is adjusted for saturated soils with the effective stress concept. The methods of determining the shear strength are noted.

3.1.2 Mohr Coulomb Failure Criteria

Stated qualitatively, soil *strength* is the capacity of a soil to withstand forces without experiencing failure, whether by rupture, fragmentation or flow.

Soil strength is easy to define, but to measure it is far from easy. Soil strength is a variable property that often changes during the measurement process, as the deformed body of soil might either decrease or increase its resistance to further deformation. Resistance to applied stresses can be characterized in terms of two parameters: *cohesiveness*, the bonding of the soil particles which must be broken, and the *angle of internal friction*, which is the resistance when soil is forced to slide over soil.

One type of strength is *shear strength*, which is induced by gradually increasing the lateral, or tangential stress until failure occurs. In other words, the shear strength of a soil is the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it. The nature of the shearing resistance must be understood to be able to analyze soil stability problems such as bearing capacity, slope stability and lateral pressure on earth-retaining structures.

For most soil mechanics problems the shear stress (τ_f) on the failure plane can be approximated as a linear function of the normal stress (σ), which can be written as:

$$\tau_f = c' + \sigma' \tan \phi' \qquad (3.1)$$

Where: τ_f = shear stress on failure plain

c' = apparent cohesion and

ϕ' = angle of internal friction (phi)

This relationship is called the *Mohr-Coulomb failure criterion.* Equation 3.1 is illustrated in Figure 3.1. Figure 3.1 shows s1 (σ1) and s3 (σ3), which is the maximum and minimum normal stresses acting on the soil element at time of failure. The difference between these two stresses is called the deviator stress. The deviator stress will be used with the shear box test results.

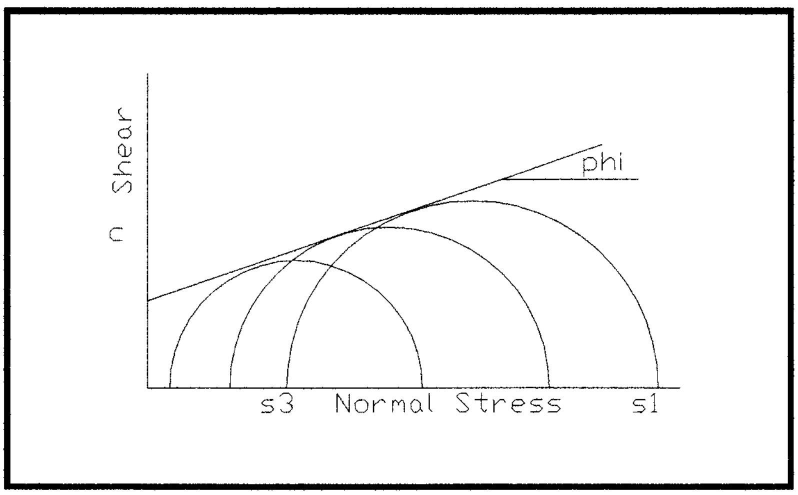

Figure 3.1: Mohr circle on Normal stress – Shear stress graph

3.1.3 Saturated Soil

For saturated soil, the principal of effective stress, as suggested by Terzaghi (1936), is valid:

$$\sigma = \sigma' + u \qquad (3.2)$$

where σ = total stress

σ' = effective stress

u = pore water pressure

The solids carry the effective stress, therefore Equation 3.1 can be rewritten as:

$$\tau_f = c + (\sigma - u)\tan\phi = c + \sigma'\tan\phi \qquad (3.3)$$

Thus for saturated soils the shear stress is determined by the effective stress as well as the magnitude of the pore water pressure. For unsaturated soils the pore-air pressure must also be considered. For the purpose of this project report, all calculations will be done for total stresses, because no pore water pressure measurements were done.

3.1.4 Determination of Shear Strength Parameters

The shear strength parameters can be determined in the laboratory by primarily two types of tests: shear box test and the triaxial test. The triaxial test was not used for this project and is therefore not discussed.

The shear vane apparatus is used to determine the undrained shear strength of cohesive soils in the field. For this project, samples were prepared and tested in the laboratory with the hand held shear vane apparatus.

3.1.5 Concluding remarks

The shear strength is used to describe the state of failure for a material. This shear strength can be determined and interpreted with different methods. These methods, namely the shear box and the shear vane, are discussed in Section 3.2 and Section 3.3 respectively.

3.2 THE SHEAR BOX

3.2.1 Introduction

This section describes the shear box test apparatus and the interpretation of its results.

3.2.2 Shear box apparatus

The shear box apparatus is illustrated in Figure 3.2. It consists of a sample (usually 100mmx100mm), that is loaded with a chosen normal force. The sample is sheared on a horizontal plane.

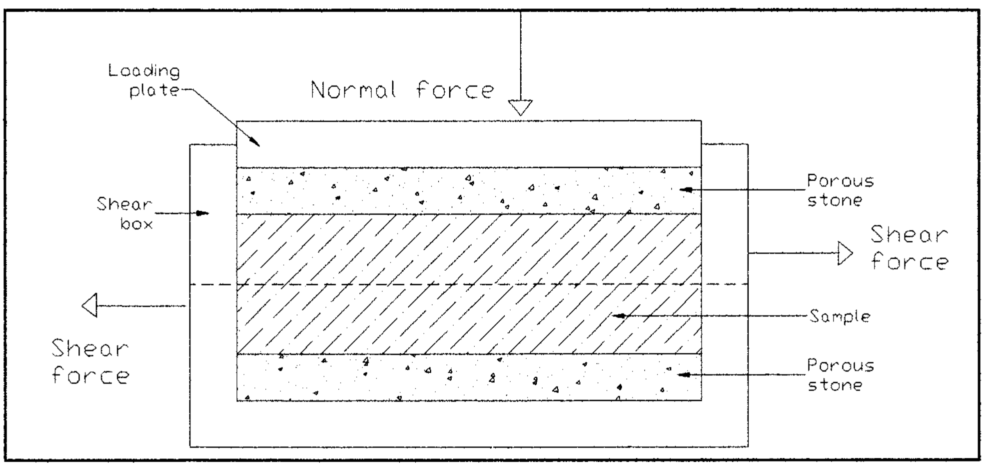

Figure 3.2: Diagram of shear box apparatus

The failure surface is therefore forced to occur at a predetermined location and is not necessarily the weakest plane.

3.2.3 Interpretation of test results

There are three different types of direct shear tests that can be performed:

- Quick test. Shear is begun before the sample consolidates. Excess pore pressure may develop with cohesive saturated samples.
- Consolidated quick test. Normal force is applied and the settlement is observed. Shear is only started after settlement has stopped.
- Consolidated slow test. Normal force is applied and shear is only started after settlement has stopped. Shear is applied so slowly that no excess pore pressure develops in the sample.

For cohesionless soil the results of the three tests will be the same, but for cohesive soils the results will depend on the test method and the degree of saturation.

For this project report the consolidated quick test was used for the following reasons:

- The sample should be consolidated in the field, by the time failure is expected.
- The shearing rate is quick, which is similar to, firstly what is expected in the field and secondly it is similar to the shear vane apparatus. Therefore the results can be compared as shown in a later chapter.

For a specific normal stress, the following information is observed and/or calculated during the test:

- Density
- Moisture content
- Loading rate
- Vertical displacement
- Horizontal displacement
- Shear stress

A graph of horizontal displacement against shear stress can be plotted as well as a graph of horizontal against vertical displacement. These two graphs give an indication of whether the material tested is cohesive or cohesionless. When the material is cohesive the graphs will indicate whether it is normally consolidated or overconsolidated. When the material is cohesionless the graphs will indicate whether it is loose or dense.

To obtain the strength parameters cohesion (c) and angle of internal friction (ϕ), a minimum of three drained tests must be done at three different normal stresses. This will give a graph as shown in Figure 3.1. If the pore pressure measurements were taken, the undrained shear strength (c_u) and the angle of internal friction (ϕ') can be read off the graph.

3.2.4 Concluding remarks

The shear box tests were done to validate the shear vane test results. As shown in Chapter 4, the results are very similar and it can be assumed that the shear vane results are reliable.

3.3. THE SHEAR VANE

3.3.1 Introduction

This section describes the shear vane apparatus. The history is given shortly. The apparatus, the test procedure and the interpretation are discussed.

3.3.2 History

Early geotechnical engineers found it difficult to determine the shear strength of very soft and sensitive clays due to the disturbance induced by poor-quality samplers. This led to the development of the vane shear test, which made it possible to determine the in-situ undrained shear strength and sensitivity of soft clays.

John Olsson, the secretary of the Swedish Geotechnical Commission, designed the first vane borer. It was used in 1919 during the construction of the Lidingoe Bridge in Stockholm. One of the problems on the site was the buckling of long piles needed to support the bridge. The soil consisted of a very soft clay with a thickness of up to 40m. John Olsson had to determine the coefficient of horizontal subgrade reaction of the clay. He designed a vane borer, consisting of two plates, which were attached to a system of pipes and a torque wrench. To compensate for friction, two tests were done at the same depth with different sized plates. It was found that rate of rotation was important and that displacements could be large if the load was applied very slowly. The author of the report concluded: "The apparatus was found to be very well suited for its purpose and it is highly desirable that further investigation of the strength of the clay should be performed with the method now described or a similar method."

A surface vane borer was used in England as early as 1944 by the Army Operational Research Group to investigate the mobility of military vehicles on the suggestion of the Soil Mechanics Section of the Building Research Station. A laboratory vane apparatus was also developed.

The vane borer as used today was presented for the first time by Lyman Carlsson (Carlson) in 1948 at the Second International Conference in Rotterdam. A report on a more advanced device was published two years later (Cadling & Odenstad, 1950).

Skempton (1948) presented a paper on vain tests in a thick deposit of soft clay in the alluvial plain of the River Forth, near Grangemouth. The vane apparatus as described in this paper was used in the bottom of a borehole and is very similar to the vane used today by most geotechnical contractors.

3.3.3 Apparatus

The vane shear test consists of pushing a four bladed (cruciform) vane, mounted on a solid rod, into the soil and rotating it from the surface. The tests may be carried out either in the field or in the laboratory. In the field they may be carried out either from ground level or from the base of a borehole. The apparatus used for the tests done for this report was a hand held Geonor shear vane as illustrated in Figure 3.3. Figure 3.3 shows the three vanes, the rod for determining shaft friction and the extension rods. The vane dimensions, testing procedure and standards are given in Appendix B1 and B2.

Figure 3.3: Geonor shear vane

3.3.4 Interpretation

The vane is used to determine the peak and residual (remoulded) undrained shear strength. The undrained strength is derived on the basis of the following assumptions:

- Penetration of the vane causes negligible disturbance, both in terms of changes in effective stress and shear distortion
- No drainage occurs before or during shear
- The soil is isotropic and homogeneous
- The soil fails on a cylindrical shear surface
- The diameter of the shear surface is equal to the width of the vane blades
- At peak and remoulded strength there is a uniform shear stress distribution across the shear surface, as shown in Figure 3.4.
- There is no progressive failure, so that at the maximum torque the shear stress at all points on the shear surface is equal to the undrained shear strength, c_u. This is illustrated in Figure 3.5.

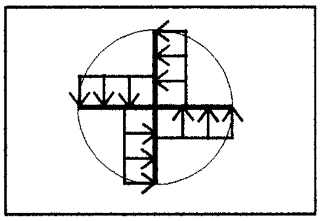

Figure 3.4: Uniform shear stress distribution

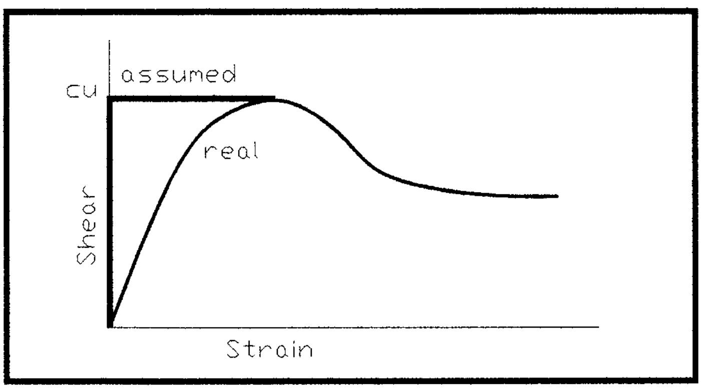

Figure 3.5: Assumed shear-strain relationship

When a uniform shear stress distribution is assumed, as illustrated in Figure 3.4, the maximum torque (T) is:

For a vane blade where H =2D

$$T = 3.667.D^3 c_u \tag{3.4}$$

$$T = \frac{\pi.D^2.H.c_u}{2} + 2\int_0^{\frac{D}{2}} 2\pi r.\delta r.r.c_u$$

$$= \frac{\pi.D^2.H.c_u}{2} + \left[\frac{4\pi r^3}{3}.c_u\right]_0^{\frac{D}{2}}$$

$$= \frac{\pi.D^2.H}{2}.\left[1 + \frac{D}{3H}\right].c_u \tag{3.5}$$

If it is assumed that the shear stress mobilized by the soil is linearly proportional to displacement, up to failure, then another simple assumption (Skempton, 1948), that the shear stress on the top and bottom of the cylindrical shear surface has a triangular distribution (as shown in Figure 3.6), is sometimes adopted.

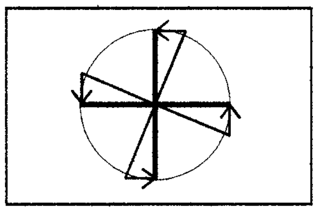

Figure 3.6: Triangular stress distribution

For the triangular stress distribution, the torque (T) is as follows:

$$= \frac{\pi.D^2.H}{2}.\left[1 + \frac{D}{4H}\right].c_u \tag{3.6}$$

For a vane blade where H = 2D

$$T = 3.53.D^3 c_u \tag{3.7}$$

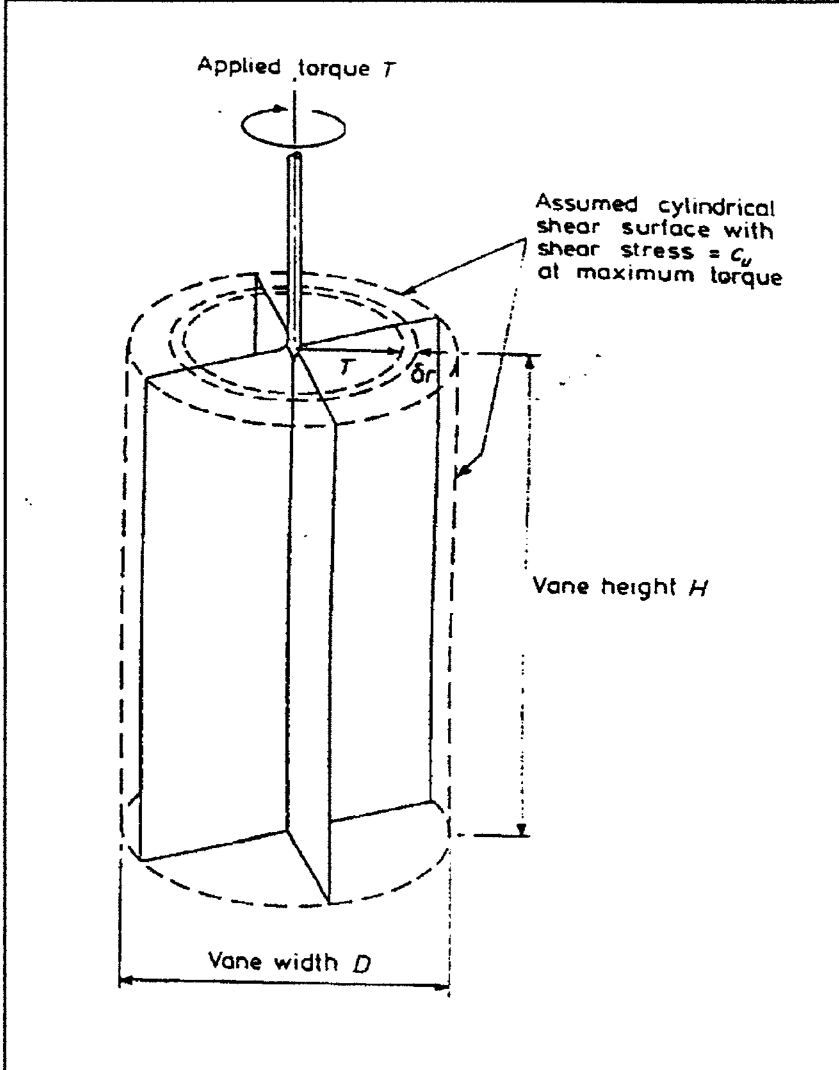

Figure 3.7: Dimensions of the vane

This gives only 4% difference in shear strength from that obtained using the uniform assumption. The dimensions used in the calculations are explained in Figure 3.7.

3.3.6 Discussion

The results of the vane shear test may be influenced by many factors, namely:

- Type of soil, especially when permeable fabric exists

- Strength anisotropy i.e. horizontal and vertical strengths are unequal
- Disturbance due to insertion of vane
- Rate of rotation or strain rate
- Time lapse between insertion of vane and beginning of test, dissipation of water pressure can take place
- Progressive or instantaneous failure of the soil around the vane

A further aspect of the vane test that is discussed by Skempton (1948) is the reliability of the vane test results compared to unconfined compression tests and undrained triaxial tests. In his paper it is shown that the in-situ vane test and the laboratory tests give the same result up to a depth of 15m, but from then on the vane test gives increasingly higher strength values than the laboratory tests. It was concluded that the sampling techniques for laboratory specimens are not always reliable and that the shear vane gives the most accurate results.

3.3.7 Concluding remarks

The shear vane apparatus is an easy-to-use method of determining the undrained shear strength, but as with many other geotechnical tests, the results are very user dependent. If the shear vane is used correctly and consistently it should give accurate values for undrained shear strength.

4 LABORATORY WORK

4.1 Introduction

This chapter sets out the experimental work done in the laboratory. Each section will explain the experimental procedure, results obtained and the conclusions that were made, for the different experiments done.

4.2 Description of tailings and experiments

The tailings consist of Red Berea Sand with a large clay content. The geology of the area consists of Miocene deposits of red clayey sand, classified as Berea Formation. The mineralogy of the fines mainly consist of quartz, kaolinite, chlorite and haematite, (Williamson, 1997). According to tests done by Williamson (1997) the tailings have the following geotechnical properties:

Table 4.1: Geotechnical properties (Williamson, 1997)

Angle of internal friction	ϕ	±28°
Cohesion	c	±5kPa
Permeability	k	10^{-6} to 10^{-8} cm/s
Specific gravity of solids	Gs	2.8
In-situ dry density of deposited fines	ρ_d	800-900kg/m^3

The Particle size analysis and Atterberg Limits of a typical sample is shown in the Section 4.3.

Tailings were collected at the pilot plant operation in Richards Bay and transported to the laboratory in 25l drums. The tailings were at ±28% solids concentration when they were collected. During transportation and storage the tailings settled out and the water could be decanted before the experiments were conducted. The water was kept for experimental purposes, to be used instead of distilled water. The tailings were used for a number of experiments; namely a foundation indicator test, a number of shearbox tests and a variety of measurements with a hand held shear vane apparatus at different moisture contents.

4.3 Particle size analysis and Atterberg limits

4.3.1 Introduction

This section gives the particle size analysis and Atterberg limits for comparison purposes with other materials.

4.3.2 Particle size analysis and Atterberg Limits

These parameters were determined with standard geotechnical testing methods. Table 4.2 on the next page contains all the relevant parameters.

4.3.3 Conclusion

The tailings can be classified as silty clay, which explains the undrained behavior during quick loading, as shown in the shear box test results. It can also explain the large strengths developed by the tailings as it desiccates – the clays can develop very large inter-particle forces or negative pore pressures, as the water is removed from the material.

Table 4.2: Particle size analysis

Material description	Dusky red slurry - silty clay
Screen analysis (mm)	**% Passing**
63.0	100
53.0	100
37.5	100
26.5	100
19.0	100
13.2	100
4.75	100
2.00	100
0.425	100
0.075	95
Hydrometer analysis	
0.040	70
0.027	67
0.013	63
0.005	58
0.002	50
% Clay	50
% Silt	34
% Sand	16
% Gravel	0
Atterberg Limits	
Liquid Limit	55
Plasticity Index	28
Linear Shrinkage	12.5
Shrinkage limit	22
Grading Modulus	0.05
Classification	A-7-6 (18)
Unified Classification	CH

4.4 Moisture Content versus Undrained Shear Strength at low overburden pressure

4.4.1 Procedure

After decanting the water, the undrained shear strength, as well as the moisture content were measured in the 251 drums at different depths, with the XXL shear vane (see Appendix B1).

4.4.2 Results

The data shown in Section A of Appendix C show no large difference in either moisture content nor undrained shear strength with small changes in depth. This was done to determine if the tailings gained any strength due to consolidation over time (±2 months), for small depths.

4.4.3 Conclusion

- It can be concluded that at very low overburden pressures the increase in strength due to consolidation is virtually zero.
- Large overburden pressures will be required to squeeze out the water and increase the strength.
- The tailings at the surface will only gain strength by desiccation due to evaporation and not by consolidation due to small overburden pressures.
- These tests give an accurate value for undrained shear strength at high moisture content. The test results are illustrated in the upper part of Figure 4.1.

