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Development of a Constitutive Model for a Heaving Clay from Rosebank

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BSc Eng (Civil)

*Thesis prepared in partial fulfilment of the requirements for the degree of Master of
Science in Civil Engineering*

**FRD/UCT
CENTRE FOR RESEARCH INTO
COMPUTATIONAL AND APPLIED MECHANICS**

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DEDICATION

This thesis is dedicated to:

יְהוָה צְבָאוֹת

(Without Whom there would be no heaving clay in the first place)

DECLARATION

The results, calculations, graphs, drawings and computer programs presented in this thesis are essentially my own work, except where otherwise indicated, and have not been submitted for a degree at any other university.

Signed by candidate

K H Wiseman

13 October 1993

ACKNOWLEDGEMENTS

Many people contributed to this research project and its presentation. I would particularly like to thank my supervisors, Professor J B Martin and Dr F Scheele, who have guided me throughout my Master's studies. Professor Martin gave valuable oversight to the project and continued guidance in how to proceed. I have been privileged to work under him. Dr Scheele has maintained a high degree of involvement in the work and has made invaluable suggestions regarding every aspect of it from conception, through research and experimentation and finally presentation.

The contributions of the following persons are also gratefully acknowledged:

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James George took oedometer test readings for most of the experimental work which was of great assistance.

Ian and Gavin Wiseman made arrangements for the provision of soil samples for the experimental programme.

Why Nikki Spottiswoode wanted to help with the graphics escapes me, but I am very grateful that she did.

SYNOPSIS

Heaving clays are partially saturated soils composed of a high fraction of the montmorillonite clay mineral. When exposed to free water they undergo volumetric expansion, which often results in differential movements at the surface and damage to building structures founded on the clay. The economic consequences of such damage is severe. Heaving clay causes in excess of R100 million damage in South Africa each year, making it the country's most significant problem soil.

The best method of dealing with a heaving clay is through appropriate design. This would be facilitated if methods such as finite element analysis were available to designers. The aim of this research project was to develop a constitutive model for an expansive clay which could be numerically implemented within the finite element method.

A review of available literature on expansive clays showed that the heave strain that clay under an applied load will undergo can be expressed in terms of the parameters *percent heave* and *heave pressure*. These parameters are influenced by the degree of moisture changes experienced by the soil, and its dry density.

Various different methods of establishing the *percent heave* and *heave pressure* have been proposed, but the values given by each differ due to the influence of different test stress paths on the results. More detailed examination of the effect of test stress path on the volume of heaving clays was therefore required to resolve the reasons for the differences in the test results, and to give a fuller understanding of the volumetric response of the material for the purposes of developing the constitutive model. Hence a series of laboratory tests was conducted on a clay from Rosebank in the Cape Province to investigate the effect of test stress path on the clay volume.

On the basis of the literature study and experimental programme a constitutive model for the volumetric behaviour of Rosebank clay was proposed. The model is formulated upon the assumption that the overall volumetric response of a heaving clay is a combination of two separate effects. These are consolidation, which is a mechanical response to changes in loading at constant moisture content; and heave, which is a chemical response to changes in moisture content at constant load. The model proposed on this basis is able to account for the different values of *percent heave* and *heave pressure* measured through different test methods, and therefore provides a greater understanding of the overall volume changes experienced by a heaving clay.

The proposed mathematical constitutive model was incorporated into the finite element method in order to test the model and to demonstrate its usefulness to a designer. Several test problems were analysed and showed that the model gives a reasonable approximation of the volumetric behaviour of the heaving clay. However, the shear strength of the clay is not well simulated by the model and therefore it is best suited to problems where the volume changes of the clay are most significant and the shear behaviour of the material will have a minor influence on the solution.

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NOMENCLATURE

Roman Characters

Symbol	Meaning
c	Cohesion component of shear resistance of a soil
E	Tangent to curve relating stresses to strains
e, e_0	Current, Initial (or reference) void ratio of soil
e_{ij}	Deviatoric strain tensor: $e_{ij} = \epsilon_{ij} - 1/3 \epsilon_v \delta_{ij}$
e_m	Edge moisture variation distance under a floor slab build over a heaving clay profile
f	Yield function for a yielding material
G	Shear modulus
G_s	Specific gravity of soil solids particles
H	Heave surface defining the heave strain in terms of the applied stress and soil suction
$I_{1,\sigma}, I_{1,\epsilon}$	First invariants of stress and strain respectively
K, K_{el}, K_{pl}	Bulk modulus, elastic bulk modulus and elastic-plastic bulk modulus respectively
m	Mound exponent defining the shape of a heaving clay dome that develops under a floor slab
M_h	Gradient of <i>percent heave</i> line with respect to the unstressed specific volume
M_p	Gradient of initial <i>heave pressure</i> with respect to the unstressed specific volume
p	Hydrostatic stress: $p = 1/3(\sigma_{11} + \sigma_{22} + \sigma_{33})$
$p_c, p_{cdry}, p_{csat}, p_{c0}$	Preconsolidation pressure or yield stress of a clay: current, partially saturated, fully saturated and initial (or reference)

	values respectively
P_h, P_{h0}	Current and initial (or reference) heave pressures respectively
P_{wet}	Applied hydrostatic pressure at which clay sample is wetted to full saturation
S_{ij}	deviatoric stress tensor: $s_{ij} = \sigma_{ij} - p\delta_{ij}$
S_r	Soil saturation
u_a	Pore air pressure
u_c, u_{c0}	Current and initial (or reference) soil suctions respectively: $u_c = (u_a - u_w)$
u_w	Pore water pressure
v, v_0	Current and initial (or reference) specific volumes of the soil; the total soil volume which contains a unit volume of solids: $v = 1 + e$ The "unstressed specific volume" refers to the value of v when the applied stress on the soil is zero
x	Horizontal distance along a heaving clay dome that develops under a floor slab
y, y_m	Vertical displacement at horizontal position x and maximum vertical displacement of a heaving clay dome that develops under a floor slab, respectively

Greek Characters

Symbol	Meaning
Δ	"Change (or increment) in"
δ_{ij}	the Kronecker delta
ϵ_{ij}	Total strain tensor
ϵ_v	Volumetric strain: $\epsilon_v = I_{1,\epsilon} = (\epsilon_{11} + \epsilon_{22} + \epsilon_{33})$
$\epsilon_v^a, \epsilon_v^e, \epsilon_v^p$	"Adjustment", elastic and plastic volumetric strains respectively
ϵ_v^h	Volumetric heave strain

H, H_0	Current and initial (or reference) value for the <i>Percent heave</i>
κ	Elastic logarithmic bulk modulus: slope of the elastic response line in a plot of the volumetric strain versus the logarithm of the applied hydrostatic stress
$\kappa_{dry}, \kappa_{sat}$	Elastic logarithmic bulk modulus for partially and fully saturated clay respectively
λ	Elastic-plastic logarithmic bulk modulus: slope of the elastic-plastic response line in a plot of the volumetric strain versus the logarithm of the applied hydrostatic stress
$\lambda_{dry}, \lambda_{sat}$	Elastic-plastic logarithmic bulk modulus for partially and fully saturated clay respectively
ν	Poisson's ratio
ρ, ρ_0	Current and initial (or reference) dry densities of a soil respectively
σ_{ij}	Stress tensor (Cauchy)
ϕ	Angle of friction for shear resistance of a soil
χ	The fraction of the area of a plane through a soil upon which the soil suction acts. χ is not a constant but a function of soil saturation and stress path to which the soil has been subjected
ψ	Phew! Finished!

CHAPTER 1

INTRODUCTION

A few years after commencement of the construction of a tower in Pisa in 1174 the phenomenon of settlement of clays was strikingly illustrated. However, "heaving" or "expansion" of clay was not recognised until far later, and the cracks and jamming of doors and windows that developed in certain buildings were blamed on shoddy workmanship and materials^{1,2}. Engineers in the United States first recognised the problem in 1930¹, and in South Africa in the 1940's². It has been estimated that the direct cost of repairs to structures damaged by heaving clays in South Africa every year is in excess of R100 million³, and in the United States heaving clay damage costs more than twice the damage from floods, hurricanes tornadoes and earthquakes⁴. Heaving clays are thus a very significant problem soil in South Africa and other parts of the world.