4.5 Moisture Content versus Undrained Shear Strength

4.5.1 Procedure

The undrained shear strength of the tailings had to be determined at different moisture contents. To achieve this, the tailings had to be dried out, but due to the extensive cracking of the tailings, (which usually left small intact blocks, which could not be used for testing purposes), as well as the size of the vanes, the tailings had to be poured into 10l containers. This resulted in larger blocks, which could be tested with more accuracy.

Ten samples were used to determine the moisture content-shear strength relationship. The data is shown in Section B, Appendix C.

Samples 2,5,9,10,14,15,W1, W2, W3 and W4 (Appendix C) were dried out for different periods of time ranging from one month to almost two months. The strength readings and moisture content calculations were done at different intervals to obtain a large variation of moisture content values. Samples W1 to W5, were prepared in deeper holders than the other samples, to accommodate the XXL shear vane. The purpose of these samples and the XXL shear vane were to determine the very low shear strengths at high moisture content (similar to previous section). The results are shown in Figure 4.1.

In the laboratory the samples took many days to lose moisture, therefore a fan was used to speed up the drying process on some of the samples (2 and 14).

4.5.2 Results

- At a moisture content of ±200%, the undrained shear strength was close to 0kPa, as shown on Figure 4.1. With a decrease in moisture content, the undrained shear strength increased. The trendline's equation is shown in Equation 4.1:

$$y = -17.6 * \ln(x) + 125.3 \qquad (4.1)$$

with correlation $R^2 = 0.8233$

y = moisture content (%)

x = undrained shear strength (kPa)

The correlation indicates an acceptable grouping of results, but it is probably not close to 1 due to the few outliers, which were found with the very dry material.

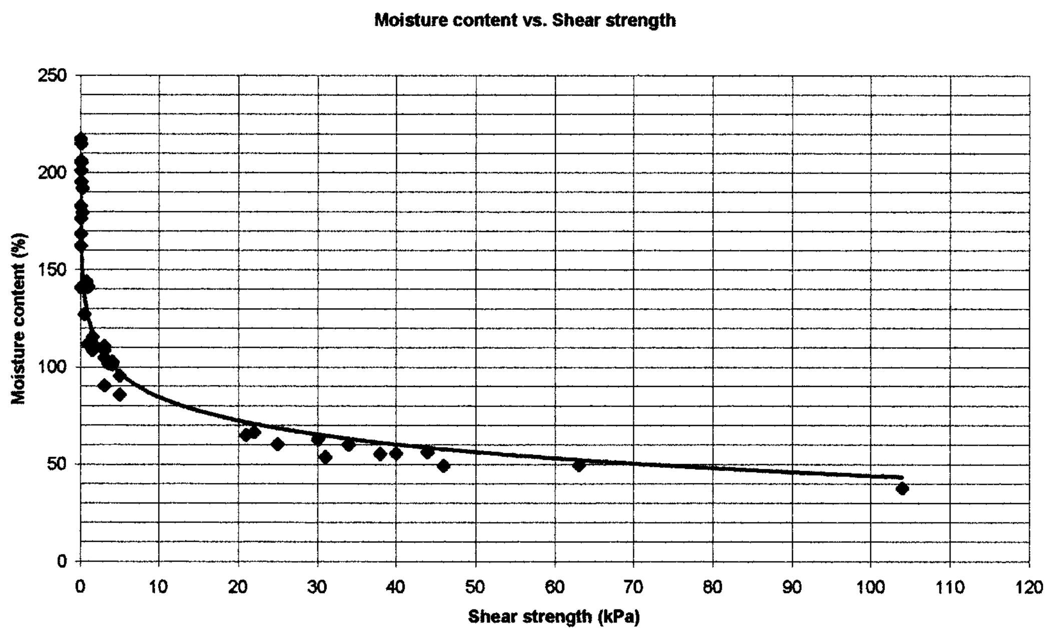

Figure 4.1 Moisture Content vs. Undrained Shear Strength

(Figure 4.1 is repeated at the end of Appendix C)

The fan achieved large reductions in moisture, see for example samples 1,2,13 and 14, as shown in Figure 4.2, (data for samples 1 and 13 is shown in Section C, Appendix C). The moisture content reductions were uniform through the samples, but the samples were smaller than bricks. The average moisture content reduction for a sample was ±40% over three days (13% per day) with the fan, compared to ±100% over one month (3% per day) without the fan.

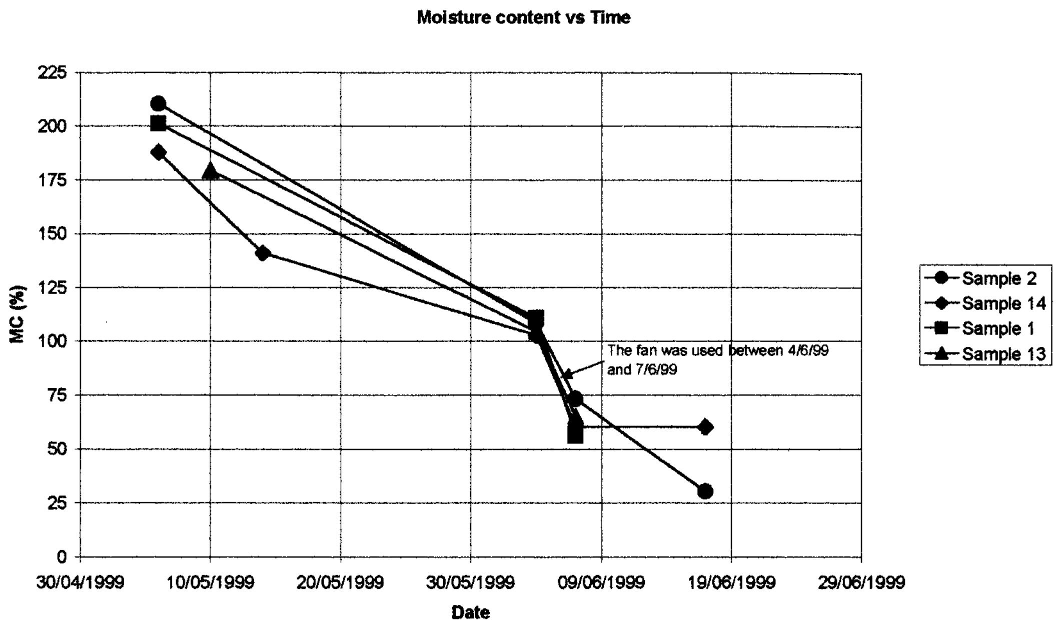

Figure 4.2: Reduction in moisture content due to fan drying

4.5.3 Conclusion

- The tailings have almost no strength at high moisture contents and only start to gain strength from ±110% moisture content, when desiccating.
- From ±100% moisture content, the tailings start to gain strength rapidly. Figure 4.1 shows a clear relationship (Equation 4.1) between moisture content and undrained shear strength.
- Figure 4.2 indicates that the use of a fan, which is similar to wind on a dam, will drastically reduce the moisture content and therefore it can be concluded that the tailings will gain strength quicker when exposed to windy conditions.

4.6 Moisture Content versus Undrained Shear Strength after Re-wetting the Samples

4.6.1 Procedure

Samples 1,3,4,6 and 13 were left to dry out and were then re-wetted. The shear strength and moisture content were measured after the samples were re-wetted. This gives an indication of the increase in moisture content and reduction of strength when tailings are exposed to wetter conditions than the existing condition. An example of this situation, is when a layer of tailings is deposited on a dried layer or after a rain event.

4.6.2 Results

The test data are shown in Section C of Appendix C. Figure 4.3 shows moisture content over time for four samples. The addition of water was on 7/6/1999, which shows an increase in moisture content, which resulted in a decrease in

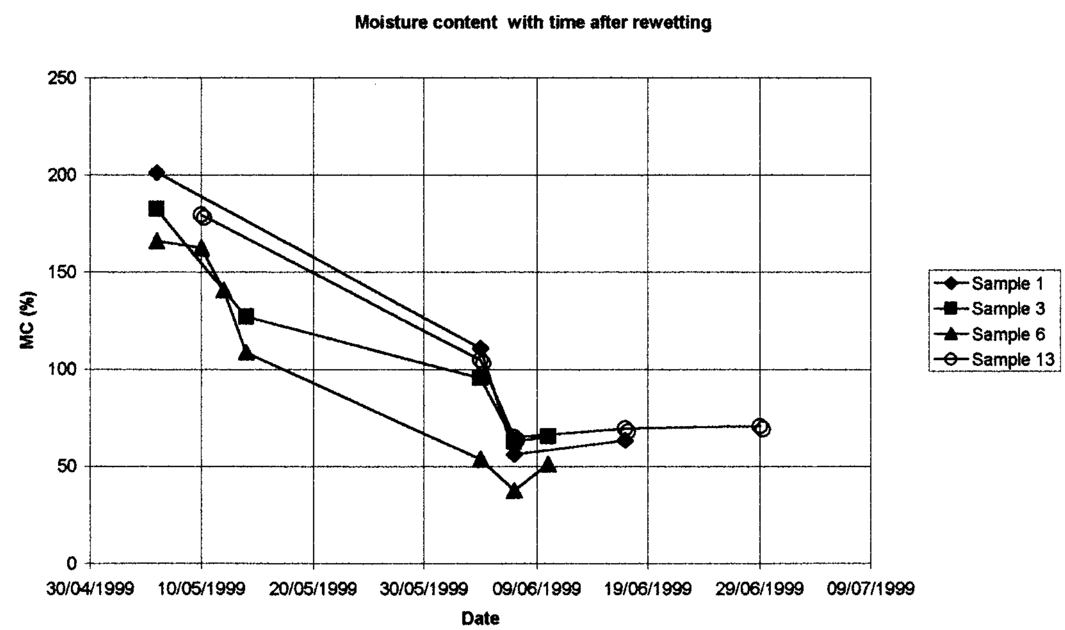

Figure 4.3 Moisture content after re-wetting the samples

undrained shear strength. The slight increase in moisture content shown in Figure 4.3, indicates that the samples did absorb some of the water. Measurement of the undrained shear strength was difficult and inaccurate due to the existing cracks in the samples, but the trend showed a significant decrease in strength after re-wetting, Refer to Section C, Appendix B. Sample 6 reduced from ±100kPa to below 30kPa in 3 days time compared to Sample 3, which reduced from 63kPa to 30kPa in the same period of time.

4.6.3 Conclusion

- From Figure 4.3 it is concluded that the loss of strength is more when the moisture content before re-wetting is lower.
- With normally consolidated soils, the moisture content and shear strength have a certain relationship with decrease in moisture content (with load increase). This relationship changes when the moisture content increases (overconsolidation). Therefore the relationship between moisture content and shear strength is dependent on whether the material is desiccating or increasing in moisture content.
- It can be concluded that it is possible that different layers of the tailings-dam may have different strength values at the same moisture content. The difference in strength will depend on the how long the previous layer has been exposed to the drying elements such as sun and wind.
- For the purpose of this report it will be assumed that the increase in moisture content is small enough to ignore. The decrease in strength is dependent on the strength before re-wetting. It was assumed that if the strength of a layer is less than 30kPa, the decrease in strength after re-wetting is negligible. If the strength is however above 30kPa, the strength will decrease to 30kPa. A value of 30kPa was chosen because the test results showed that below this value the decrease was in fact negligible, but when the strength was even as high as 100kPa, the strength reduced to ±30kPa after re-wetting. During the analyses, it was found that the "upper drying crust" did not reach more than

30kPa strength during the worst case scenario, thus the reduction in strength could be ignored.

- It is recommended that this aspect of strength loss after re-wetting, be investigated in further studies and modeling techniques should be used to determine the exact magnitudes of change.

4.7 Column Test

4.7.1 Procedure

An experiment (referred to as the column test) was done to determine if the overburden pressure has any influence on the undrained shear strength of the tailings at shallow depths. A 300mm inside diameter, 1m high pipe was filled with tailings and left for a few days to settle. The settlement was measured over the drying period. The undrained shear strength and moisture content were measured at different depths.

4.7.2 Results

The test data is shown in Section D of Appendix C.

In this test the tailings started at a height of 900mm and settled 276mm in 22 days. The initial average moisture content was 213% and it dried out to 140%. The undrained shear strength was found to be 1kPa at this moisture content. The settlement is shown in Figure 4.4. The curved line is a trendline of the measured values, which is shown by the straight line.

Due to the high moisture content of the material, the difference in undrained shear strength, with depth, is negligible as shown in Figure 4.5.

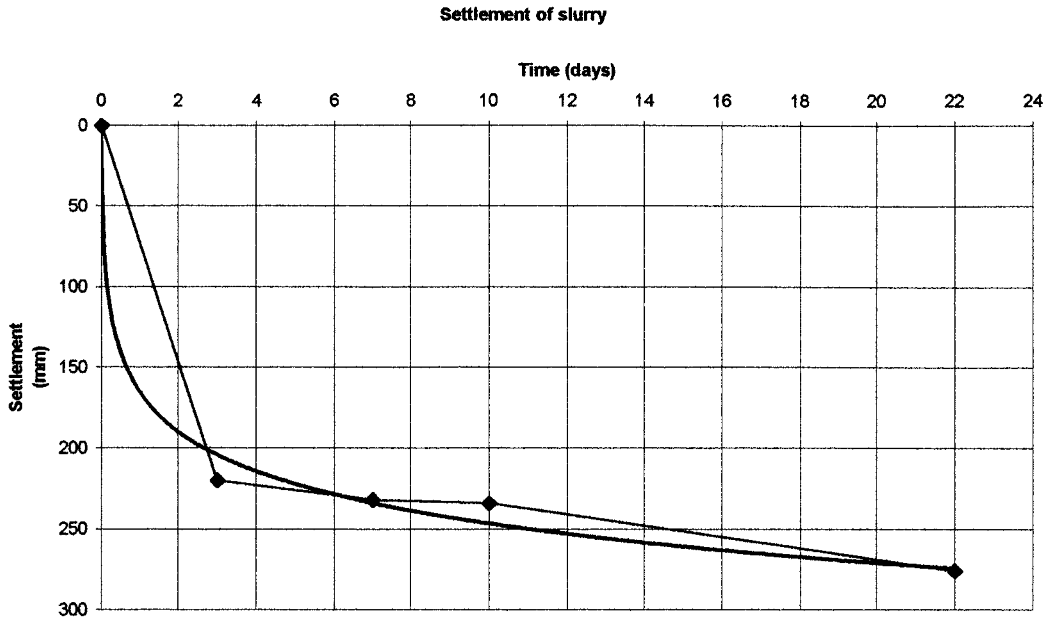

Figure 4.4: Settlement of the slurry in the column test

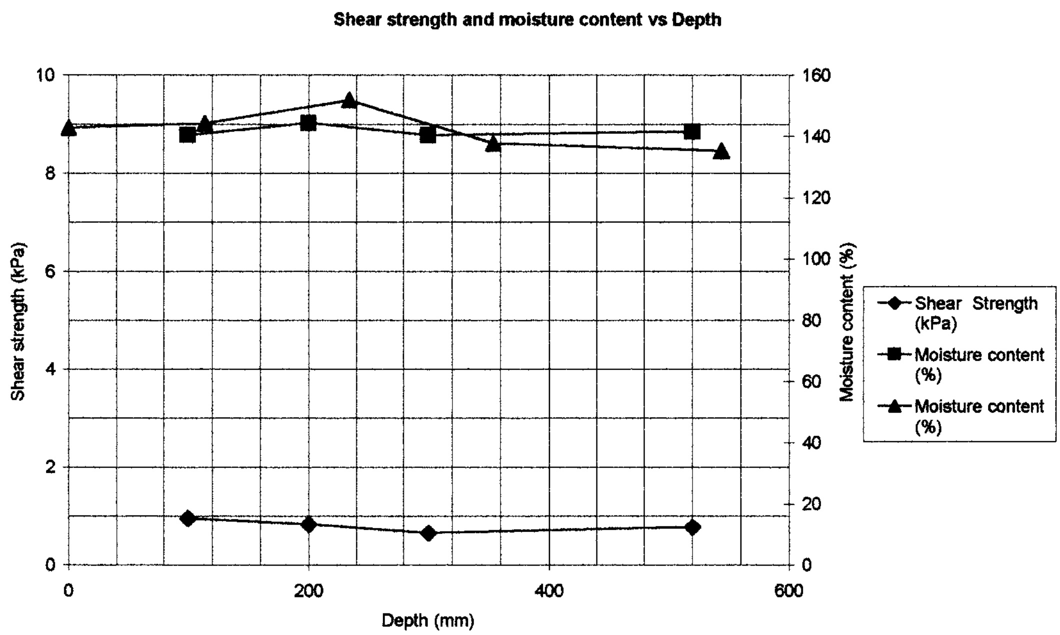

Figure 4.5: Undrained shear strength and moisture content with depth

4.7.3 Conclusion

- The above results are similar to Section 4.4, therefore it can be assumed that the slurry gains most of its strength when it desiccates and only a small percentage when pressure is applied in undrained conditions. This is only true for low depths.
- In Chapter 2, it was noted that there is an "upper drying crust" in which the constant increase in strength with depth does not hold true (Bjerrum, 1954). The above-mentioned results illustrate this point.

4.8 Undrained Shearbox Tests

4.8.1 Procedure

Twenty-seven consolidated undrained shearbox tests were conducted by a commercial laboratory and three more tests were done by the author at a laboratory at the University of Witwatersrand, South Africa, to confirm the findings.

The tests done by the commercial laboratory were done at different normal stresses (100, 200 and 400kPa) and the tests done at Wits were at 50, 100 and 200kPa.

The shear box test results, done by the author, were used to calculate the coefficient of consolidation (c_v). The Square-root-of-Time method (Das, 1994) was used to calculate the c_v value.

A relationship between the normal stress and the moisture content, similar to the Equation 4.1, was derived from the test results.

4.8.2 Results

The results of the three shear box tests done, are given in Appendix D. Figure 4.6 shows the moisture content against shear strength for the commercial tests and the tests done by the author. There were a few outliers, which shown, but they were not taken into account for calculation of the trendlines.

Most of the test results correlate well with the shear vane test results. Figure 4.7 is the combined data of Figure 4.6 and Figure 4.1. It can be seen that the results lie on the same trendline. This confirms that the shearbox tests were done at a rate fast enough for the material to behave undrained.

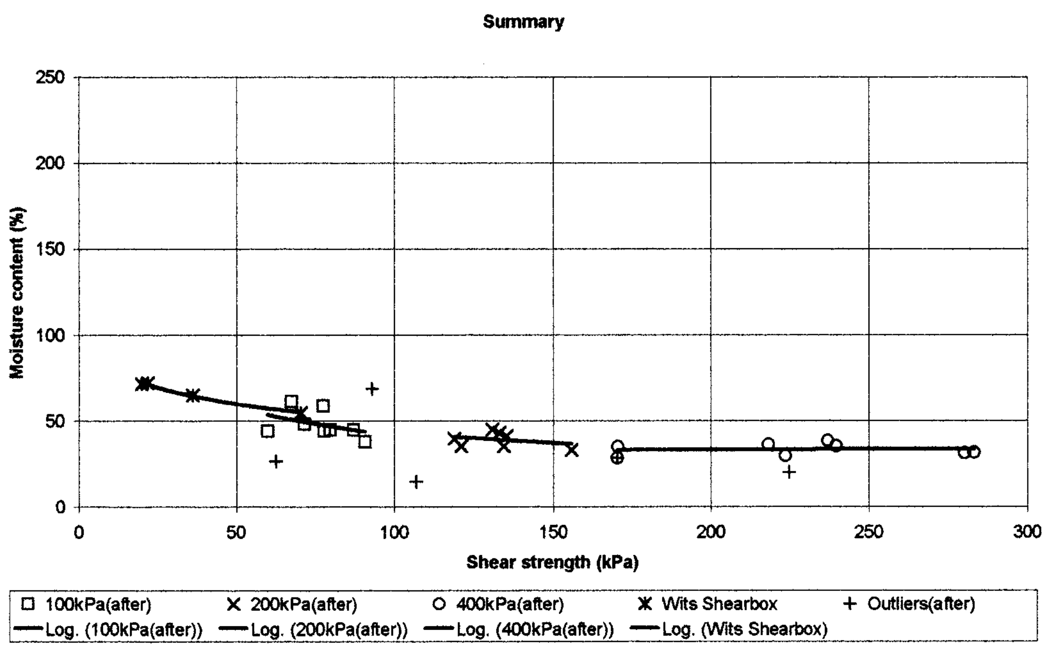

Figure 4.6: Moisture content vs. shear strength from shearbox tests

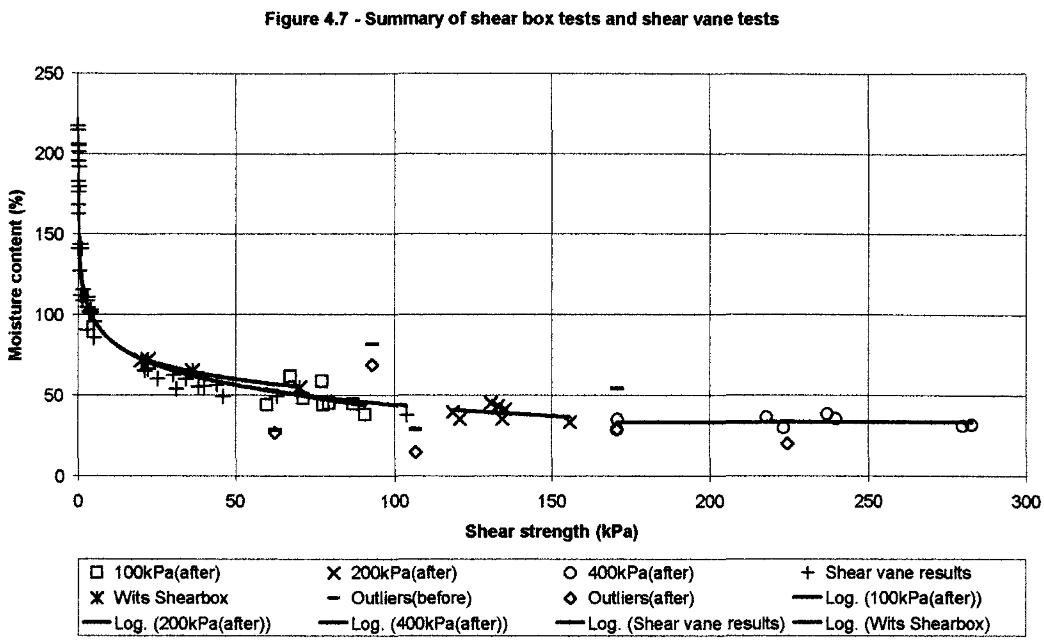

Figure 4.7: Comparison between shear vane and shear box results

(Figure 4.6 and Figure 4.7 are repeated in Appendix E)

The shear box tests done by the author were used to calculate *the coefficient of consolidation* and as was expected, it was different depending on the stress level.

The value of c_v was found to range from 7m²/year at 50kPa normal stress, to 0.4m²/year at 200kPa normal stress. An example of the calculation is shown in Figure 4.8. The results of all the tests are shown in Appendix D

The shear box tests done by the commercial laboratory were not totally consolidated and therefore the coefficient of consolidation could not be determined from these tests. Usually the coefficient of consolidation is determined from conventional oedometer tests or from Rowe-cell tests. These tests could unfortunately not be performed due to time constraints imposed by duration of test and availability of equipment. The Rowe-cell test has to be started by pouring slurry into the apparatus and letting it consolidate over a few weeks, while adding material continuously, as the slurry settle in the Rowe-cell apparatus. After the material has reached some consistency, the test could be started. If the low permeability of the material is considered, it could be assumed that the test could take up to a few weeks to complete. It is recommended to do these test to obtain a more accurate answer for coefficient of consolidation. For the scope of this report the above mentioned values will be assumed accurate.

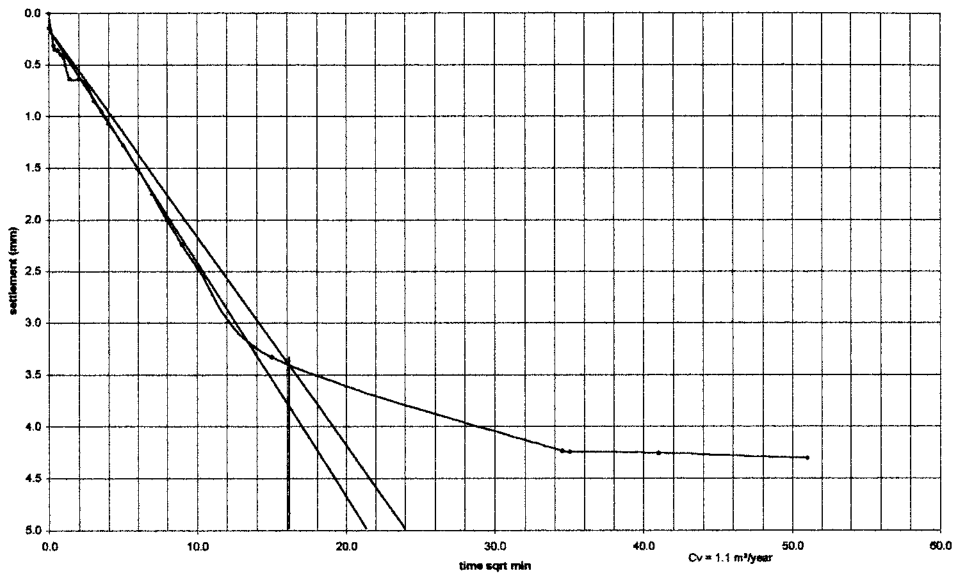

Figure 4.8: Calculation of cv

A relationship between moisture content and normal stress was derived as shown in Figure 4.9. The equation for the relationship is similar to Equation 4.1 and is shown below:

$$y = -12.2\mathrm{Ln}(x) + 119.97 \qquad (4.2)$$

with: $R^2 = 0.9885$ and

y = moisture content (%)

x = effective normal stress (kPa)

This equation can be used when the moisture content needs to be determined at a certain depth. This moisture content can then be used to calculate the undrained shear strength at this depth. These results can be used to draw up a profile for the slope stability analysis.

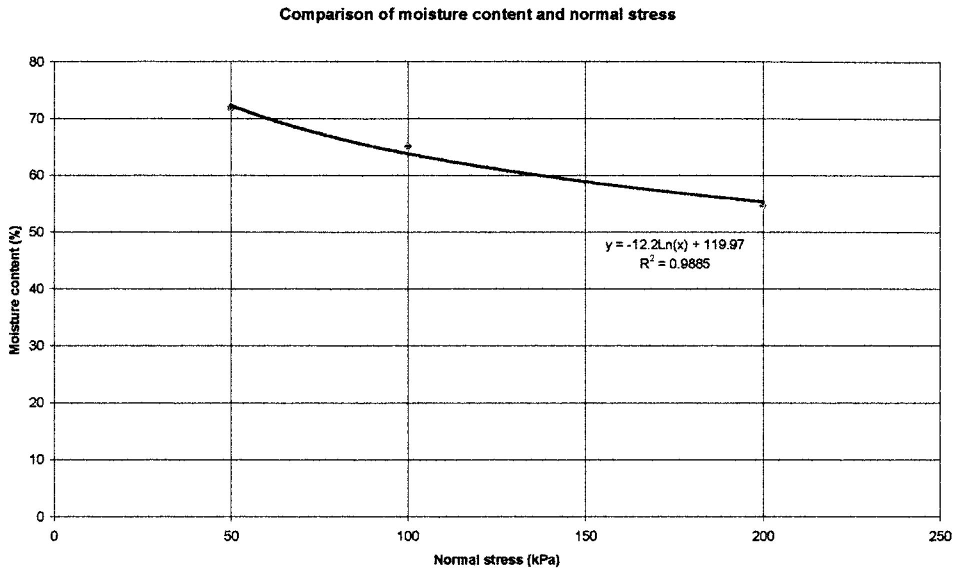

Figure 4.9: Normal stress vs. Moisture content

In conventional consolidation calculations, a void ratio against pressure plot is used. The void ratio is defined as the ratio of volume of voids to the volume of solids. For saturated soils the void ratio is related to the moisture content with the following equation:

$$e = w * Gs \qquad (4.3)$$

e = void ratio
w = moisture content (%)
Gs = Specific weight (2.7)

The data used for Figure 4.9 is shown in Figure 4.10, as a void ratio – log pressure plot.

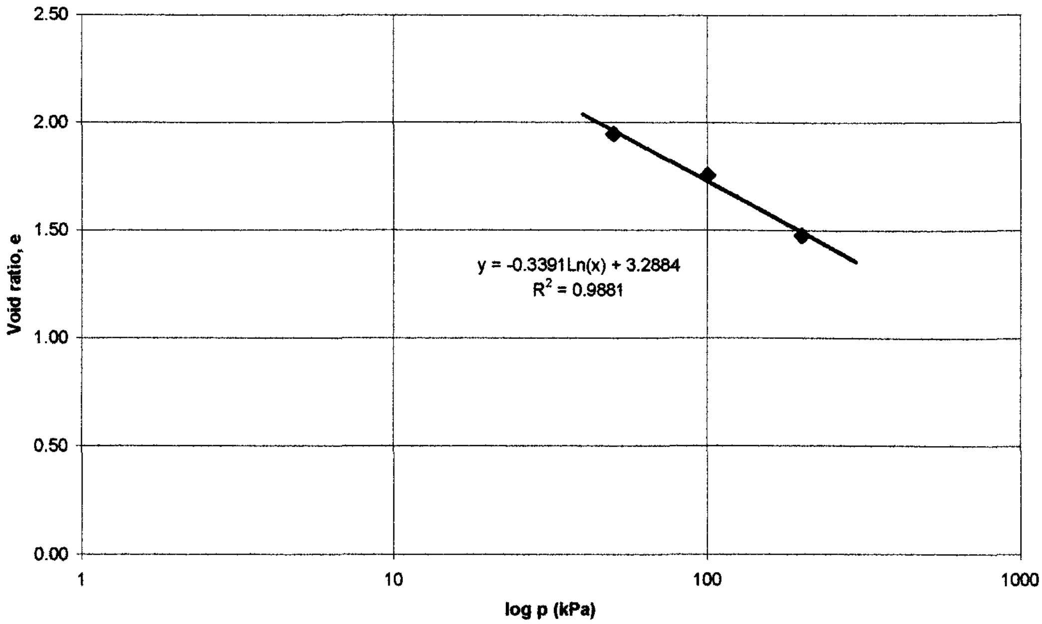

Figure 4.10: Void ratio – log pressure

4.8.3 Conclusion

- The undrained consolidated shear box tests correlate well with the shear vane tests.
- The rate of shear was quick enough for the material to react undrained.
- Coefficient of consolidation values were determined for different normal stresses.
- There is a relationship between the moisture content and the normal stress, which could be used to determine the undrained shear strength at a certain depth, for slope stability purposes.

4.9 Concluding remarks

- There is a relationship between the moisture content and the undrained shear strength, as well as the moisture content and the normal stress. The equations for the two relationships are given in Equation 4.1 and Equation 4.2, respectively.

- The shear vane gave a well-correlated spread of values for the Red Berea sand tailings.
- Both these equations are only applicable for undrained and consolidated conditions.
- The moisture content is drastically reduced by evaporation (Section 4.5) and only slightly reduced by overburden pressure (Section 4.4), at shallow depths. Thus if the drainage path is short, as is the case with the shearbox test, the time for strength increase due to load, is short.

5 INTERPRETATION OF RESULTS

5.1 Introduction

This chapter discusses the findings of the previous chapters and how the results can be interpreted and used for practical stability calculations, as shown in Chapter 6.

5.2 Undrained shear strength in the consolidated layer of the dam.

In Chapter 2 it was noted that Bjerrum (1954) showed that the normally consolidated Norwegian clays show a linear increase in undrained shear strength with depth, which can be expressed by a constant ratio of shear strength over effective overburden pressure, c_u/p'. He determined this ratio for various clays and found a close correlation between c_u/p' and the plasticity index (PI).

In Chapter 4, the PI of the ilminite tailings is shown to be 28. From the data presented by Bjerrum (1954), a normally consolidated Norwegian clay with a PI of 28, will have a c_u/p' ratio of ± 0.23.

Skempton (1948) showed that there is a relationship between the Liquid Limit (LL) and the c_u/p' ratio. From the data presented for five different clays, a clay with a LL of 55 and a PI of 29, will have a c_u/p' ratio of ± 0.25. This similar to the findings presented by Bjerrum (1954).

When a trendline is drawn through the data from the shearbox tests, the avarage c_u/p' ratio is found to be 0.32, which is very close to the findings from Bjerrum and Skempton. This ratio can be used to determine the undrained shear strength, due to overburden pressure, at any depth, if the height of the water table is known.

Table 5.1 shows a calculation of the undrained shear strength, which can be expected in the layers of the tailings-dam, which are below the "upper drying crust". As noted in Chapter 2, Bjerrum (1954), showed that the linear increase in undrained shear strength does not hold good for the upper drying crust. This layer is discussed in Section 5.3.

The first part of Table 5.1 shows the undrained shear strength (last columns), just after deposition, assuming the water table is at the surface. The second part of Table 5.1 shows the same information after a few days when the water table has decreased to 2 meters below the surface. The last part shows the undrained shear strength values, assuming that the water table has dropped to 5 meters below the surface.

A number of simplifying assumptions have been made to illustrate the effect of the undrained shear strength/effective overburden pressure ratio. These assumptions are as follows:

- The moisture content is a function of the normal stress (as shown by Equation 4.2). This will not always be true due to layering and differences in permeability, etc.
- There is a direct relationship between the moisture content and the density of the material. This will always be true.
- The depth of the water table and the moisture contents were assumed values for illustrative purposes.

The following method and equations were used to determine the values in Table 5.1:

- For a particular depth (first column), a moisture content was randomly chosen (third column).
- The saturated density (fourth column) was calculated with Equation 5.1 (as shown below).

- The saturated density minus the density of water gave the effective density (fifth column).
- The effective overburden pressure (sixth column) was calculated at the specific height with the level of the water table as indicated in the second column.
- Equation 4.2 was used to determine if the chosen moisture content (seventh column) was correct, if not, an iterative process was followed until the chosen moisture content was the same as the calculated moisture content.
- The undrained shear strength (eighth column) was determined with the ratio ($c_u/p'=0.32$), as discussed previously.

$$\gamma sat = Gs\gamma w(1+w)/(1+wGs) \quad (5.1)$$

with

γsat = saturated density (kN/m³)

Gs = Specific weight (2.7)

γw = density of water (9.81kN/m³)

w = moisture content (%)

Table 5.1: Illustration of change in undrained shear strength with depth, for different stages of desiccation

Directly after deposition							
Depth (m)	Depth of water (m)	Chosen moisture content (%)	Saturated density (kN/m³)	Effective density (kN/m³)	Effective overburden pressure (kPa)	Moisture content (%)	Undrained Shear Strength (kPa))
5	0	81	15.0	5.2	26.2	81	8.4
10	0	71	15.5	5.7	57.2	71	18.3
20	0	62	16.0	6.2	124.7	62	39.9
30	0	57	16.4	6.6	197.1	57	63.1
40	0	52	16.7	6.9	277.5	52	88.8

Few days after deposition							
Depth (m)	Depth of water (m)	Chosen moisture content (%)	Saturated density (kN/m³)	Effective density (kN/m³)	Effective overburden pressure (kPa)	Moisture content (%)	Undrained Shear Strength (kPa))
5	2	74	15.4	5.6	47.4	74	15.2
10	2	68	15.7	5.9	78.4	68	25.1
20	2	60	16.2	6.4	146.9	60	47.0
30	2	55	16.5	6.7	221.0	55	70.7
40	2	51	16.8	7.0	300.3	52	96.1

Many days after deposition							
Depth (m)	Depth of water (m)	Chosen moisture content (%)	Saturated density (kN/m³)	Effective density (kN/m³)	Effective overburden pressure (kPa)	Moisture content (%)	Undrained Shear Strength (kPa))
5	5	68	15.7	5.9	78.5	68	25.1
10	5	64	15.9	6.1	110.2	64	35.3
20	5	58	16.3	6.5	179.0	58	57.3
30	5	54	16.6	6.8	252.6	54	80.8
40	5	50	16.9	7.1	332.9	50	106.5

As shown in Table 5.1, the undrained shear strength in the consolidated layer increases with depth and with time, as the water table descends. This increase is mainly due to consolidation effects caused by the self weight of the material.

5.3 Undrained shear strength in the upper crust of the dam

The "upper drying crust" is the upper layer in the tailings dam which is subjected to continuous drying and re-wetting cycles. To determine the exact strength in every layer in this crust would require modeling and is beyond the scope of this project.

It is assumed for this project, that this crust will probably be ±5m thick at the end of construction, on a 40m high tailings dam. This value was obtained from the results published by Bjerrum (1954), but should be measured on the actual tailings dam. This depth is dependent on the present and the previous level of the water table and therefore the overconsolidation ratio (Parry, 1970). The following assumptions were made in determining the strength of the whole layer.

- The strength at the bottom of the drying layer is the same as the strength calculated for the top of the consolidated layer in the previous section, as shown in Table 5.1.
- Just before deposition, the existing layers would be dried out to a certain moisture content and shear strength, depending on the thickness of the layer, deposition cycle time and the weather conditions. At this stage the upper crust would consist of blocks of material with cracks running down to the consolidated layer.
- The actual depth of the cracks is dependent on the undrained shear strength of the material, the bulk density value, the permeability and the capillary stresses, and could be determined with accurate modeling.
- When the new layer of material is deposited onto the dam, the material flows into these cracks, fills them up and then forms a new surface layer,

which is at the deposition moisture content. The deposition moisture content ranges from 200% to 150%. For the purpose of stability calculations, the initial moisture content will be taken at 150%, which relates to a undrained shear strength of 0.2kPa. A value of 150% was chosen to compromise for the expected run-off water that will be decanted from the pond at the penstock.

- When stability is considered in Chapter 6, it will be assumed that the failure circle will go through the newly deposited material in the vertical crack and not through the existing material. Therefore the whole upper layer's strength is the same as the fresh tailings.
- Equation 4.1 was used to calculate the shear strength of the material in the crack, at different moisture contents during the drying process.
- These assumptions are used to determine the values in Table 6.2, which was used for the stability calculations.

5.4 Strength required for rehabilitation purposes

Table 5.2 shows a calculation to determine the minimum required strength for the tailings to enable a track mounted vehicle to enter onto the dam for rehabilitation purposes. It is shown that the minimum required strength is 16kPa, which is achieved at 78% moisture content.

Table 5.2: Required undrained shear strength for rehabilitation purposes

Assumptions:		
Tractor with mass of	8000	kg
Track dimensions:	0.635	m wide
	2.065	m long
Maximum track pressure, q_u =	30	kPa

Calculation:		
Ultimate bearing pressure, q_u=		cuNc
Assuming	ϕ =	0 °
For cohesive soil:	Nc =	6.1
And with	FOS =	3
Allowable bearing pressure, qall=	q_u/FOS	
Minimum required shear stress	c_u =	15.2 kPa

5.5 Time required for desiccation

The exact time required for desiccation is difficult to determine and depends on a number of factors such as:

- The weather conditions, which include the evaporation, rainfall, wind etc. (as shown in Chapter 4)
- The exact geotechnical properties of the deposited tailings. These include permeability, grading and coefficient of consolidation.
- Depth of layer and quantity and size of cracks formed during desiccation.

In Chapter 4 it was shown that under laboratory conditions, the normal drying rate was estimated at 3% moisture content reduction per day. When the material was subjected to constant windy conditions, the drying rate went up to 13% per day. An average value of 5% per day will be used in this report.

Therefore the required time to reach 15kPa as determined in the previous section, from a deposition moisture content of 150%, will be ±15 days.

5.6 Concluding remarks

In conclusion on the interpretation of the results, there are two layers identified in the tailings dam, namely the "upper drying crust" and the consolidated layer beneath it.

The required strength for rehabilitation purposes was found to be ±15kPa, and it will require ±15 days to achieve this strength. These values are estimated values and should be re-calculated for different conditions. It does not, for instance, include the vibration effects that a dozer will have on the material, which can lead to liquefaction.

6 STABILITY ANALYSIS

6.1 Introduction

This chapter illustrates how the findings from the previous chapters can be used to calculate the slope stability of the tailings deposit. A few assumptions were made to simplify the problem and to illustrate the use of the findings more clearly. The geometry, material properties and other assumptions are discussed in the next section.

6.2 Geometry and material properties

The dimensions and material properties of the starter wall and base material, were taken from the proposed design for the tailings deposit at Richards Bay (Williamson, 1998). The geometry used for the slope stability calculations is shown in Figure 6.1.

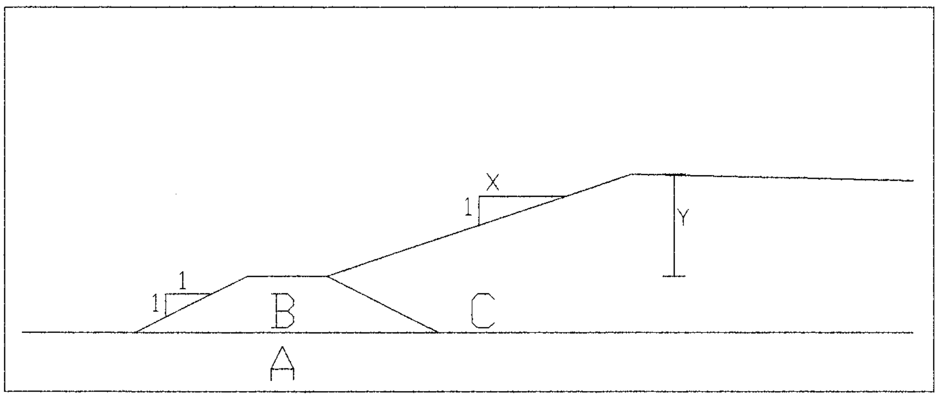

Figure 6.1: Geometry used for slope stability calculations

The section consists of a base material (A), a starter wall (B) and tailings material (C). The starter wall is 5m high, 10m wide at the top and has a slope of 1:1. The tailings material has an initial built-up slope of 1:X, which will be varied in the analyses. The rest of the material has a deposition slope of 1:80.