Soils which heave generally have a high clay fraction, and a high clay activity. The montmorillonite clay mineral must also be present in significant quantities. The clay must also be desiccated to some extent (ie partially saturated). Heaving occurs when free water moves into the clay matrix, increasing the degree of saturation and resulting in volumetric expansion of the soil. This expansion results in heaving of the soil surface. Where differential soil movements occur this results in damage to walls and floor slabs of structures. If the structure is founded on piles then the upward movement of soil around the pile causes friction along the shaft which can potentially crack the pile or move it upwards.

Repairs to damage by heaving clays can cost up to 40% of the original value of a structure². The best solution to the problem is to take precautions at the design stage. This involves identifying whether the soil in the area is potentially expansive, estimating the magnitude of total and differential heave that will occur, and then designing the

structure to cope with the anticipated foundation movements. The task of predicting soil movements in advance is difficult and would be assisted by tools such as mathematical models of the soil behaviour and analysis methods such as finite element analysis. It was this need for analytical tools that prompted the research work presented in this thesis. A complete motivation of the need for developing a constitutive model for heaving clay is presented in Chapter 2.

The research work proceeded in the following manner. After establishing the need for development of a model for expansive clay, a literature study was conducted as a basis for the development of the constitutive model. The salient points of the study are presented in Chapters 3 and 4. It became apparent after the study that there was insufficient information available on which to base the development of a general constitutive model for heaving clays. It was therefore necessary to conduct a series of experiments on a potentially expansive clay dug from a site in Rosebank, Cape Town in order to establish the characteristics of its volumetric behaviour. The laboratory work and the model of the clay behaviour developed from the test results are presented in Chapters 5 and 6.

Once a mathematical model of the volumetric behaviour of the clay had been established it was possible to develop the procedures for numerical modelling of the clay. This was done by writing a user defined material subroutine for the finite element package *ABAQUS*⁵. The numerical procedure and the results of some numerical analyses are presented in Chapter 7.

The thesis closes with a discussion of the research work and the presentation of conclusions in Chapter 8.

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CHAPTER 2

THE NEED FOR A CONSTITUTIVE MODEL FOR HEAVING CLAY

The first structures built on many expansive clay profiles were timber dwellings or steel frame structures. These are flexible and able to accommodate differential movements of a heaving clay profile. Heavier and more rigid structures such as brick dwellings and industrial facilities cannot tolerate such movements so they are susceptible to damage from heave. The simplest solution would be to avoid building on such soils, but in many places, and particularly in South Africa, this is no longer practical. Figure 2.1 gives the distribution of heaving clays in South Africa, and shows that area of most significant heave activity is in the Pretoria-Witwatersrand-Vereeniging region, which is the industrial and economic heart of the country. The Lethabo Power Station near Vereeniging is founded on heaving clay because despite more expensive foundations the cost was still cheaper than transporting the coal from Vereeniging to an area without heaving clay.

The option of avoiding heaving clay when building is often not available. Further, the size and complexity of the structures being founded on heaving clays is increasing. Retaining walls, embankments and piled foundations are being installed in such clay, presenting greater challenges to the designer than the smaller and cheaper dwellings that were previously built in these areas. The expense of large installations justifies the costs of detailed soil testing and the use of techniques such as the finite element method. This means that the development of a constitutive model for heaving clays is desirable, and provided the motivation for this research project.