The height Y, is the height from the top of the starter wall to the point where the slope changes from the initial slope to the flatter slope of 1:80.

The properties of the materials, A, B and C, as indicated in Figure 6.1, are tabulated in Table 6.1 and were obtained from the design report by Williamson (1998).

Table 6.1: Material properties

Material	**Saturated Unit Weight (kN/m^3)**	**Cohesion – c (kPa)**	**Internal angle of Friction - ϕ (°)**
A – Peat	12.5	10	20
B – Starter wall	18.8	5	35
C – Tailings	*	*	0**
* See Table 6.2 ** Taken as zero to illustrate the effect of changes in the other properties as shown in Table 6.3			

The undrained shear strength in the "upper crust" is almost at its lowest just after deposition, when the cracks are filled with tailings, which have no shear strength. After a few days, the freshly deposited tailings still have little strength and the water from the deposited tailings has had a chance to penetrate the existing tailings and soften the material. Therefore the strength after a few days is assumed to be the worst case and will be used in the stability analyses.

In Table 5.1 the undrained shear strength for the consolidated layer was determined for different depths, with different water levels. For the general case it was assumed that the water table would have reduced to 2m below the surface. From Table 5.1, the undrained shear strength values are repeated in Table 6.2. Table 6.2 also show the moisture content (calculated with Equation 4.1) and the saturated density (calculated with Equation 5.1).

The moisture content for the top part of the upper crust (depth = 1m), was assumed and from this value, the undrained shear strength was determined with Equation 4.1. The saturated density was also calculated with Equation 5.1 as for the consolidated layers.

For the stability analyses both the upper crust and the consolidated layer were divided into 2,5m thick layers. The average saturated density and undrained shear strength were calculated for each layer. Table 6.2 shows the data that was used in the stability analysis of the general case. The data is also shown in Figure 6.5.

Table 6.2: Properties of tailings (Material C)

Consolidated layer			
Depth (m)	**Undrained shear strength (kPa)**	**Moisture content (%) (From Equation 4.2)**	**Saturated density (kN/m^3) (From Equation 5.1)**
7	19.1	71	15.5
9	23.1	69	15.6
15	35.9	63	16
20	47	60	16.2

Upper crust			
Depth (m)	**Undrained shear strength (kPa)**	**Moisture content (%) (From Equation 4.1)**	**Saturated density (kN/m^3) (From Equation 5.1)**
1	4.4	100	14.3
3	8.1	89	14.7
5	15.2	79	15.1

It was assumed that the material properties in a specific layer would be constant throughout the layer in a vertical and horizontal direction. In practice the material will form layers with different properties, depending on the deposition-method and properties of the deposited material.

It was further assumed that the water table would be at the surface as soon as the tailings are deposited and it descends with time. In truth the water from the freshly deposited tailings will penetrate the existing tailings at a rate governed by the current permeability of the drier tailings. Therefore the water will be moving downwards through the tailings, towards the existing water table, which is at a certain depth within the existing dam. This depth can be measured on site with piezometers.

6.3 Slope stability analysis

The slope stability analyses were done with the PCStabl software package (Lovell, C.W, 1988). The Bishop circular analysis method was applied to calculate the factor of safety (FOS) for different geometries and material properties. The factor of safety was calculated for illustrative purposes only, to indicate the influence of the parameters on the safety of the dam. The actual factors of safety will have to be calculated by taking into account the influence of the change in permeability and angle of internal friction, but this falls outside the scope of this project report.

Three parameters were identified to compare different scenarios:

- Height above starter wall (Y)
- Initial slope (X)
- Depth of water table from surface

Two of the parameters were held constant, while the other parameter was varied. A graph was then drawn for the change in FOS with change in that particular parameter. Table 6.3 shows the three scenarios that were analysed.

Table 6.3: Different scenarios

Scenario	Height (Y)	Initial slope (X)	Depth of water table (m)	Refer
I	Varied	3	2m	Section 6.3.1
II	15m	Varied	2m	Section 6.3.2
III	15m	3	Varied	Section 6.3.3

A water table depth of 2m was used because this was assumed to be the worst case, as explained in Section 6.2. The initial slope of 3 was chosen due to geometric constraints in the proposed design. The height above the starter wall of 15m was identified in the proposed design as the average height of the dam.

6.3.1 Scenario I: Constant slope and water depth

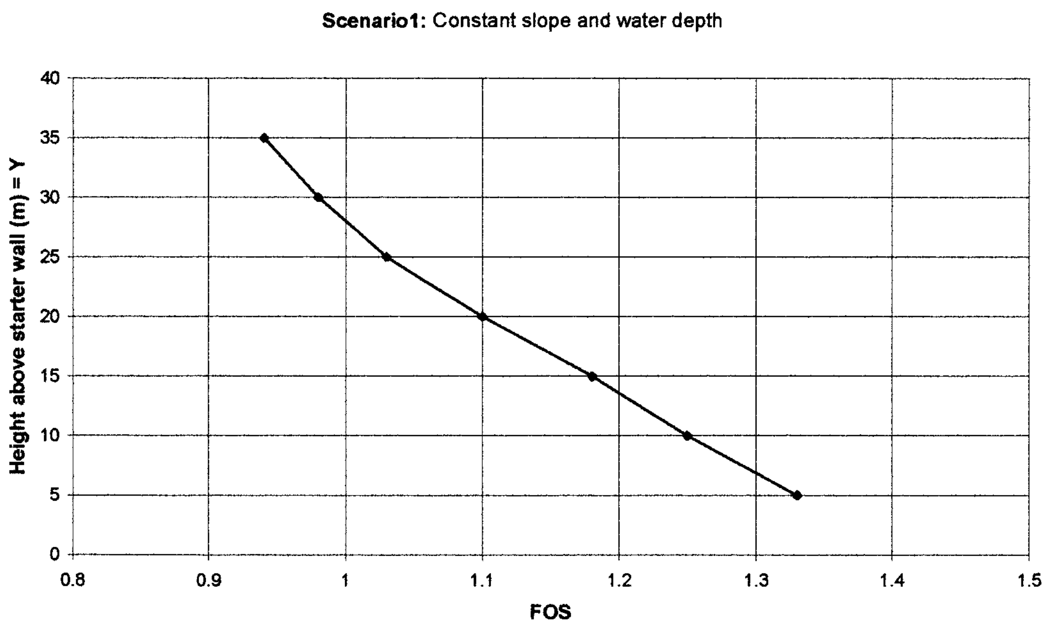

Figure 6.2: Change in FOS with change in height

A water table depth of 2m and a constant initial slope of 1:3 were chosen for this scenario. Figure 6.2 illustrates that the change in FOS with change in height above the starter wall is almost to linear.

6.3.2 Scenario II: Constant height and water depth

A height of 15m above the starter wall was used, which gives a total height of 20m. The depth of the water table was kept at 2m below the tailings surface and the slope was varied to determine the influence on the FOS. Figure 6.3 illustrates the how the FOS changes with change in initial slope of the tailings material.

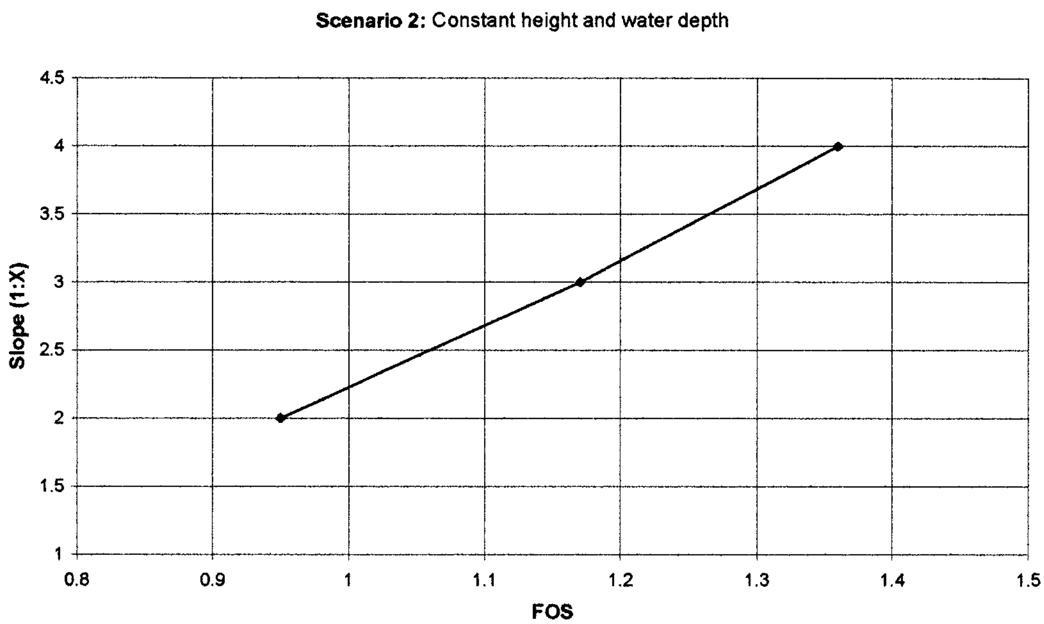

Figure 6.3: Change of FOS with change in initial slope

An initial slope of 1:3 gives a FOS of 1.18 for a constant water depth of 2m and total height of 20m. Figure 6.3 shows that the FOS reduces linearly with an increase in slope angle.

6.3.3 Scenario III: Constant height and initial slope

A constant slope of 1:3 and a constant total height of 20m were chosen for this scenario. Figure 6.4 illustrates how the FOS changes with change in water depth. Table 5.1 was used to determine the change in density of the material, with change in water depth.

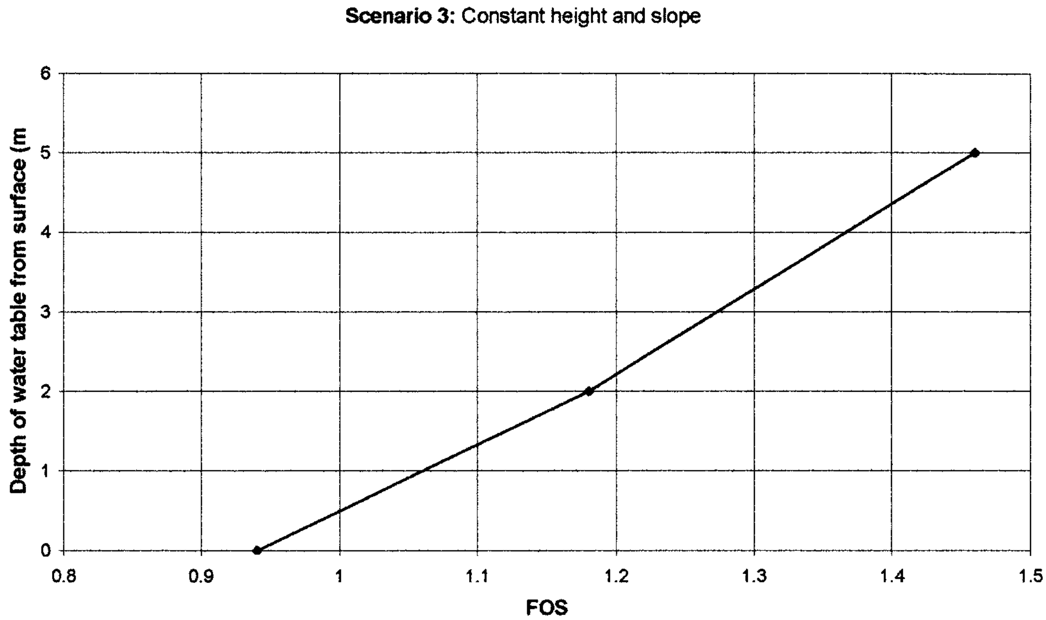

Figure 6.4: Change in FOS with change in water depth

Figure 6.4 gives a FOS of 1.18 for tailings with a water depth of 2m, height of 20m and initial slope of 1:3.

Figure 6.5 shows the slope section, with the 10 most critical failure surfaces, for a water depth of 2m, initial slope of 1:3 and height of 20m. The material properties are as shown in Table 6.1 and 6.2. The input parameters are shown in Appendix F.

The failure circles were forced to pass through the toe of the tailings, just above the starter wall. This was done to determine the FOS against failure through the tailings and not through the starter wall and the base material. It was actually found that the most critical failure surface is through the starter wall, but it could be remedied by just changing the starter wall material or by compacting the wall during construction.

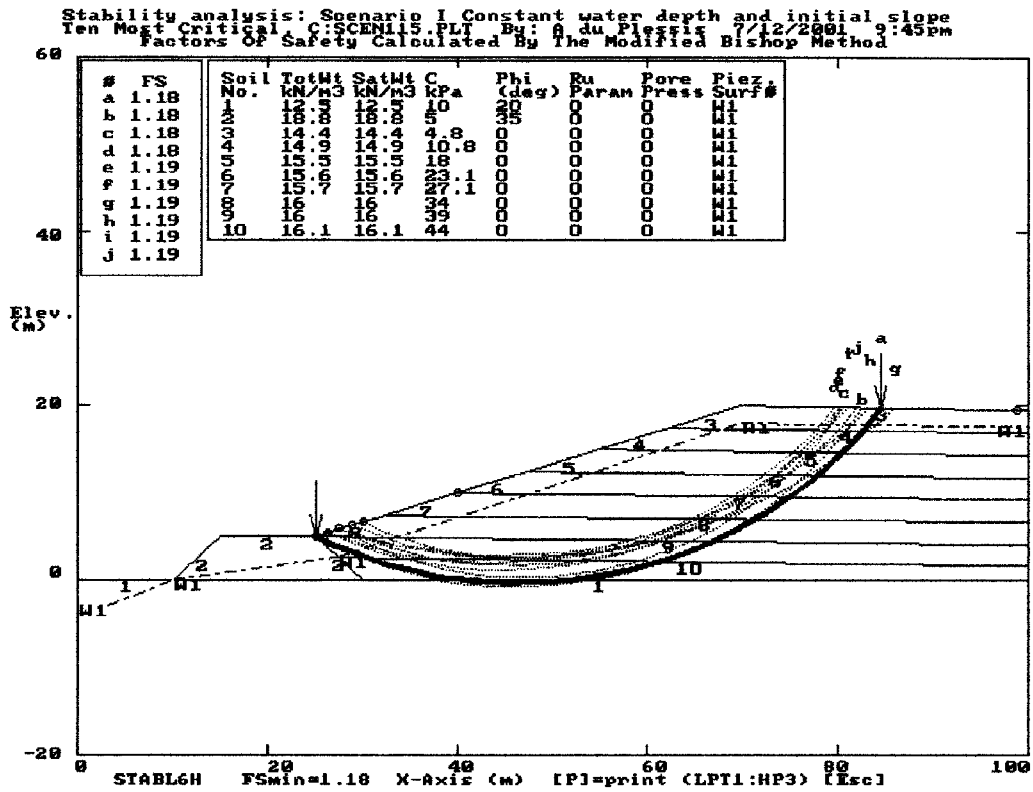

Figure 6.5: Slope Stability Analysis

6.4 Concluding remarks

The geometry and the material properties of the starter wall and the base material were chosen to simulate conditions in Richards Bay, but these parameters are variable and should be adjusted according to the situation in which they may be applied.

The purpose of this chapter was to illustrate the use of the moisture content – undrained shear strength relationship defined in Equation 4.1, in a stability analysis.

It must be noted that the assumption of layers with increasing strength simplifies the analyses. In practice there will be different moisture contents throughout the layers. The surface will have lower moisture content than the lower levels, and there will be layers within the deposit that may have higher or lower moisture content than their surrounding layers. The moisture content of a particular layer depends on the length of time the layer was allowed to desiccate. With this said, it can also be stated that the difference in moisture content between adjacent layers should not be large, and with time may be in equilibrium.

It is therefore possible to use the relationship between the moisture content and the undrained shear strength, as determined with the shear vane, to determine soil properties that can be used for slope stability calculations. The layering and exact moisture content with depth has to be refined for each particular geometry and type of tailings.

7 CONCLUSION

7.1 Introduction

This chapter concludes this project report. A summary of all the conclusions made during the investigation will be given, as well as some recommendations on further possible investigations.

7.2 Conclusions made from this project report

- The purpose of this project report was to determine an alternative method of measuring the stability of a Red Berea Sand tailings deposit. It was determined that there is a relationship between moisture content, which can easily be measured, and the undrained shear strength. This relationship can be used to determine the stability of the deposit.
- Chapter 2 discusses factors which influence the stability of tailings. Factors such as suction, capillary stresses, degree of consolidation or overconsolidation, mineralogy, geotechnical properties and deposition method etc. determine the stability and behavior of a tailings deposit.
- What was concluded from this chapter is that tailings dry out due to evaporation of fluid. When tailings dry out, pore pressures reduce and even become negative (suction). When pore pressures reduce, the particles move closer to each other and the density increases and the material gains strength. When the tailings are wetted, it loses strength. The magnitude of strength gain or loss, depends on the material properties and surrounding conditions.
- Chapter 3 describes shear strength and how to determine it. Shear strength is used to describe the state of failure for a material. Undrained shear strength can be determined and interpreted with different methods, namely a shear box test or a shear vane. The shear vane and the shear box tests give good comparable values for the undrained shear strength.

- In Chapter 4 all the laboratory work done for this project report are described. A number of geotechnical tests, including shear vane , shear box and grading analysis were done. From the shear vane results, a relationship between the moisture content and the undrained shear strength (Equation 4.1), was determined. This equation is only applicable for undrained and consolidated conditions.
- The influence of different weather conditions (wind as well as rain) were illustrated and it was found that they have a noticeable influence on the behavior.
- It was found that the results of the shear box tests are very similar to the shear vane results and it can be assumed that the shear vane results are reliable.
- The shear vane apparatus is a easy-to-use method of determining the undrained shear strength, but as with many other geotechnical tests, the results are user dependant. If the shear vane is used correctly and consistently, it should give accurate values for undrained shear strength.
- It is possible to use the relationship between the moisture content and the undrained shear strength (determined with the shear vane), to determine the slope stability.
- The layering, which causes anisotropy (different properties in different directions), and exact moisture content with depth have to be determined for each particular geometry and type of tailings.
- The tailings deposit is divided into two layers, namely the "upper drying crust", and the consolidated layer. The strength values are determined differently for the two layers.
- The moisture content values for the upper crust are assumed, but they can easily be measured in the field. The undrained shear strength can be calculated using Equation 4.1.
- The consolidated layer's strength is determined from the relationship between the undrained shear strength and the effective overburden pressure. This relationship is mentioned in Chapter 2 and discussed in more detail in

Chapter 5. In Chapter 5 it is shown that the c_u/p' ratio for Red Berea sand tailings is ±0.32.

- In Chapter 5, it is also illustrated how the results could be applied to calculate the time required, before rehabilitation can be started.
- Required bearing capacity can be calculated with Terzaghi's equation (see Table 5.2). Equation 4.1 can be used to calculate the required moisture content. By monitoring the deposition moisture content and the magnitude of evaporation, the time to reach the required moisture content can be calculated.
- In Chapter 6 the geometry used for the stability analyses is described (Figure 6.2). The material properties used and the assumptions, are also stated.
- Three variables (height, initial slope and depth of water table), were identified and each is expressed as a function of the factor of safety (FOS), while the other two are kept constant.
- Figure 6.2 shows that for an assumed water table depth of 2m and a constant slope of 1:3, there is a linear decrease in FOS when the height of the dam is increased.
- It is found that for a change in initial slope and change in water table depth, the change in FOS is also linear.
- In conclusion to Chapter 6, it is stated that the input parameters must be adjusted for the conditions of the situation being analysed.

7.3 Recommendations

- The effect of layering causes different geotechnical properties (for instance permeability and grading) in different directions (anisotropy). Therefore the migration of water is influenced and the magnitude of change in moisture content is also altered. This will cause strength differences in successive layers of tailings, which will influence the slope stability. It is therefore recommended to determine the permeability for different moisture contents.

- It is further recommended to determine the magnitude of moisture intake for different initial moisture contents. This, together with the above mentioned effect of layering, can be used to determine the strength decrease in an existing layer during and after depositioning.
- The rate of desiccation is a very important aspect for construction of tailings deposits. It will influence the time required between depositions. The rate of dessication is determined by a number of factors such as material properties, evaporation, climate etc. It is recommended to obtain a relationship between the evaporation and the change in moisture content. This will be very dependent on the type of tailings and the layer thickness. If this can be achieved, the moisture content can be predicted or calculated by measuring the evaporation. The strength and stability can then be determined, by using the methods illustrated in this project report.
- When measuring the evaporation, it must be kept in mind that the evaporation from the tailings surface and normal pan evaporation will probably be different. Therefore it is recommended to do on site evaporation measurements together with moisture content measurements on the actual tailings deposit. This will give a relationship, which could be used to predict strength values during the construction period.

7.4 Concluding remarks

This project report gives the designer a method of determining when the tailings have gained enough strength to allow another deposition. Although there is still a large quantity of information required to fully understand and specify an exact method of construction, this project report gives an acceptable indication of what behavior can be expected.

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APPENDIX A

STATUTORY REQUIREMENTS

STATUTORY REQUIREMENTS

LEGISLATION APPLICABLE TO MINE RESIDUE DISPOSAL

ACT	ASPECTS COVERED
Minerals Act (Act 50 of 1991) (replaces Mines & Works Act – Act 27 of 1956)	Regulations concerning protection of public safety, environmental protection, rehabilitation plans, removal of topsoil, stream and river diversions, waste disposal, revegetation, storm water and effluent control.
Water Act (Act 54 of 1956) as amended from time to time and including Act 96 of 1984	Treatment and disposal of effluents, water use and entitlements, surface water management, water pollution control, water care works, disposal of underground water, effluent water quality. Dam safety: R1560 Tailings dams must be registered (R1560 Reg. 15) Tailings dams excempted from other Dam Safety Regulations until 24/7/1995 (R1299)
Water Act (Act 36 of 1998)	Dam safety: Chapter 12
Environment Conservation Act (Act 73 of 1989) (as amended from time to time)	Protection of the environment against disturbance, deterioration, defacement, poisoning or destruction, control and management of disposal sites.
Conservation of Agricultural Resources Act (Act 43 of 1983)	Maintenance of production potential of land, combatting and prevention of erosion and weakening or destruction of water resources, protection of vegetation, combating of weeds and invader plants.
Physical Planning Act (Act 88 of 1967) (as amended from time to time)	Zoning and use of land for industrial purposes, quarry development and rehabilitation.
Health Act (Act 63 of 1977)	Air pollution control, water pollution control
Atmospheric Pollution Prevention Act (Act 45 of 1965)	Dust control
Nuclear energy Act (Act 131 of 1993)	Control over radio active waste
Hazardous Sunstances Act (Act 15 of 1973)	Control of transport, dumping, storage or disposal of any hazardous waste, including radioactive material.
Mine Health and Safety Act (Act 29 of 1996)	Regulates health and safety on Mines
Draft Code of Practice for Mine Residue Deposits (SABS 0286: 1998)	Management of mine reidue deposits.

APPENDIX B

SHEAR VANE DIMENSIONS AND TESTING PROCEDURE

APPENDIX B: SECTION 1: Vane dimensions

Table B.1 shows the codes used for the different vane sizes with the dimensions in millimeters as well as the correction factors.

Table B.1: Vane sizes used for laboratory work (Chapter 5)

Code	Height (mm)	Diameter (mm)	Correction factor
S	32	16	2
M	40	20	1
L	50.8	25.4	0.5
XL	102	52	0.06
XXL	175	88.2	0.01

In the United Kingdom the dimensions of the field vanes are controlled by BD1377: 1990: part 9 clause 4.4.2, (see Table B.2). The height must be twice the diameter. The standard states that experience has shown that the following overall dimensions are suitable:

Table B.2: Vane dimensions from BS1377: 1990

Undrained shear strength (kPa)	Vane diameter (mm)	Vane height (mm)	Rod diameter (mm)
< 50	75	150	< 13
50 – 75	50	100	< 13

APPENDIX B: Section 2: Testing procedure and standards

Test procedure

The test procedure is described as follows:

- Push the vane into undisturbed soil, slowly and with a single thrust. Ensure that the vane is not rotated during this stage.
- Rotate the torque wrench at a slow but continuous rate. BS 1377:1990 specifies a rate of 6 – 12° per minute, whilst ASTM D2573 specifies that the rate shall not exceed 6° per minute.
- Measure the maximum torque required to shear the material. After the material has been sheared totally, reset the torque measuring device and restart shearing to determine the remoulded strength of the soil.

Standards

The following standards are known to exist for the vane test:

• USA	ASTM D2573-72 (Reapproved 1978)
• United Kingdom	BS 1377
• Australia	AS F2.2 1977
• Germany	DIN 4096 1980
• India	IS 4434 1978

APPENDIX C

EXPERIMENTAL RESULTS

Section A: Moisture content versus Undrained shear strength at low overburden pressure

Moisture content vs Shear Strength at different depths

Depth (mm)	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comment
250	28	20.5	28.4	23.2	2	1	L	193	MC influenced by water at top
250	31	20.5	32.7	24.6	2	1	L	198	MC influenced by water at top
200	12	20.5	30	23.7	2	1	L	197	MC influenced by water at top
150	32	20.5	25.8	22.3	2	1	L	194	MC influenced by water at top
100	10	20.7	25.5	22.3	2	1	L	200	MC influenced by water at top
Decant water totally									
150	33	20.4	43.9	28.9	2	1	L	176	MC influenced by water at top
200	4	20.8	41.8	28.4	2	1	L	176	MC influenced by water at top
250	7	20.7	41.1	28.2	2	1	L	172	MC influenced by water at top
-	27	20.8	81.4	44.4	-	-	-	**157**	MC of whole sample
190	11	20.5	65.8	36.5	84	0.8	XXL	183	-
175	6	20.6	77.5	41	18	1.1	XL	179	-
175	28	20.5	50.2	31.2	18	1.1	XL	178	-
175	12	20.5	55.9	34	20	1.2	XL	162	-
140	22	20.4	60.3	36.3	19	1.1	XL	151	-
140	31	20.4	51.4	32.8	26	1.6	XL	150	-
175	18	20.4	46.5	30.5	18	1.1	XL	158	-

Section B:

Moisture content vs Shear Strength Experimental Data

Sample 2

Date	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comments
06/05/1999	6	20.5	74.2	37.8	-	-	-	210	Prepare sample
04/06/1999	19	20.7	47.2	33.4	6	3	L	109	-
07/06/1999	28	20.5	25.7	23.5	Unable to penetrate		M	73	Large drop in MC due to fan
17/06/1999	34	20.9	46.6	40.6	Unable to penetrate		S	30	Large drop in MC due to fan

Sample 5

Date	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comments
30/04/1999	-	-	-	-	-	-	-	173	Prepare sample
04/05/1999	13	20.6	35.9	26.3	0	0	L	168	Layer thickness = 87mm
04/05/1999	3	20.6	34.7	25.7	0	0	L	176	Layer thickness = 87mm
06/05/1999	18	20.4	68.5	39.1	-	-	-	157	-
14/05/1999	27	20.9	48.2	35.6	10	5.0	L	86	-

Sample 9

Date	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comments
26/03/1999	-	-	-	-	-	-	-	-	Prepare sample
04/05/1999	27	20.8	35.9	30.5	40	40	M	56	Layer thickness = 65mm
04/05/1999	18	20.4	38.1	31.8	38	38	M	55	Layer thickness = 65mm
04/05/1999	9	20.6	36.4	30.1	22	22	M	66	Layer thickness = 65mm
04/05/1999	2	20.5	48.4	39.2	>46	>46	M	49	Sample broke

Sample 10

Date	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comments
26/03/1999	-	-	-	-	-	-	-	-	Prepare sample
04/05/1999	11	20.4	39.8	30	3.5	3.5	M	102	Layer thickness = 45mm
04/05/1999	4	20.7	32.6	26.6	4	4	M	102	Layer thickness = 45mm
04/05/1999	24	20.6	35.7	28.1	4	4	M	101	Layer thickness = 45mm
04/05/1999	21	20.7	32.4	26.5	3.5	3.5	M	102	Layer thickness = 45mm
04/05/1999	1	20.8	63.8	42	-	-	-	103	Larger sample taken 15mm from top
04/05/1999	7	20.8	79	49.5	-	-	-	103	Larger sample taken 15mm from bottom

Sample 14

Date	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comments
06/05/1999	9	20.6	65.5	36.2	-	-	-	188	Prepare sample
14/05/1999	7	20.8	43.7	30.3	2	1.0	L	141	-
04/06/1999	11	20.5	47.9	34	8	4	L	103	-
07/06/1999	8	21	36.4	30.6	34	34	M	60	Large drop in MC due to fan
17/06/1999	21	20.7	53.9	41.4	72	36	L	60	-

Sample 15

Date	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comments
10/05/1999	32	20.5	39.7	27	2	0.1	XL	195	Prepare sample
10/06/1999	22	20.5	40.3	27.5	0	0	L	183	Sample was sealed

Sample W1

Date	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comments
12/05/1999	13	20.6	54.9	31.8	2	0.1	XL	206	-
12/05/1999	31	20.4	74.6	38.4	10	0.1	XXL	201	-
14/05/1999	2	20.6	79	40.6	27	0.3	XXL	192	-
04/06/1999	22	20.5	36.6	27.1	1.5	0.8	L	144	-
17/06/1999	1	20.8	87.2	47.6	18	1.08	XL	148	-

Sample W2

Date	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comments
12/05/1999	22	20.5	60.8	33.7	9	0.1	XXL	205	-
30/06/1999	4	20.8	65.5	41.9	15	0.9	XL	112	-

Sample W3

Date	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comments
12/05/1999	20	20.7	51.5	30.4	5	0.1	XXL	218	-

Sample W4

Date	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comments
12/05/1999	8	20.7	65.4	34.9	6	0.1	XXL	215	-

Section C:

Moisture content vs Shear Strength after rewetting the sample

Sample 1

Date	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comments
06/05/1999	11	20.5	69.9	36.9	-	-	-	201	Prepare sample
04/06/1999	32	20.5	45.8	32.5	6	3	L	111	-
07/06/1999	24	20.8	35.5	30.2	44	44	M	56	Large drop in MC due to fan
17/06/1999	Add 2.459 litres of water								
17/06/1999	28	20.4	35.6	29.7	30	30	M	63	MC measured directly after water was added

Sample 3

Date	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comments
06/05/1999	8	20.7	86.8	44.1	-	-	-	182	Prepare sample
14/05/1999	9	20.6	36.5	27.6	1	0.5	L	127	-
04/06/1999	26	20.4	32.2	26.6	6	3	L	90	-
04/06/1999	1	20.8	47.2	34.3	10	5	L	96	-
07/06/1999	13	20.6	35.9	30	30	30	M	63	-
07/06/1999	6	20.5	36.5	31.2	63	63	M	50	Bottom of sample
07/06/1999	20	21	48.3	41.7	Unable to penetrate		M	32	Top of sample
07/06/1999	Add 2.834 litres of water to the bucket								
10/06/1999	28	20.4	29.5	25.9	>24	>24	M	65	-
10/06/1999	26	20.4	34.9	30.7	Sample cracked on entry of vane			41	Strength is lower due to cracks of previous test
					Most of strength retained				

Sample 4

Date	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comments
06/05/1999	20	20.6	100.5	49.3	-	-	-	178	Prepare sample
14/05/1999	4	20.8	47.3	33.1	3	1.5	L	115	-
04/06/1999	12	20.6	33.1	28.4	25	25	M	60	-
07/06/1999	Add 2.619 litres of water to the bucket								
10/06/1999	12	20.5	30.4	26.8	>56	>56	M	57	MC taken inside sample
10/06/1999	4	20.8	32.6	28.4	>42	>42	M	55	Strength is lower due to cracks of previous test
10/06/1999	10	20.7	39.7	33.5	-	-	-	48	MC taken outside sample
					Strength is more at same MC				

Sample 6

Date	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comments
06/05/1999	22	20.4	75.2	42.1	-	-	-	166	Prepare sample
10/05/1999	1	20.8	39.7	28	0	0	L	163	-
12/05/1999	24	20.6	38.9	28.2	0	0	L	141	-
14/05/1999	18	20.4	37.1	28.4	3	1.5	L	109	-
04/06/1999	3	20.5	52.8	41.5	62	31	L	54	-
07/06/1999	10	20.7	49.9	41.9	>104	>104	M	38	Sample did not shear yet
07/06/1999	Add 1.553 litres of water to the bucket								
10/06/1999	9	20.7	32.5	28.4	>32	>16	L	53	MC taken inside sample
10/06/1999	1	20.8	57.7	45.2	>52	>26	L	51	MC taken inside sample
10/06/1999	21	20.6	55.6	44.2	-	-	-	48	MC taken outside sample
					Most of the strength was retained				

Sample 13

Date	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)	Comments
06/05/1999	2	20.5	68.2	37.5	-	-	-	181	Prepare sample
10/05/1999	10	20.7	68.5	37.8	4	0.2	XL	180	Vane touched the bottom
04/06/1999	21	20.7	51	35.5	6	3	L	105	-
07/06/1999	31	20.5	35.2	29.4	21	21	M	65	Large drop in MC due to fan
17/06/1999	Add 2.248 litres of water to the bucket								
17/06/1999	33	20.8	35.2	29.3	18	18	M	69	MC was taken just after water was added
29/06/1999	22	20.5	51.2	38.8	42	21	L	68	-
29/06/1999	10	20.8	51.2	38.6	36	18	L	71	-

Section D:

Settlement over time determined with column

Date	Day	Slurry level from top (mm)	Total Settlement (mm)	Comments	
07/06/1999	0	110	0.01	Prepared sample	
10/06/1999	3	330	220	-	
14/06/1999	7	342	232	-	
17/06/1999	10	344	234	-	
29/06/1999	22	386	276	-	

Moisture content vs Shear Strength at different depths determined with column

Date	Density	Date taken in field	Time	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Moisture content (%)
07/06/1999	31	9/12/98	13:20	33	20.6	88.4	47	157
	30	9/12/98	16:00	34	20.6	63.4	32.2	269
					Average moisture content =			213

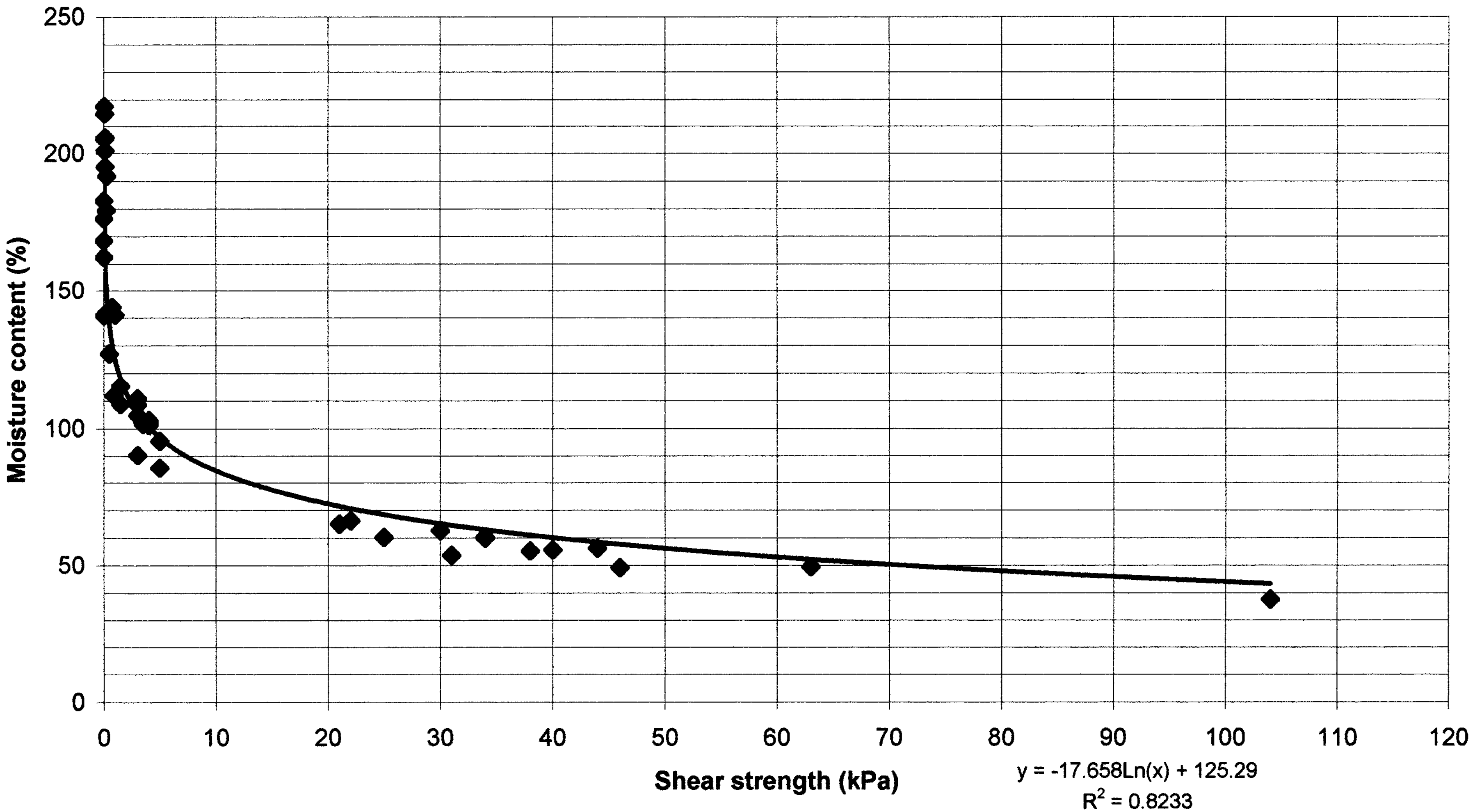
29/06/1999

Average Depth (mm)	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Vane Reading peak	Shear Strength (kPa)	Vane code	Moisture content (%)
100	26	20.4	81	45.6	16	0.96	XL	140
200	12	20.5	94.3	50.7	14	0.84	XL	144
300	8	20.9	72.6	42.4	11	0.66	XL	140
520	24	20.6	67.7	40.1	13	0.78	XL	142

Calculation of moisture content at different depths

Average Depth (mm)	MC Holder	Empty mass (g)	Wet mass (g)	Dry mass (g)	Moisture content (%)
0	14	20.5	82.7	46.1	143
114	29	21.1	85.6	47.5	144
234	25	20.6	92.4	49.1	152
354	7	20.7	80.6	45.9	138
544	19	20.7	76.7	44.5	135

Figure 4.1 - Moisture content vs. Shear strength



APPENDIX D:

SHEAR TEST RESULTS

Remolded box shear test

Project: Hillendale Sample: 1 Date: 5/8/1999

Area of sample: 0.01 m^2
Initial volume of sample: 0.000293 m^3
Final volume of sample: 0.000279 m^3

Height of sample before consolidation: 29.3 mm
Height of sample after consolidation: 27.9 mm

Mould (g): 308.366

	Sample	Trimmings		
Container number	10	107	101	
Mass of wet sample + mould/cont before test	774.731	84.373	91.32	
Mass of wet sample + container after test	629.615			
Mass of dry sample + container	441.13	61.405	63.751	
Mass of container	178.435	27.205	23.86	
Moisture content before test (%)	77.53	67.16	69.11	
Moisture content after test (%)	71.75			

Before test:
Bulk density (kg/m^3) 1592 Dry density (kg/m^3) 952
After test:
Bulk density (kg/m^3) 1619 Dry density (kg/m^3) 943