Most research into heaving clay behaviour in the past has concentrated on its volumetric behaviour as it expands when wetted. Consequently a significant number of experimental observations of the expansive behaviour are available. However, the volume of the clay is not the only factor that is effected during wetting. The shear

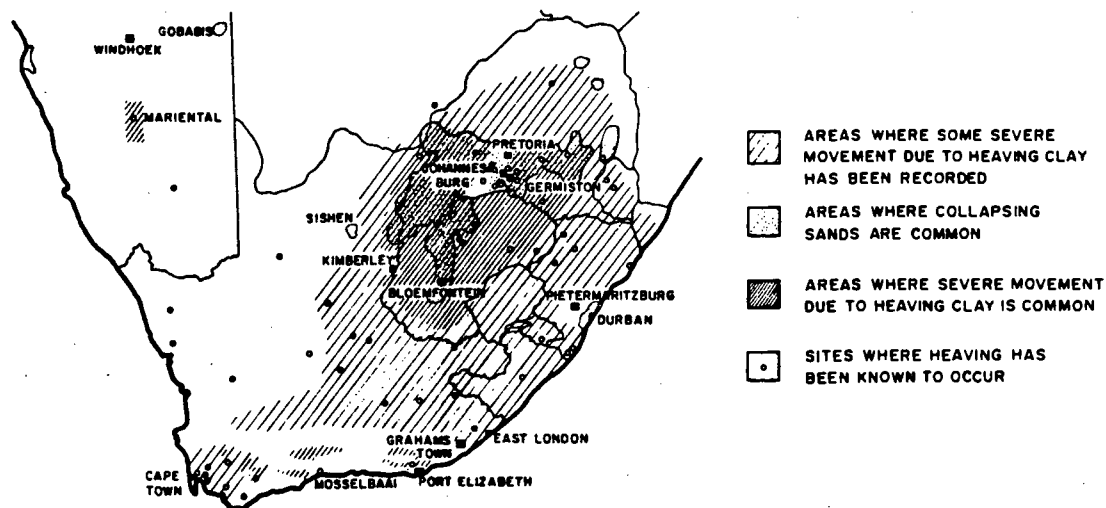


Figure 2.1 Distribution of heaving clays in South Africa (after Williams, Pidgeon and Day¹).

resistance tends to decrease noticeably as the clay becomes more saturated. This aspect has also been investigated, but has not been combined with the observed volumetric behaviour to form a full constitutive model for heaving clay.

The preoccupation of researchers with the volumetric behaviour of the clay under wetting has had two important consequences. Firstly, the volumetric behaviour of the clay under different loading paths at different saturation conditions has not been studied in great detail. This means that significant aspects of the clay behaviour which would have to be described by a constitutive model have not been investigated thoroughly.

A second consequence of the research focus has been a lack of clarity over the meaning and definition of some of the parameters which characterise heave. Different test procedures have led to different values being measures for the same parameter of the same clay. The type of tests that have been conducted have not been carried

sufficiently far to resolve the reason for the observed discrepancies.

This research project therefore sought to develop a constitutive model for the following reasons. Firstly, it would make a sophisticated design tool available to designers in the form of the finite element method, allowing them to analyse advanced problems involving the volumetric and shear behaviour of heaving clay. Secondly, it would allow for more a more detailed consideration of the influence of soil moisture conditions on heave movements. And thirdly, the experimental research conducted would lead to a greater understanding of the volumetric behaviour of heaving clay under loading and wetting, thereby clarifying the discrepancies observed while measuring heave parameters.

The aims of the project described above led to the following plan of execution. Firstly, a literature survey was conducted to establish the characteristics of heaving clay behaviour as identified by other researchers. This also showed what sort of tests and testing procedures would be required to supplement the information available in literature. Experimental research was then performed as required, and the results of the literature survey and experimental work were then combined into a constitutive model for heaving clay behaviour.

The need for conducting experimental work limited the scope of the research. Firstly, only the volumetric behaviour of the clay was investigated, and only two soil moisture conditions were studied. No investigation of the change of shear strength with soil moisture was performed. Secondly, because only one type of clay was tested the results of the volumetric behaviour cannot be generalised for other heaving clays. Thus the research work became focused on the volumetric behaviour of Rosebank clay only.

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the Art. *The Civil Engineer in South Africa*, July 1985. pp 367-377 & 407.