Consolidation

Applied normal stress (kPa)= 50 vertical dial gauge division = 0.002

Time	time (sqrt min)	vertical dial gauge reading (divisions)	settle-ment (mm)
0	0.0	224	0.000
5.4s	0.3	240	0.032
15.0s	0.5	490	0.532
38.4s	0.8	523	0.598
1.0m	1.0	545	0.642
2.25m	1.5	595	0.742
4.0m	2.0	630	0.812
6.25m	2.5	665	0.882
12.25m	3.5	714	0.980
16.0m	4.0	730	1.012
25.0m	5.0	759	1.070
36.0m	6.0	784	1.120
49.0m	7.0	806	1.164
1h04m	8.0	820	1.192
1h21m	9.0	833	1.218
1h40m	10.0	844	1.240
3h45m	15.8	891	1.334
6h40m	17.6	900	1.352
24h04m	36.5	943	1.438

sqrt t90 = 5 min cv = 0.848 H^2 / t90 = 2.12E-05 m^2/min = **11.14** m^2/year

This indicate a clay with a high cv or material with some fine silt

Shear box Test
Shear Data

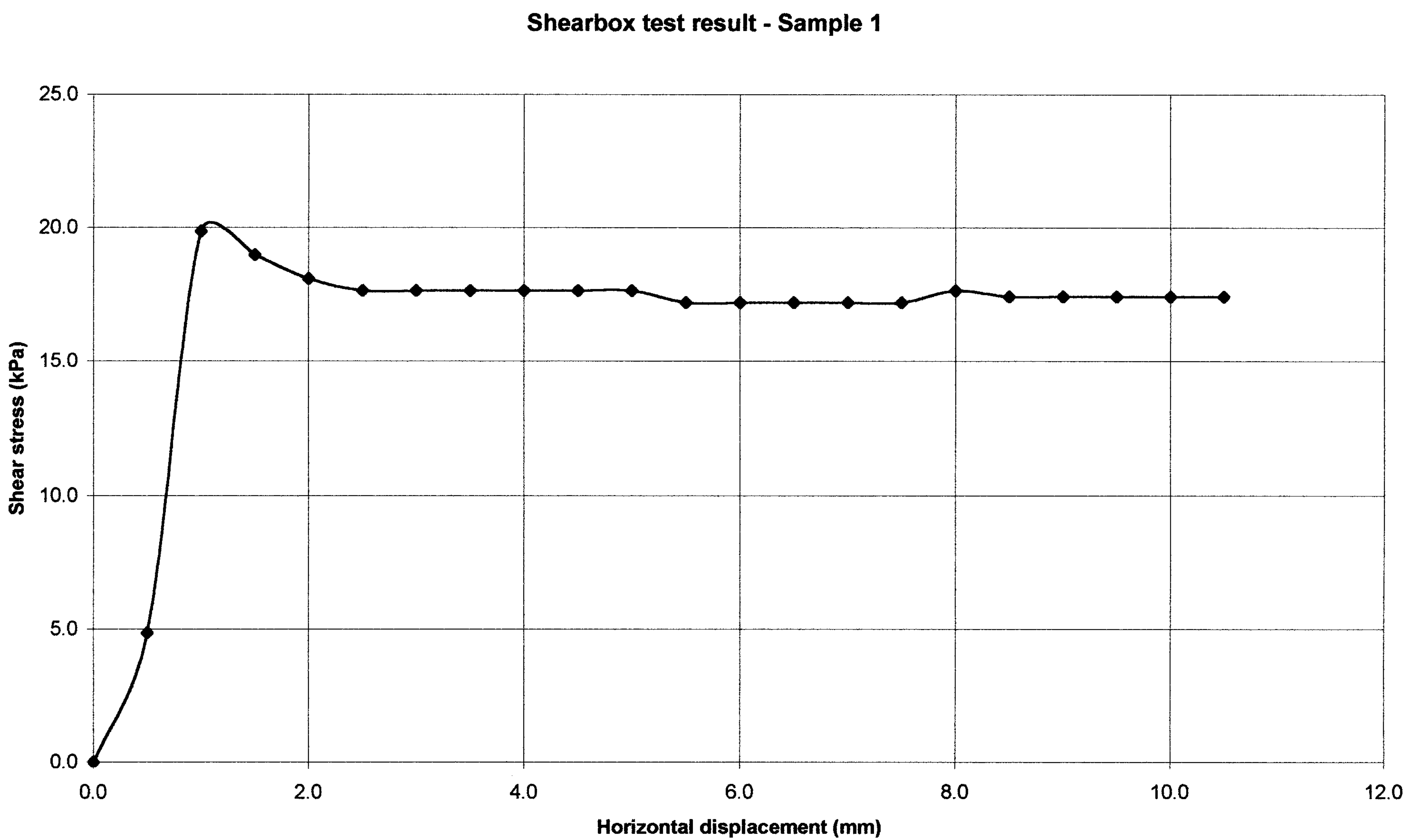
Project: Hillendale Sample: 1 Date: 6/8/1999

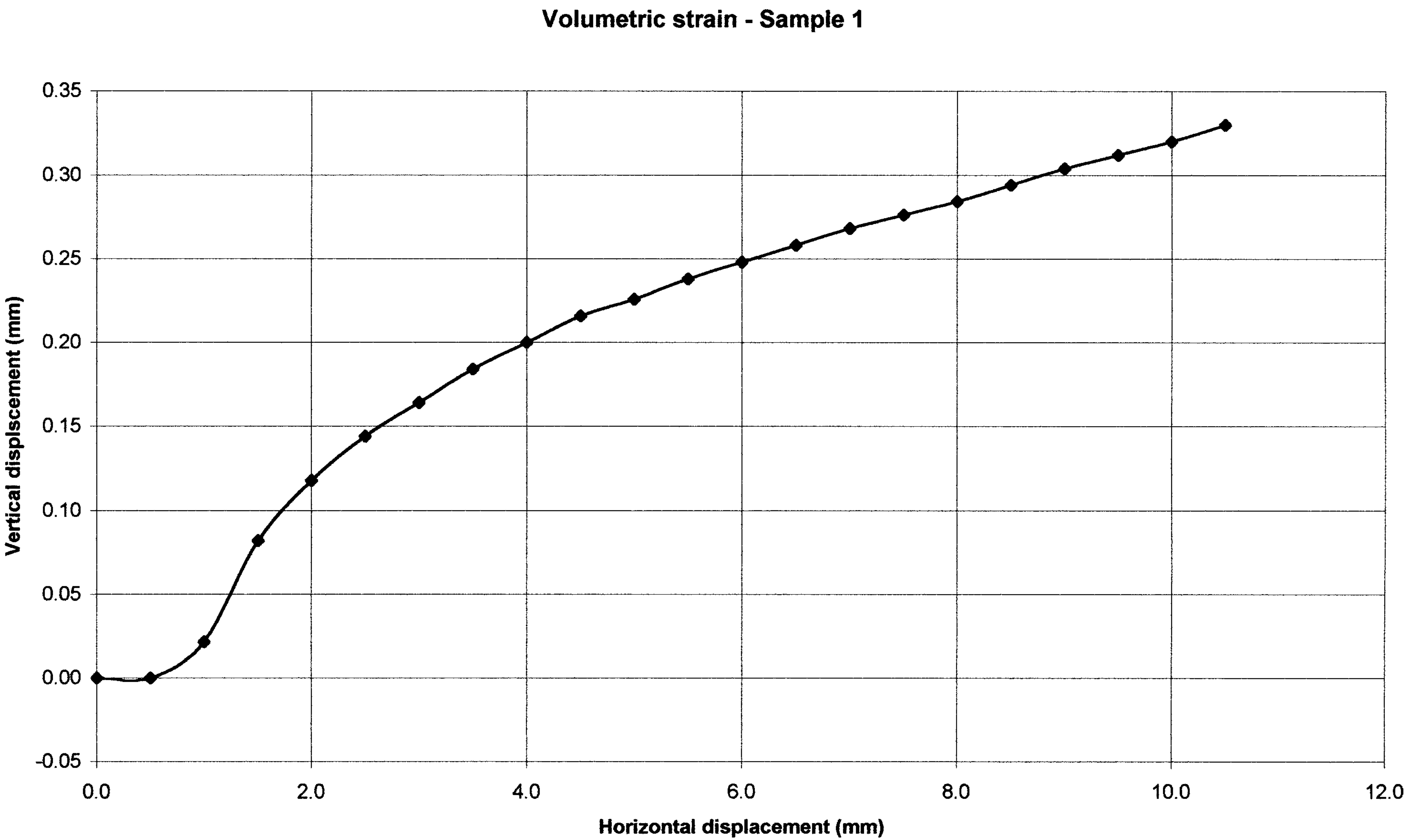
Horizontal dial gauge division: 0.01 Vertical dial gauge division: 0.002

Height of sample: 27.9 mm Applied normal stress (kPa): 50

Proof ring number: 4843 Proof ring callibration factor: 4.416N/div

Horizontal dial gauge reading (divisions)	Proof ring dial gauge reading (divisions	Vertical dial gauge reading (divisions)	Horizontal displace ment (mm)	Shear stress (kPa)	Vertical displace ment (mm)
400	0	1000	0.0	0.0	0
450	11	1000	0.5	4.9	0.000
500	45	1011	1.0	19.9	0.022
550	43	1041	1.5	19.0	0.082
600	41	1059	2.0	18.1	0.118
650	40	1072	2.5	17.7	0.144
700	40	1082	3.0	17.7	0.164
750	40	1092	3.5	17.7	0.184
800	40	1100	4.0	17.7	0.200
850	40	1108	4.5	17.7	0.216
900	40	1113	5.0	17.7	0.226
950	39	1119	5.5	17.2	0.238
1000	39	1124	6.0	17.2	0.248
1050	39	1129	6.5	17.2	0.258
1100	39	1134	7.0	17.2	0.268
1150	39	1138	7.5	17.2	0.276
1200	40	1142	8.0	17.7	0.284
1250	39.5	1147	8.5	17.4	0.294
1300	39.5	1152	9.0	17.4	0.304
1350	39.5	1156	9.5	17.4	0.312
1400	39.5	1160	10.0	17.4	0.320
1450	39.5	1165	10.5	17.4	0.330


Shearbox test result - Sample 1
Shear stress (kPa)
25.0
20.0
15.0
10.0
5.0
0.0
0.0
2.0
4.0
6.0
8.0
10.0
12.0
Horizontal displacement (mm)


Volumetric strain - Sample 1
Vertical displscement (mm)
Horizontal displacement (mm)
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
-0.05
0.0
2.0
4.0
6.0
8.0
10.0
12.0

Remolded box shear test

Project: Hillendale   Sample: 2   Date: 16/8/1999

Area of sample: 0.01 m^2 Height of sample before consolidation: 29.3 mm
Initial volume of sample: 0.000293 m^3 Height of sample after consolidation: 25.5 mm
Final volume of sample: 0.000255 m^3

Mould (g) 308.348

	Sample	Trimmings		
Container number	1	102	105	
Mass of wet sample + mould/cont before test	767.54	65.658	67.061	
Mass of wet sample + container after test	615.291			
Mass of dry sample + container	430.205	50.484	49.276	
Mass of container	173.544	27.905	25.135	
Moisture content before test (%)	78.91	67.20	73.67	
Moisture content after test (%)	72.11			

Before test:
Bulk density (kg/m^3) 1567   Dry density (kg/m^3) 937
After test:
Bulk density (kg/m^3) 1730   Dry density (kg/m^3) 1005

Consolidation

Applied normal stress (kPa)= 50   vertical dial gauge division = 0

Time	time (sqrt min)	vertical dial gauge reading (divisions)	settle-ment (mm)
0	0.0	0.00	0.000
5.4s	0.3	1300	2.600
15.0s	0.5	1330	2.660
38.4s	0.8	1365	2.730
1.0m	1.0	1385	2.770
2.25m	1.5	1455	2.910
4.0m	2.0	1490	2.980
6.25m	2.5	1525	3.050
9.0m	3.0	1551	3.102
12.25m	3.5	1573	3.146
16.0m	4.0	1597	3.194
25.0m	5.0	1633	3.266
38.0m	6.2	1668	3.336
49.0m	7.0	1690	3.380
2h57m	13.3	1785	3.570
24h04m	38.0	1881	3.762
24h42m	38.5	1883	3.766

sqrt t90 = 5.2 min   cv = 0.848 H^2 / t90 = 1.96E-05 m^2/min = **10.30** m^2/year

Shear box Test
Shear Data

Project: Hillendale Sample: 2 Date: 17/8/99

Horizontal dial gauge division: 0.01 Vertical dial gauge division: 0.002

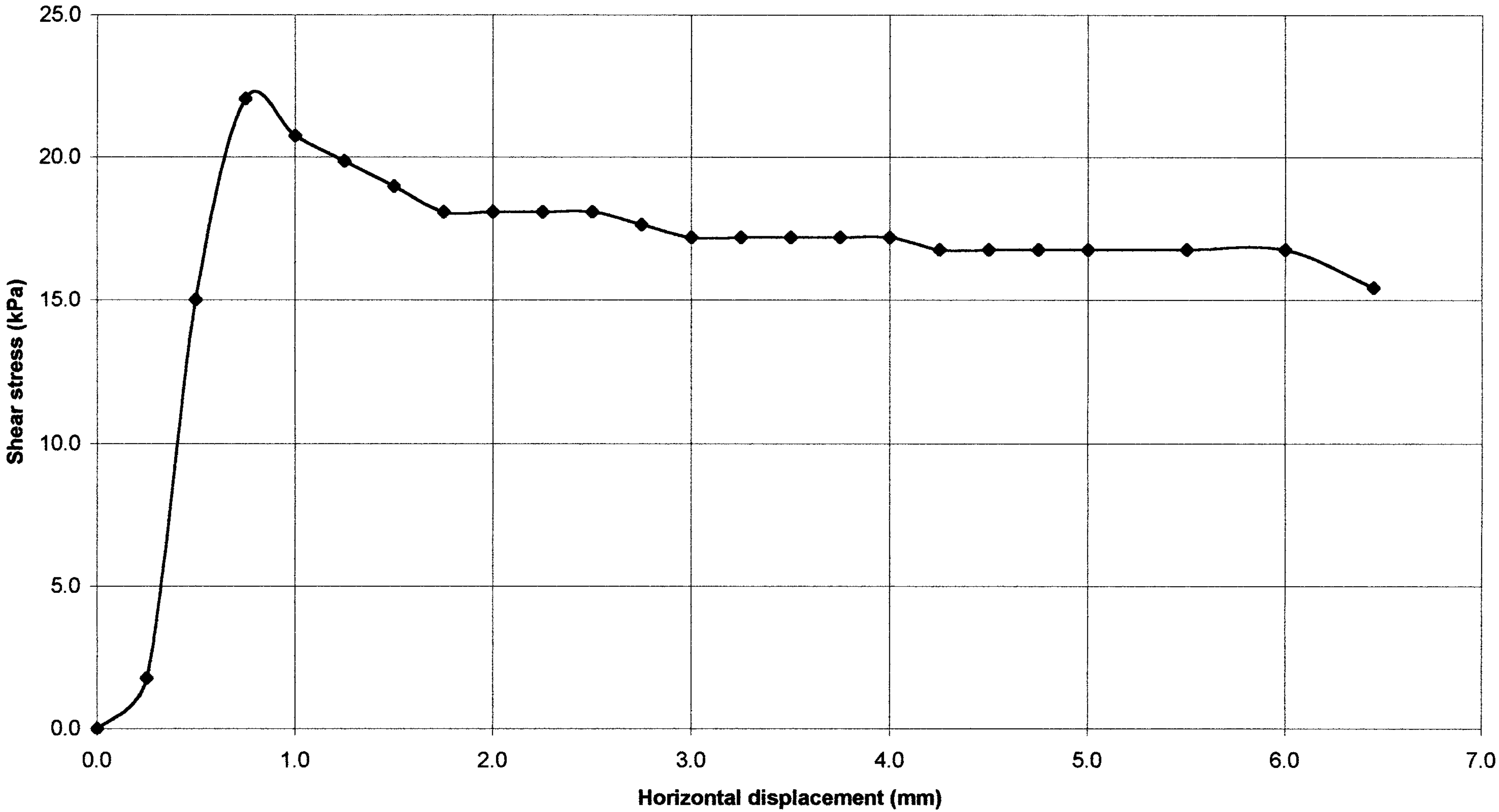
Height of sample: 25.5 mm Applied normal stress (kPa): 50

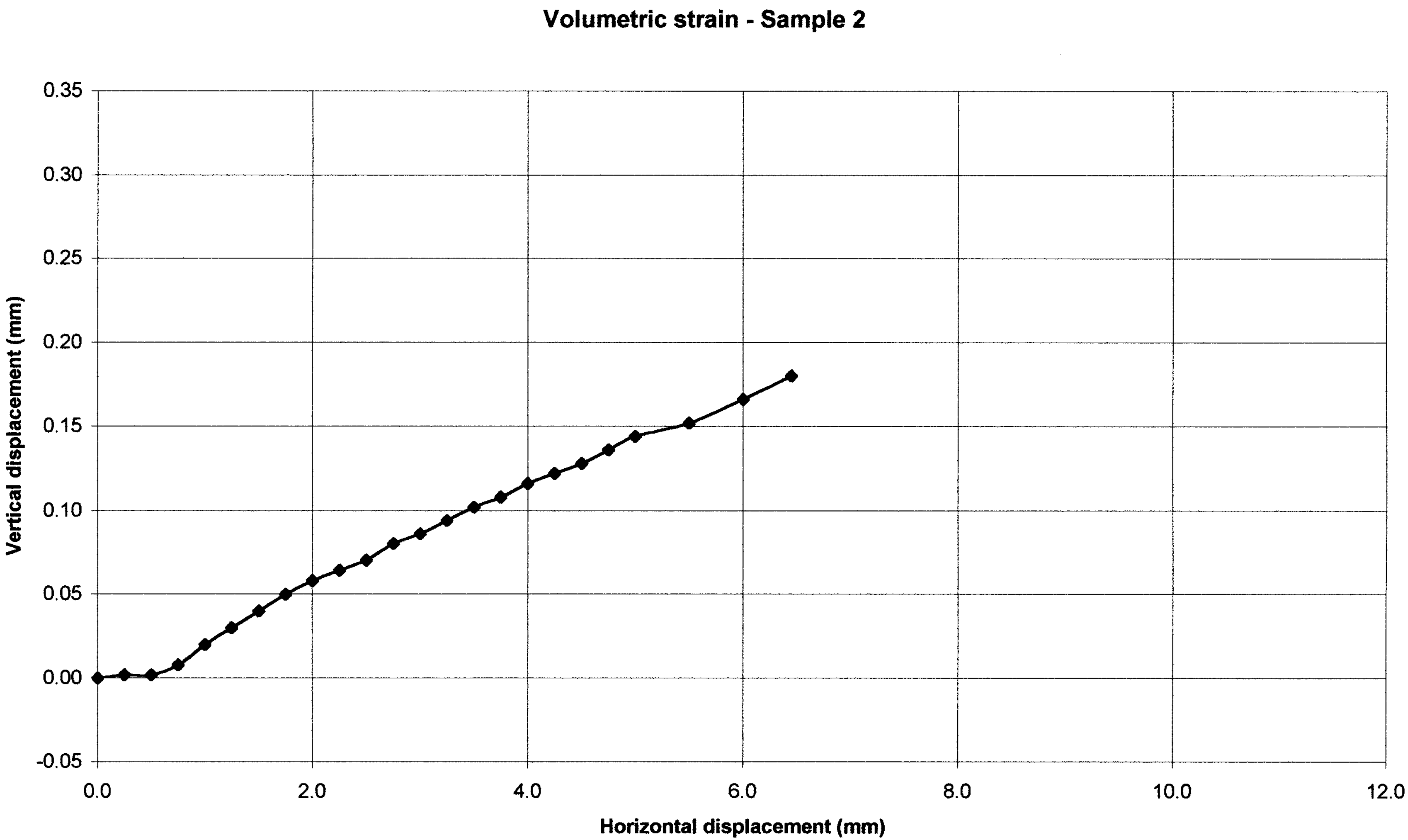
Proof ring number: 4843 Proof ring callibration factor: 4.416N/div

Horizontal dial gauge reading (divisions)	Proof ring dial gauge reading (divisions)	Vertical dial gauge reading (divisions)	Horizontal displacement (mm)	Shear stress (kPa)	Vertical displacement (mm)
400	0	1000	0.0	0.0	0
425	4	1001	0.3	1.8	0.002
450	34	1001	0.5	15.0	0.002
475	50	1004	0.8	22.1	0.008
500	47	1010	1.0	20.8	0.020
525	45	1015	1.3	19.9	0.030
550	43	1020	1.5	19.0	0.040
575	41	1025	1.8	18.1	0.050
600	41	1029	2.0	18.1	0.058
625	41	1032	2.3	18.1	0.064
650	41	1035	2.5	18.1	0.070
675	40	1040	2.8	17.7	0.080
700	39	1043	3.0	17.2	0.086
725	39	1047	3.3	17.2	0.094
750	39	1051	3.5	17.2	0.102
775	39	1054	3.8	17.2	0.108
800	39	1058	4.0	17.2	0.116
825	38	1061	4.3	16.8	0.122
850	38	1064	4.5	16.8	0.128
875	38	1068	4.8	16.8	0.136
900	38	1072	5.0	16.8	0.144
950	38	1076	5.5	16.8	0.152
1000	38	1083	6.0	16.8	0.166
1045	35	1090	6.5	15.5	0.180

Rate of strain measured: 0.015 mm/s

Shearbox test result - Sample 2




Volumetric strain - Sample 2
Vertical displacement (mm)
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
-0.05
0.0
2.0
4.0
6.0
8.0
10.0
12.0
Horizontal displacement (mm)

Remolded box shear test

Project: Hillendale Sample: 3 Date: 17/8/1999

Area of sample: 0.01 m^2 Height of sample before consolidation: 29.3 mm
Initial volume of sample: 0.000293 m^3 Height of sample after consolidation: 25.0 mm
Final volume of sample: 0.000250 m^3

Mould (g) 308.348

	Sample	Trimmings		
Container number	1	102	105	
Mass of wet sample + mould/cont before test	752.72	86.083	69.89	
Mass of wet sample + container after test	565.7			
Mass of dry sample + container	411.14	61.878	50.184	
Mass of container	173.55	27.896	25.124	
Moisture content before test (%)	87.03	71.23	78.64	
Moisture content after test (%)	65.06			

Before test:
Bulk density (kg/m^3) 1517 Dry density (kg/m^3 886
After test:
Bulk density (kg/m^3) 1569 Dry density (kg/m^3 951

Consolidation

Applied normal stress (kPa)= 100 vertical dial gauge division = 0.002

Time	time (sqrt min)	vertical dial gauge reading (divisions)	settle-ment (mm)
Settle more than 2000 div on first attempt,			
sample probably not fully inserted.			
0	0.0	0.00	0.000
5.s	0.3	155	0.310
10s	0.4	175	0.350
20.0s	0.6	180	0.360
40s	0.8	200	0.400
1.0m	1.0	215	0.430
2m	1.4	320	0.640
4.0m	2.0	320	0.640
6m	2.5	344	0.688
9.0m	3.0	424	0.848
12.25m	3.5	475	0.950
16.0m	4.0	535	1.070
25.0m	5.0	640	1.280
36m	6.0	757	1.514
49.0m	7.0	876	1.752
1h04m	8.0	1000	2.000
1h21m	9.0	1120	2.240
1h40m	10.0	1230	2.460
3h45m	15.0	1665	3.330
19h50m	34.5	2118	4.236
20h30m	35	2123	4.246
27h50m	41	2128	4.256
43h02m	51	2153	4.306

sqrt t90 = 16 min cv = 0.848 H^2 / t90 = 2.07E-06 m^2/min : **1.09** m^2/year

Shear box Test
Shear Data

Project: Hillendale Sample: 3 Date: 19/8/99

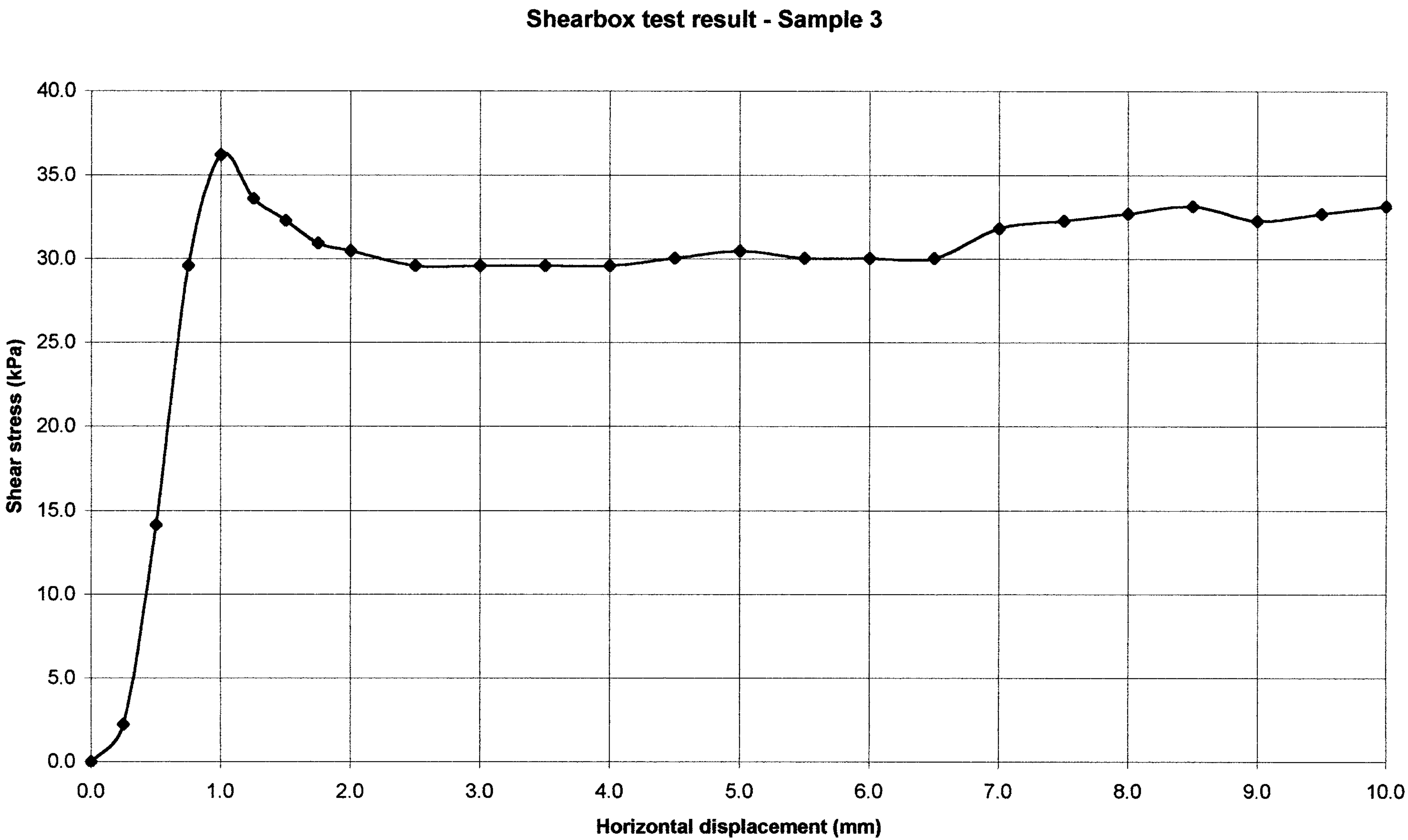
Horizontal dial gauge division: 0.01 Vertical dial gauge division: 0.002

Height of sample: 25 mm Applied normal stress (kPa): 100

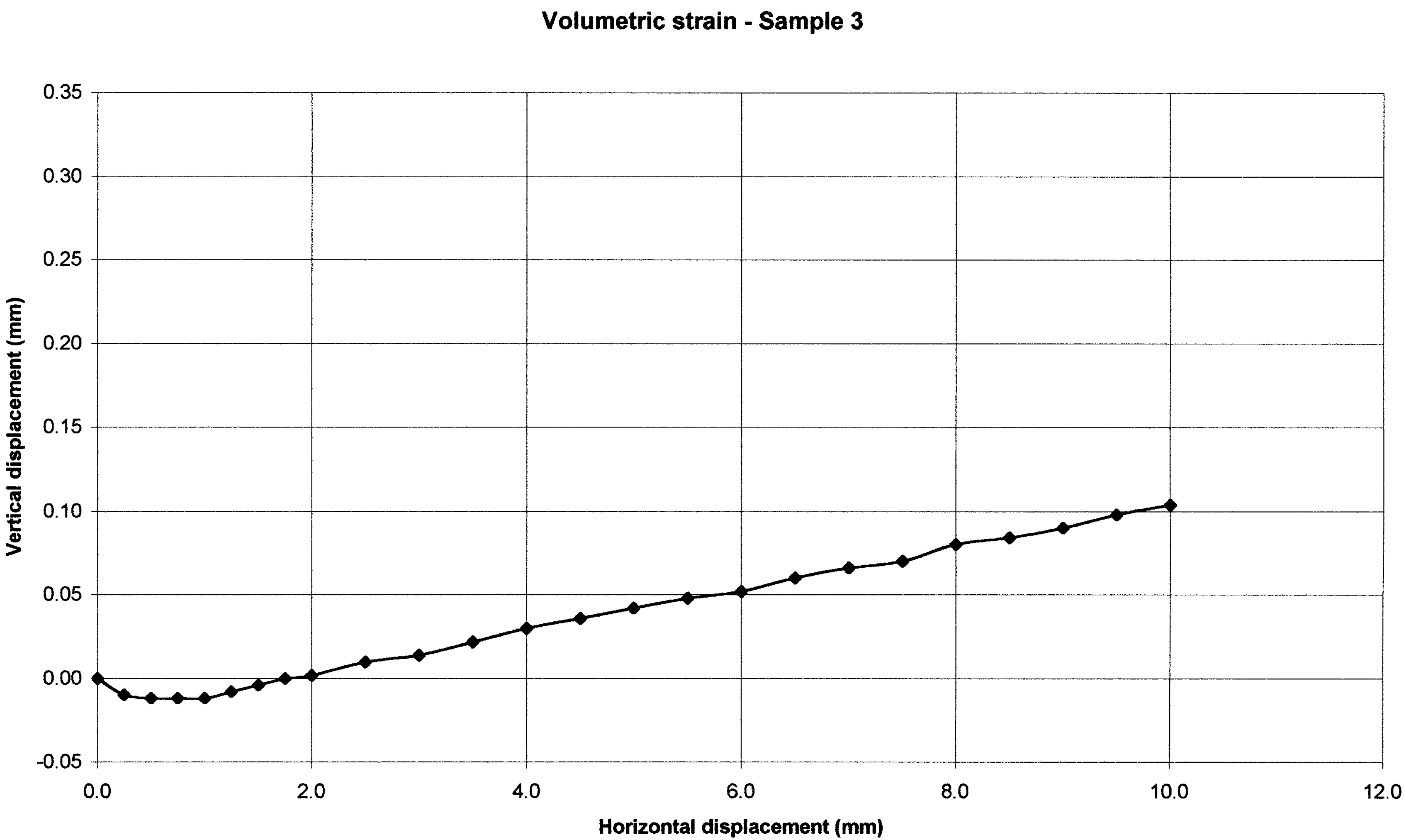
Proof ring number: 4843 Proof ring callibration factor: 4.416N/div

Horizontal dial gauge reading (divisions)	Proof ring dial gauge reading (divisions)	Vertical dial gauge reading (divisions)	Horizontal displacement (mm)	Shear stress (kPa)	Vertical displacement (mm)
400	0	1000	0.0	0.0	0
425	5	995	0.3	2.2	-0.010
450	32	994	0.5	14.1	-0.012
475	67	994	0.8	29.6	-0.012
500	82	994	1.0	36.2	-0.012
525	76	996	1.3	33.6	-0.008
550	73	998	1.5	32.2	-0.004
575	70	1000	1.8	30.9	0.000
600	69	1001	2.0	30.5	0.002
650	67	1005	2.5	29.6	0.010
700	67	1007	3.0	29.6	0.014
750	67	1011	3.5	29.6	0.022
800	67	1015	4.0	29.6	0.030
850	68	1018	4.5	30.0	0.036
900	69	1021	5.0	30.5	0.042
950	68	1024	5.5	30.0	0.048
1000	68	1026	6.0	30.0	0.052
1050	68	1030	6.5	30.0	0.060
1100	72	1033	7.0	31.8	0.066
1150	73	1035	7.5	32.2	0.070
1200	74	1040	8.0	32.7	0.080
1250	75	1042	8.5	33.1	0.084
1300	73	1045	9.0	32.2	0.090
1350	74	1049	9.5	32.7	0.098
1400	75	1052	10.0	33.1	0.104

Rate of strain measured: 0.015 mm/s



Shearbox test result - Sample 3
Shear stress (kPa)
Horizontal displacement (mm)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0


Volumetric strain - Sample 3
Vertical displacement (mm)
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
-0.05
0.0
2.0
4.0
6.0
8.0
10.0
12.0
Horizontal displacement (mm)

Undisturbed/remolded ring/box shear test

Project: Hillendale Sample: 4 Date: 19/8/1999

Area of sample: 0.01 m^2
Initial volume of sample: 0.000293 m^3
Final volume of sample: 0.000197 m^3

Height of sample before consolidation: 29.3 mm
Height of sample after consolidation: 19.7 mm

Mould (g): 308.348

	Sample	Trimmings		
Container number	1	109	102	
Mass of wet sample + mould/cont before test	749.949	64.314	71.837	
Mass of wet sample + container after test	510.644			
Mass of dry sample + container	391.443	46.728	51.725	
Mass of container	173.536	25.334	27.898	
Moisture content before test (%)	102.66	82.20	84.41	
Moisture content after test (%)	54.70			

Before test:
Bulk density (kg/m^3) 1507 Dry density (kg/m^3) 827
After test:
Bulk density (kg/m^3) 1710 Dry density (kg/m^3) 1105

Consolidation

Applied normal stress (kPa)= 200 vertical dial gauge division = 0

Time	time (sqrt min)	vertical dial gauge reading (divisions)	settle-ment (mm)
0	0.0	0.00	0.000
5.s	0.3	725	1.450
10s	0.4	750	1.500
20.0s	0.6	765	1.530
40s	0.8	790	1.580
1.0m	1.0	810	1.620
2m	1.4	852	1.704
4.0m	2.0	918	1.836
9.0m	3.0	1045	2.090
16.0m	4.0	1195	2.390
25.0m	5.0	1355	2.710
36m	6.0	1534	3.068
54m	7.3	1796	3.592
1h04m	8.0	1930	3.860
1h22m	9.0	2150	4.300
1h40m	10.0	2324	4.648
3h15m	14.0	3193	6.386
4h52m	17	3810	7.620
6h13m	19.3	4183	8.366
24h19m	38.2	4717	9.434
98h00m	76.7	4792	9.584

sqrt t90 = 22.5 min cv = 0.848 H^2 / t90 = 6.5E-07 m^2/min = **0.34** m^2/year

Shear box Test
Shear Data

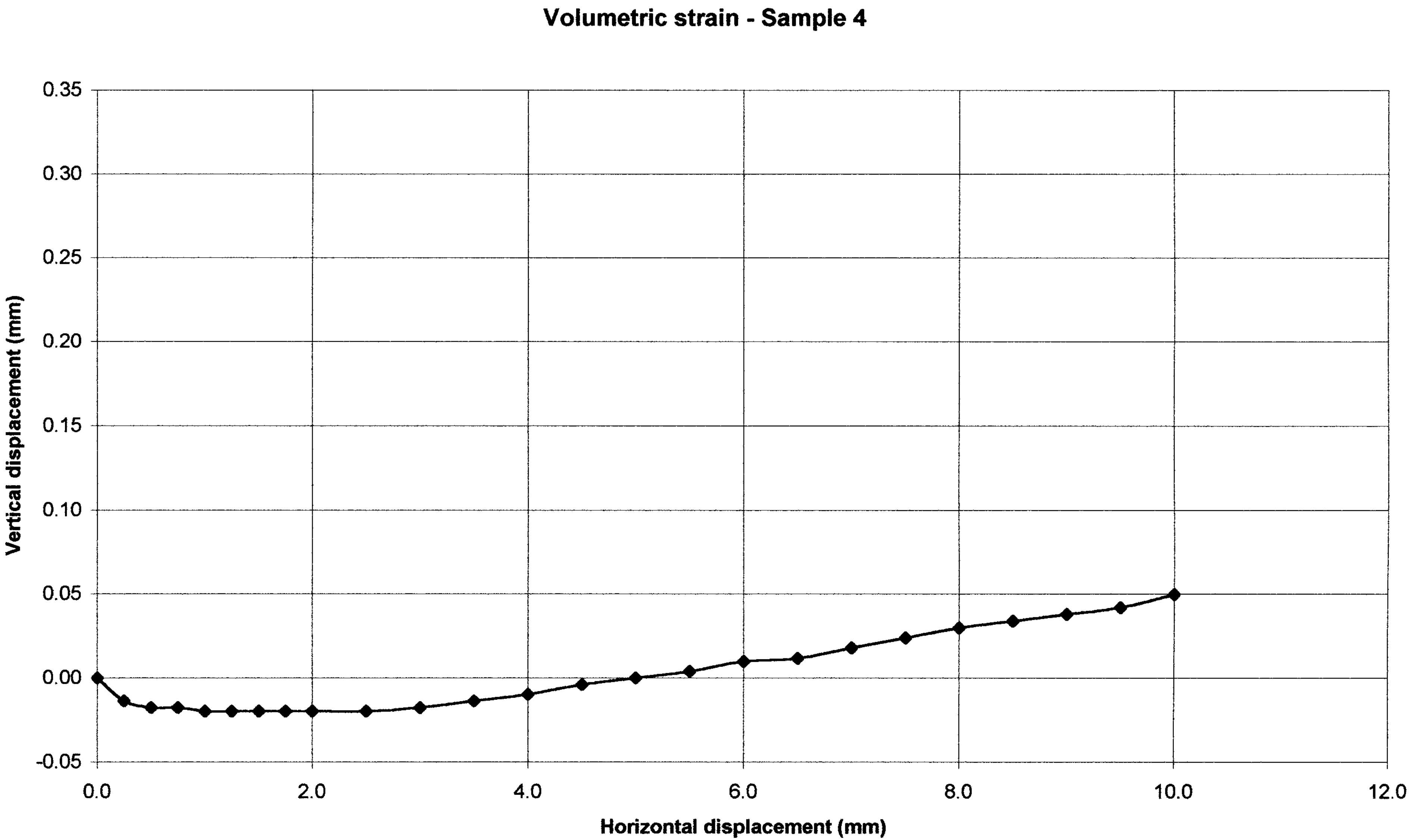
Project: Hillendale   Sample: 4   Date: 23/8/99

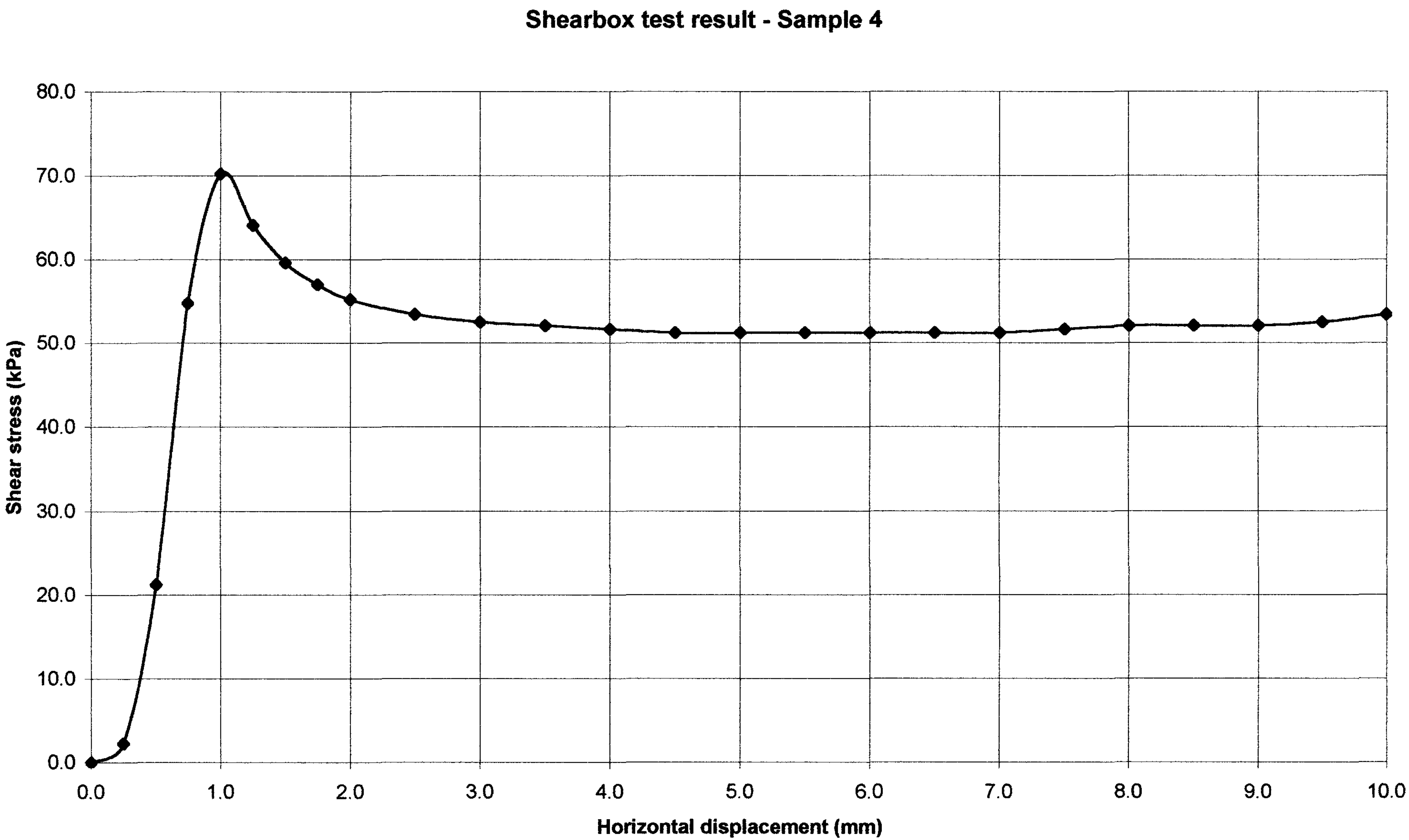
Horizontal dial gauge division: 0.01   Vertical dial gauge division: 0.002

Height of sample: 19.7 mm   Applied normal stress (kPa): 200

Proof ring number: 4843   Proof ring callibration factor: 4.416N/div

Horizontal dial gauge reading (divisions)	Proof ring dial gauge reading (divisions)	Vertical dial gauge reading (divisions)	Horizontal displace ment (mm)	Shear stress (kPa)	Vertical displace ment (mm)
400	0	1000	0.0	0.0	0
425	5	993	0.3	2.2	-0.014
450	48	991	0.5	21.2	-0.018
475	124	991	0.8	54.8	-0.018
500	159	990	1.0	70.2	-0.020
525	145	990	1.3	64.0	-0.020
550	135	990	1.5	59.6	-0.020
575	129	990	1.8	57.0	-0.020
600	125	990	2.0	55.2	-0.020
650	121	990	2.5	53.4	-0.020
700	119	991	3.0	52.6	-0.018
750	118	993	3.5	52.1	-0.014
800	117	995	4.0	51.7	-0.010
850	116	998	4.5	51.2	-0.004
900	116	1000	5.0	51.2	0.000
950	116	1002	5.5	51.2	0.004
1000	116	1005	6.0	51.2	0.010
1050	116	1006	6.5	51.2	0.012
1100	116	1009	7.0	51.2	0.018
1150	117	1012	7.5	51.7	0.024
1200	118	1015	8.0	52.1	0.030
1250	118	1017	8.5	52.1	0.034
1300	118	1019	9.0	52.1	0.038
1350	119	1021	9.5	52.6	0.042
1400	121	1025	10.0	53.4	0.050


Volumetric strain - Sample 4
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
-0.05
Vertical displacement (mm)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
Horizontal displacement (mm)


Shearbox test result - Sample 4
Shear stress (kPa)
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
Horizontal displacement (mm)