CHAPTER 3

REVIEW OF OBSERVED BEHAVIOUR OF EXPANSIVE CLAYS

The phenomenon known as "heave" of clays occurs when certain types of clay in a state of partial saturation are exposed to free water. This results in a volumetric expansion of the clay, and a change in some of its properties. Soils such as sands, silts and even some types of clay do not exhibit this behaviour, which suggests that heave is not a mechanical effect but instead a chemical response of certain clay minerals to free water. This chapter thus begins with a consideration of the mineralogy of heaving clays to give insight into the characteristic changes in volume and shear strength that such clays display. The volumetric and shear behaviour of the clays is discussed in the remainder of the Chapter.

3.1 Influence of Mineralogy on Heave

All clays are composed of three basic types of clay minerals: kaolinite, illite and montmorillonite. These consist of sheets of silica tetrahedrons and alumina octahedrons chemically joined to form a planar lattice structure. In kaolinite these lattices consist of one sheet of silica tetrahedrons and one sheet of alumina octahedrons, while illite and montmorillonite consist of a sheet of alumina octahedrons sandwiched between two sheets of silica tetrahedrons.

These planar lattices are in turn stacked in layers. The surface of each lattice has a negative electrostatic charge due to displacements of hydroxyl ions from the clay crystal. These negative charges cause water molecules, which are dipolar, to become adsorbed onto the surface of the lattice. Positive ions such as sodium and potassium cations are also attracted onto the mineral layer

surface.

Figure 3.1 is a schematic representation of montmorillonite clay lattices stacked in sheets, with adsorbed water molecules and cations present in the interlayer spaces.