APPENDIX E:

FIGURE 4.6: MOISTURE CONTENT VS SHEAR STRENGTH FOR THE SHEAR BOX TEST RESULTS

FIGURE 4.7: COMPARISON BETWEEN SHEAR VANE AND SHEARBOX TEST

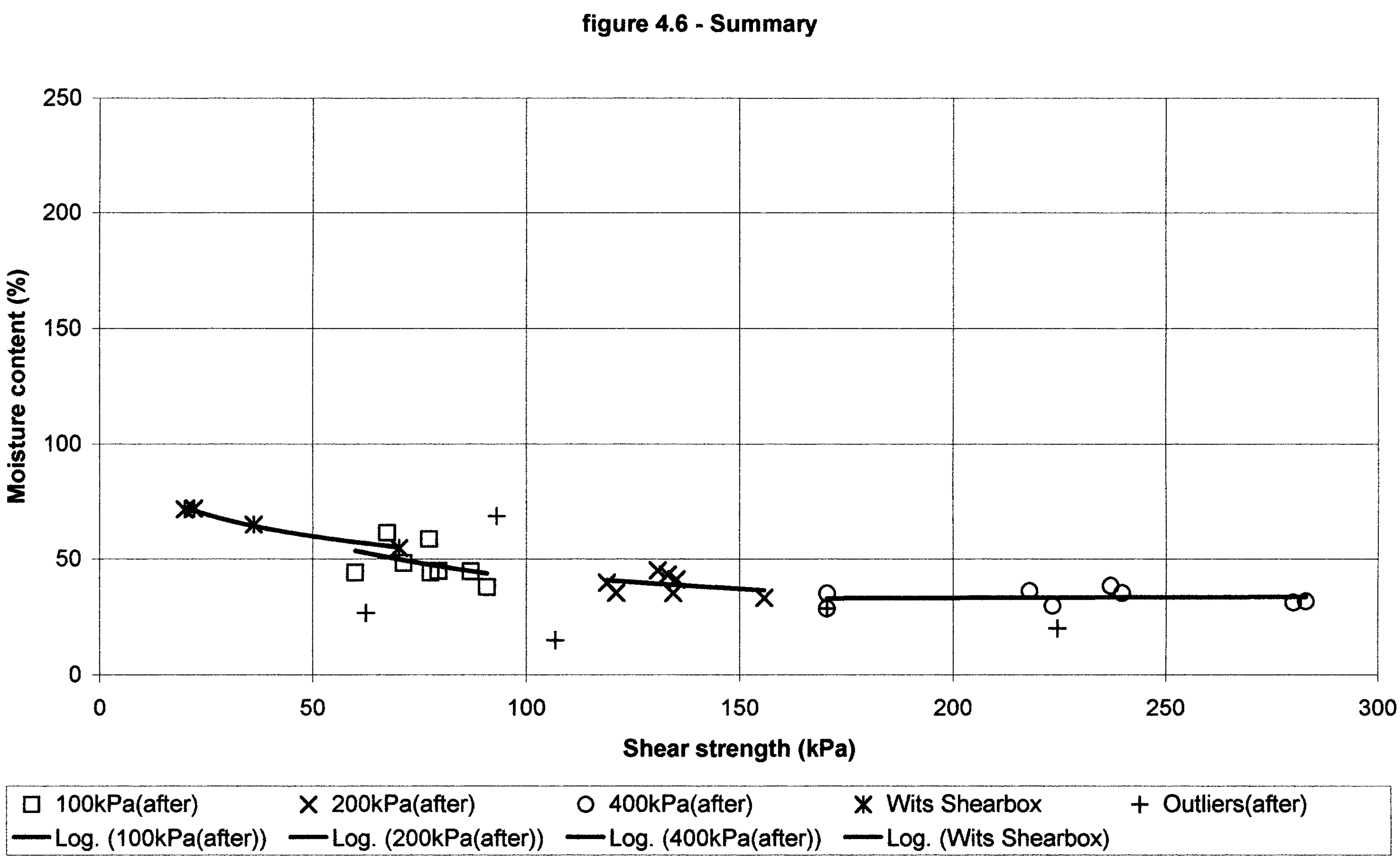
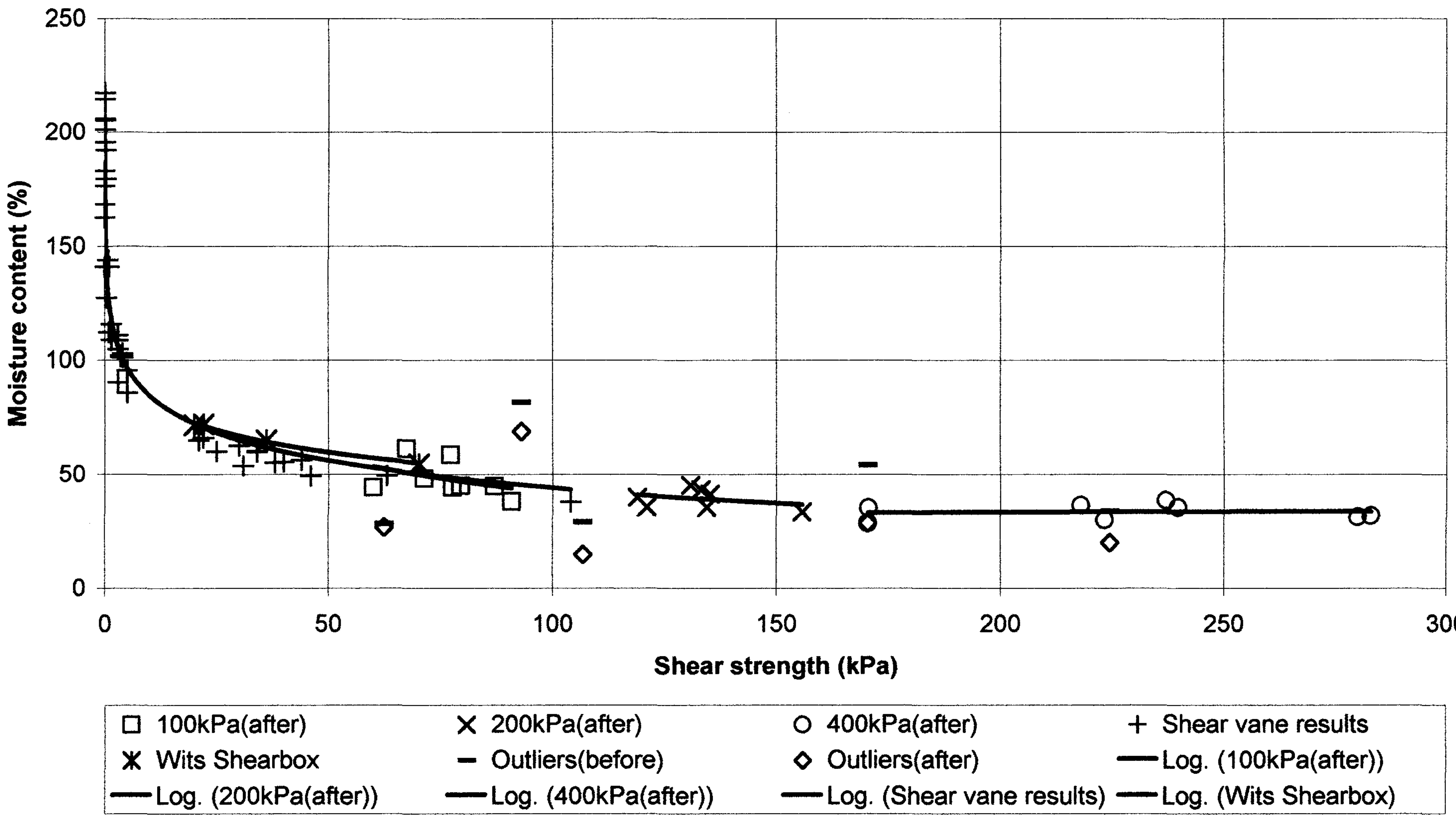


figure 4.6 - Summary
Moisture content (%)
250
200
150
100
50
0
0
50
100
150
200
250
300
Shear strength (kPa)
100kPa(after)
200kPa(after)
400kPa(after)
Wits Shearbox
Outliers(after)
Log. (100kPa(after))
Log. (200kPa(after))
Log. (400kPa(after))
Log. (Wits Shearbox)

Figure 4.7 - Summary of shear box tests and shear vane tests



APPENDIX F:

INPUT PARAMETERS USED IN THE STABILITY ANALYSIS

** STABL6H **
by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

Run Date:	7/12/2001
Time of Run:	9:45pm
Run By:	A du Plessis
Input Data Filename:	C:SCEN115.SI
Output Filename:	C:SCEN115.OUT
Plotted Output Filename:	C:SCEN115.PLT

PROBLEM DESCRIPTION Stability analysis: Scenario I
Constant water depth and initial slope

BOUNDARY COORDINATES

10 Top Boundaries
19 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	20.00	10.00	20.00	1
2	10.00	20.00	15.00	25.00	2
3	15.00	25.00	25.00	25.00	2
4	25.00	25.00	32.50	27.50	8
5	32.50	27.50	40.00	30.00	7
6	40.00	30.00	47.50	32.50	6
7	47.50	32.50	55.00	35.00	5
8	55.00	35.00	62.50	37.50	4
9	62.50	37.50	70.00	40.00	3
10	70.00	40.00	100.00	39.60	3
11	62.50	37.50	100.00	37.00	4
12	55.00	35.00	100.00	34.40	5
13	47.50	32.50	100.00	31.80	6
14	40.00	30.00	100.00	29.30	7
15	32.50	27.50	100.00	26.70	8
16	25.00	25.00	100.00	24.10	9
17	27.50	22.50	100.00	21.60	10
18	25.00	25.00	30.00	20.00	2
19	10.00	20.00	100.00	20.00	1

ISOTROPIC SOIL PARAMETERS

10 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.

1	12.5	12.5	10.0	20.0	.00	.0	1
2	18.8	18.8	5.0	35.0	.00	.0	1
3	14.4	14.4	4.8	.0	.00	.0	1
4	14.9	14.9	10.8	.0	.00	.0	1
5	15.5	15.5	18.0	.0	.00	.0	1
6	15.6	15.6	23.1	.0	.00	.0	1
7	15.7	15.7	27.1	.0	.00	.0	1
8	16.0	16.0	34.0	.0	.00	.0	1
9	16.0	16.0	39.0	.0	.00	.0	1
10	16.1	16.1	44.0	.0	.00	.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 9.80

Piezometric Surface No. 1 Specified by 5 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	.00	17.00
2	10.00	20.00
3	27.50	22.50
4	70.00	38.00
5	100.00	37.60

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

500 Trial Surfaces Have Been Generated.

100 Surfaces Initiate From Each Of 5 Points Equally Spaced Along The Ground Surface Between X = 25.00 ft.
and X = 30.00 ft.

Each Surface Terminates Between X = 40.00 ft.
and X = 99.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = .00 ft.

2.00 ft. Line Segments Define Each Trial Failure Surface.

1

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	25.00	25.00
2	26.79	24.11
3	28.61	23.29
4	30.47	22.55
5	32.36	21.90
6	34.28	21.32
7	36.22	20.83
8	38.18	20.43
9	40.15	20.11
10	42.14	19.88
11	44.13	19.73
12	46.13	19.67
13	48.13	19.69
14	50.13	19.81
15	52.12	20.00
16	54.10	20.29
17	56.06	20.66
18	58.01	21.12
19	59.94	21.65
20	61.84	22.28
21	63.71	22.98
22	65.55	23.76
23	67.35	24.63
24	69.12	25.57
25	70.84	26.58
26	72.52	27.67
27	74.15	28.84
28	75.72	30.07
29	77.24	31.37
30	78.71	32.73
31	80.11	34.15
32	81.45	35.64
33	82.73	37.18
34	83.93	38.77
35	84.65	39.80

Circle Center At X = 46.5 ; Y = 65.8 and Radius, 46.1

*** 1.175 ***

Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	25.00	25.00
2	26.75	24.03
3	28.54	23.14
4	30.37	22.34
5	32.24	21.62
6	34.14	21.00
7	36.07	20.46
8	38.02	20.02
9	39.99	19.67
10	41.97	19.41
11	43.96	19.25
12	45.96	19.18

13	47.96	19.21
14	49.96	19.33
15	51.95	19.55
16	53.92	19.86
17	55.88	20.27
18	57.82	20.76
19	59.73	21.35
20	61.61	22.03
21	63.46	22.80
22	65.27	23.65
23	67.03	24.59
24	68.75	25.61
25	70.42	26.71
26	72.04	27.89
27	73.60	29.14
28	75.09	30.47
29	76.52	31.86
30	77.89	33.33
31	79.18	34.85
32	80.40	36.44
33	81.54	38.08
34	82.61	39.77
35	82.64	39.83

Circle Center At X = 46.4 ; Y = 61.4 and Radius, 42.2

*** 1.176 ***

1

Failure Surface Specified By 30 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	30.00	26.67
2	31.81	25.81
3	33.65	25.04
4	35.54	24.37
5	37.45	23.79
6	39.39	23.31
7	41.36	22.93
8	43.34	22.65
9	45.33	22.47
10	47.33	22.40
11	49.33	22.42
12	51.32	22.55
13	53.31	22.77
14	55.28	23.10
15	57.24	23.53
16	59.17	24.05
17	61.07	24.68
18	62.93	25.40
19	64.76	26.21
20	66.55	27.11
21	68.28	28.11
22	69.97	29.19
23	71.59	30.35
24	73.16	31.59
25	74.66	32.92
26	76.09	34.31
27	77.45	35.78
28	78.73	37.32
29	79.93	38.92

Circle Center At X = 47.8 ; Y = 61.8 and Radius, 39.4

*** 1.180 ***

Failure Surface Specified By 30 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	28.75	26.25
2	30.59	25.47
3	32.47	24.78
4	34.38	24.18
5	36.31	23.68
6	38.27	23.26
7	40.24	22.94
8	42.23	22.72
9	44.23	22.59
10	46.23	22.56
11	48.23	22.62
12	50.22	22.78
13	52.20	23.04
14	54.17	23.39
15	56.12	23.83
16	58.05	24.37
17	59.95	24.99
18	61.81	25.71
19	63.64	26.52
20	65.43	27.42
21	67.18	28.39
22	68.87	29.46
23	70.51	30.60
24	72.10	31.82
25	73.63	33.11
26	75.09	34.48
27	76.48	35.91
28	77.81	37.41
29	79.06	38.97
30	79.71	39.87

Circle Center At X = 45.9 ; Y = 64.2 and Radius, 41.7

*** 1.180 ***

Failure Surface Specified By 32 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	26.25	25.42
2	28.07	24.59
3	29.93	23.84
4	31.81	23.18
5	33.73	22.62
6	35.67	22.14
7	37.64	21.75
8	39.61	21.46
9	41.60	21.26

11	45.60	21.14
12	47.60	21.22
13	49.59	21.39
14	51.57	21.65
15	53.54	22.01
16	55.49	22.46
17	57.42	23.01
18	59.31	23.64
19	61.18	24.36
20	63.01	25.16
21	64.80	26.05
22	66.55	27.03
23	68.25	28.08
24	69.90	29.21
25	71.49	30.42
26	73.02	31.71
27	74.50	33.06
28	75.91	34.48
29	77.25	35.96
30	78.52	37.51
31	79.71	39.11
32	80.22	39.86

Circle Center At X = 44.9 ; Y = 63.8 and Radius, 42.7

*** 1.186 ***

Failure Surface Specified By 32 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	27.50	25.83
2	29.23	24.83
3	31.01	23.92
4	32.84	23.11
5	34.71	22.39
6	36.61	21.77
7	38.54	21.26
8	40.50	20.85
9	42.48	20.54
10	44.47	20.34
11	46.46	20.24
12	48.46	20.25
13	50.46	20.37
14	52.45	20.59
15	54.42	20.92
16	56.37	21.35
17	58.30	21.88
18	60.20	22.52
19	62.06	23.25
20	63.88	24.09
21	65.65	25.01
22	67.37	26.03
23	69.03	27.14
24	70.63	28.34
25	72.17	29.62
26	73.64	30.98
27	75.03	32.41
28	76.35	33.92
29	77.58	35.49
30	78.73	37.13
31	79.79	38.83

Circle Center At X = 47.3 ; Y = 58.0 and Radius, 37.7

*** 1.186 ***

1

Failure Surface Specified By 33 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	27.50	25.83
2	29.39	25.18
3	31.31	24.60
4	33.24	24.09
5	35.19	23.66
6	37.16	23.29
7	39.14	22.99
8	41.12	22.77
9	43.12	22.62
10	45.12	22.54
11	47.12	22.54
12	49.11	22.61
13	51.11	22.75
14	53.10	22.96
15	55.08	23.25
16	57.04	23.61
17	59.00	24.04
18	60.93	24.54
19	62.85	25.11
20	64.75	25.75
21	66.61	26.46
22	68.46	27.24
23	70.27	28.09
24	72.05	29.00
25	73.80	29.97
26	75.51	31.01
27	77.18	32.11
28	78.81	33.27
29	80.39	34.49
30	81.93	35.76
31	83.43	37.09
32	84.87	38.48
33	86.14	39.78

Circle Center At X = 46.2 ; Y = 77.3 and Radius, 54.8

*** 1.188 ***

Failure Surface Specified By 32 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	27.50	25.83
2	29.40	25.19
3	31.31	24.63
4	33.25	24.13
5	35.21	23.72
6	37.18	23.37

7	39.16	23.11
8	41.15	22.92
9	43.15	22.81
10	45.15	22.78
11	47.15	22.82
12	49.14	22.94
13	51.13	23.14
14	53.12	23.41
15	55.08	23.76
16	57.04	24.19
17	58.97	24.69
18	60.89	25.27
19	62.78	25.92
20	64.65	26.64
21	66.48	27.43
22	68.29	28.29
23	70.06	29.23
24	71.79	30.23
25	73.48	31.29
26	75.13	32.43
27	76.73	33.62
28	78.29	34.88
29	79.80	36.19
30	81.25	37.56
31	82.65	38.99
32	83.41	39.82

Circle Center At X = 45.0 ; Y = 74.5 and Radius, 51.8

*** 1.188 ***

1

Failure Surface Specified By 32 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	26.25	25.42
2	28.12	24.70
3	30.01	24.07
4	31.94	23.51
5	33.88	23.04
6	35.84	22.65
7	37.82	22.34
8	39.80	22.11
9	41.80	21.98
10	43.80	21.92
11	45.80	21.95
12	47.79	22.06
13	49.78	22.26
14	51.76	22.55
15	53.73	22.91
16	55.68	23.36
17	57.61	23.89
18	59.51	24.51
19	61.39	25.20
20	63.23	25.97
21	65.04	26.82
22	66.82	27.74
23	68.55	28.74
24	70.24	29.81
25	71.88	30.95
26	73.47	32.16

27	75.01	33.44
28	76.50	34.78
29	77.93	36.18
30	79.29	37.64
31	80.59	39.16
32	81.14	39.85

Circle Center At X = 44.1 ; Y = 69.2 and Radius, 47.3

*** 1.190 ***

Failure Surface Specified By 32 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	27.50	25.83
2	29.33	25.03
3	31.20	24.31
4	33.10	23.68
5	35.02	23.13
6	36.97	22.67
7	38.93	22.29
8	40.91	22.01
9	42.90	21.81
10	44.90	21.70
11	46.90	21.69
12	48.90	21.76
13	50.89	21.92
14	52.87	22.17
15	54.84	22.51
16	56.80	22.94
17	58.73	23.46
18	60.64	24.06
19	62.52	24.74
20	64.36	25.51
21	66.17	26.36
22	67.94	27.30
23	69.67	28.31
24	71.35	29.39
25	72.97	30.55
26	74.55	31.79
27	76.07	33.09
28	77.53	34.46
29	78.92	35.89
30	80.25	37.39
31	81.51	38.94
32	82.18	39.84

Circle Center At X = 46.3 ; Y = 66.3 and Radius, 44.6

*** 1.191 ***

1

Y A X I S F T

.00 12.50 25.00 37.50 50.00 62.50

```
X         .00 +---------+---W-*---+---------+---------+---------+
              -
              -
              -
              -
              -               *
        12.50 +
              -                   *
              -
              -
              -
              -
A       25.00 +                   *
              -                 *156
              -               *..163
              -               .21543.
              -              ..1173.*
              -            ...2153...
X       37.50 +            ...1593....
              -           ....1543....*
              -           ....153.......
              -          ....2153.......
              -          ....2153........
              -         .....2153.......*
I       50.00 +         .....2153.........
              -         ......1538........
              -         ......1634........*
              -         ......2133..........
              -         .......1538.........
              -         .......1653..........
S       62.50 +         ........1634........*
              -         ........11034.........
              -         ..........1634........
              -          .........1.384........
              -          ..........1733.....W.*
              -          ...........12634......
        75.00 +           ...........117394....
              -            ............172634..
              -             ............1128334
              -              .............71293
              -               ..............112
              -                ..............71
F       87.50 +                  ..............
              -                   .............
              -                     ...........
              -                       .........
              -                         .  .....
              -                            ....
T      100.00 +              ** * * * *  * * *
```