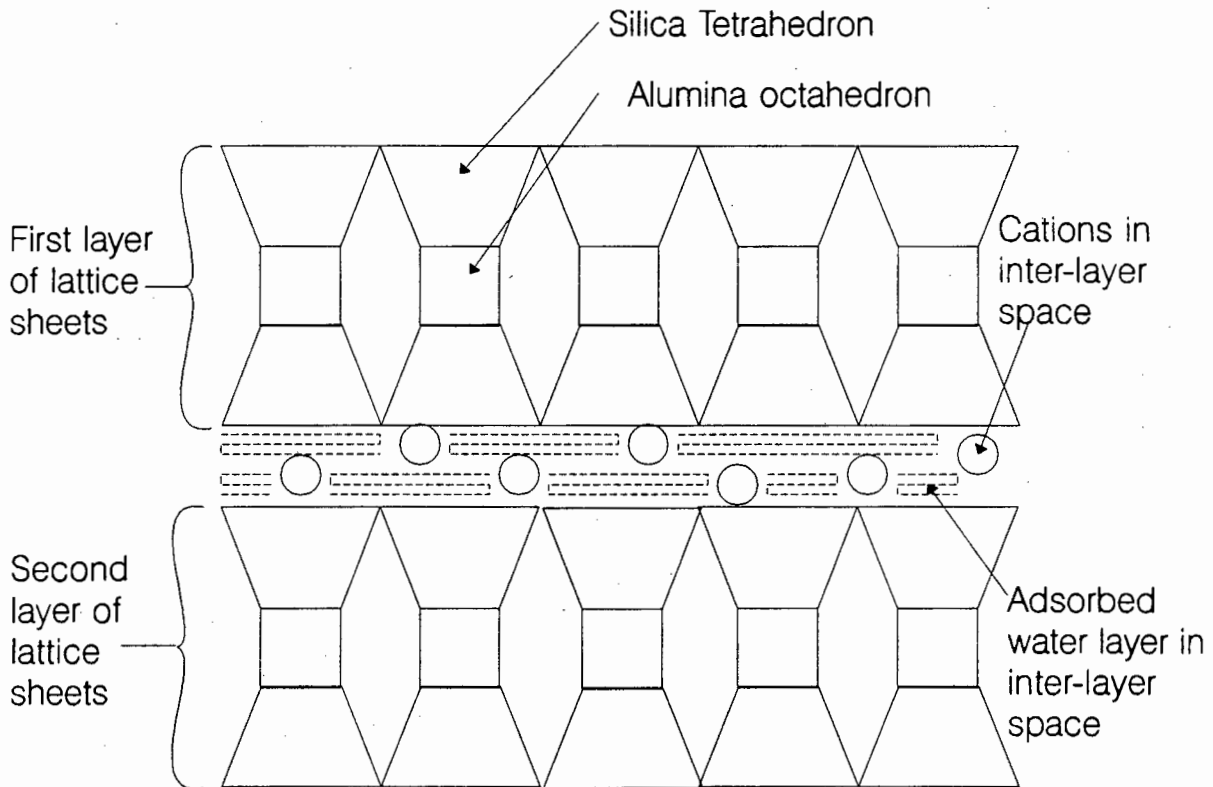


Figure 3.1 Arrangement of mineral layers, water and cations in a montmorillonite clay (after Gromko¹).

A complex equilibrium exists between the positive and negative electrostatic charges of the clay minerals, the dipolar water molecules and the cations in the interlayer spaces. These forces must also be in equilibrium with the external macroscopic conditions such as presence of free water and the external loads. For example, if the clay undergoes drying, adsorbed water is removed from the interlayer spaces and the lattices will move closer together so that electrostatic equilibrium is re-established between the mineral layers, adsorbed water

molecules and cations. The volume of the soil will therefore decrease as the clay dries out.

Heaving of the clay represents the reverse of the situation described above. The clay will be in a partially desiccated state and hence has the ability to adsorb free water. If water becomes available it will move into the interlayer spaces, causing the layers to move apart and the volume of the clay to increase. If the clay is confined in some way then an equilibrium between the externally applied loads and the internal electrostatic forces must be achieved and the clay will exert a heaving pressure against the confining mechanism.

The various clay minerals have different electrostatic charge distributions and different mineral layer thicknesses. This means that different minerals show different affinities for adsorbing free water and cations into the interlayer spaces. The montmorillonite mineral has by far the greatest potential for adsorbing water and hence undergoes the greatest volume changes.

Consideration of clay mineralogy suggests that the following aspects will characterise heaving clay behaviour. Firstly, the clay must exist in a partially saturated state in order to heave, and will only heave when their degree of saturation increases. Experimental and field observations confirm that heave never occurs in deposits of fully saturated clay, and that for partially saturated clays with saturation levels above the shrinkage limit the increase in the volume of the sample is equal to the volume of water added.

Secondly, chemical considerations suggest that heaving clay will exert a "heaving pressure" if it is constrained from expanding freely. This has also been observed experimentally².

Lastly, the clay content of a soil and clay mineral type are dictate whether a soil will heave significantly or not. A soil with a relatively high montmorillonite content

will show significant heave behaviour, while other clay minerals will not heave sufficiently to cause concern.

The rôle of clay mineralogy in heave behaviour is so significant that the potential for expansion of a clay has been closely linked to its Atterberg limits, particularly the plastic limit, plasticity index, clay fraction and clay activity^{1,2}. These observations have formed the basis of several widely used empirical formulae for the prediction of heave^{3,4}. Several models for describing some aspects of heave behaviour have also been based on consideration of mineralogy^{5,6}. This suggests that heave cannot be regarded as a purely mechanical effect like the consolidation or shearing of fully saturated clay, and this must be taken into account when developing models of heave.

3.2 Volumetric Behaviour of Heaving Clay

The discussion of clay mineralogy suggests that heaving clays expand when water is added to them, and that this process can occur even when the clay is loaded by external forces. This behaviour has been extensively studied by many researchers, usually in oedometer tests, and has led to the establishment of the parameters of *percent heave* and *heave pressure* to characterise the volumetric behaviour of expansive clay. This section discusses the definition of these parameters and several different methods that have been proposed for measuring them. Various factors which have been found to influence the parameters are then described.

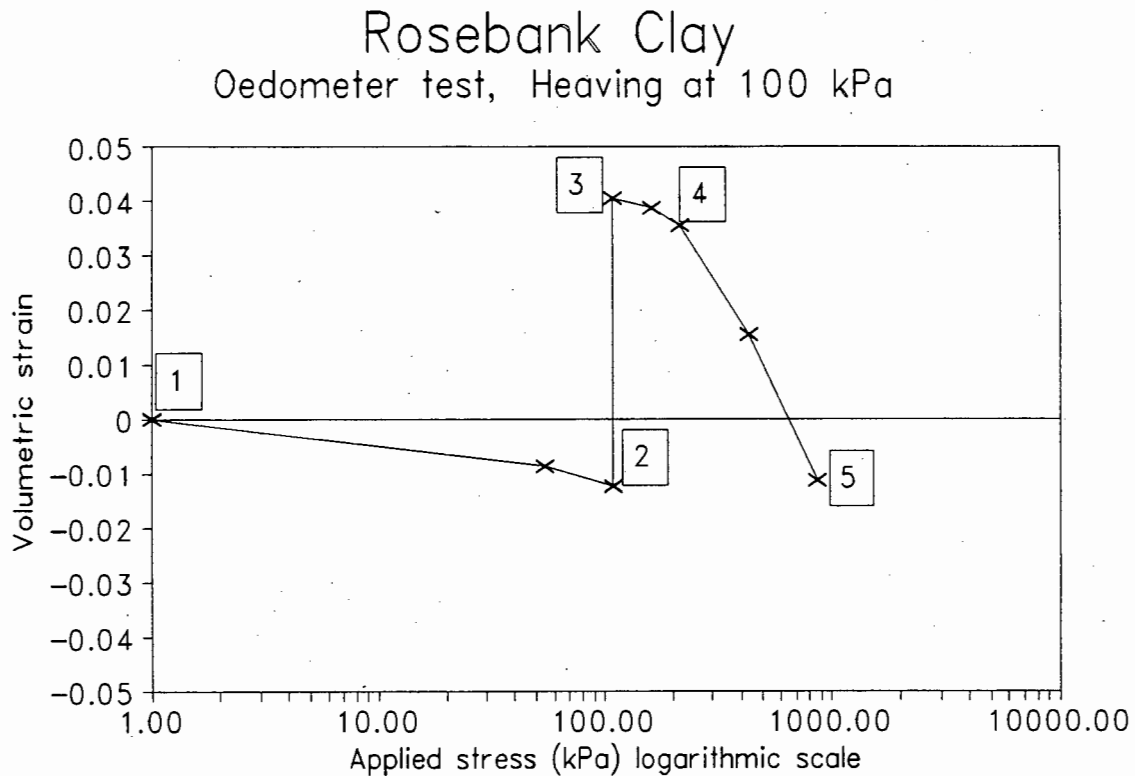


Figure 3.2 Plot of oedometer test on remoulded heaving clay from Rosebank, Cape Province. Test data from de Sousa Vinagre⁷.

3.2.1 Definition of Parameters Characterising Heaving Clay Behaviour.

The characteristic heave parameters are best defined by consideration of a typical oedometer test on an expansive clay. Figure 3.2 shows a plot of the volumetric strain of a sample versus the logarithm of the applied stress, and was generated from test data from de Sousa Vinagre⁷. The sample was loaded at natural moisture content from 0 (point ①) to 100 kPa (②). The oedometer reservoir was then flooded with water, saturating the clay and triggering heave to ③. The load was then increased to 800 kPa at ⑤.

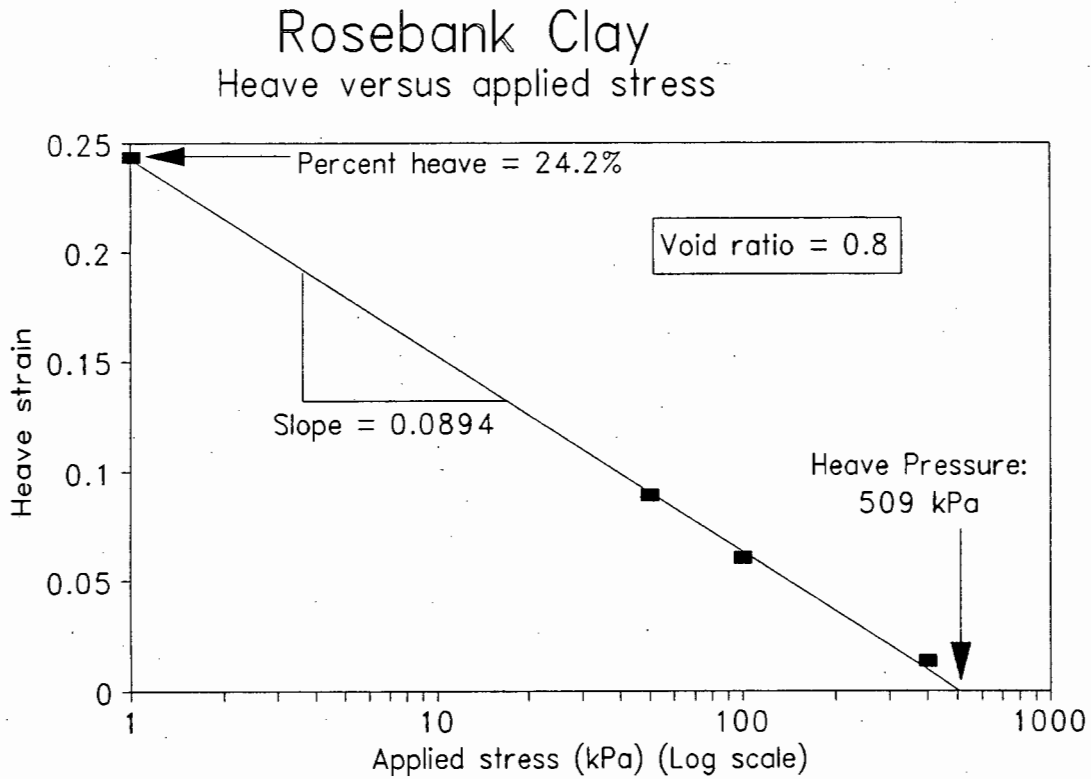


Figure 3.3 Graph of heave strain versus applied pressure at wetting for remoulded Rosebank clay (plotted from test results of de Sousa Vinagre⁷).

If several similar samples are tested and wetted at different applied stresses, then the resulting heave strains can be plotted against the logarithm of the applied stress at heave. Figure 3.3 indicates such a plot for Rosebank clay. The plot indicates that there is a linear relationship between the heave strain and the logarithm of the applied stress at wetting. This is a fundamental characteristic of expansive clay behaviour, and the linear relationship shown for Vinagre’s test data has been confirmed in the work of Chen⁸, Brackley⁹, Pidgeon¹⁰ and others.

Figure 3.3 also illustrates the parameters *heave pressure* and *percent heave* which characterise the volumetric behaviour of heaving clay. The *percent heave* is the heave strain (expressed as a percentage) for a small applied load at wetting. This load is arbitrary and is usually selected

as 1 kPa, because the logarithm of the applied pressure is zero. The *heave pressure* is that value of the applied stress which constrains the heave of the sample to be exactly zero.

Most other researchers refer to these parameters as the "percent swell" and "swelling pressure", but this terminology has been avoided in this thesis in order to distinguish the processes of "heave" (the volumetric expansion of a heaving clay upon wetting at constant load) and "swell" (the elastic rebound of soils when unloading at constant moisture content).

There have been several different test methods proposed by researchers for establishing these parameters, which sometimes yield different results for the same soil. The different test methods are described below in order to give greater insight into the behaviour of heaving clays under loading.

3.2.2 Test Procedures for Establishing Percent Heave and Heave Pressure.

El Sayed and Rabbaa¹¹ review four different methods for establishing the *heave pressure* and *percent heave*. These are:

1) Different Pressures Method

This method is the method described above and illustrated in Figure 3.3. A minimum of two tests are performed on the expansive clay with wetting occurring at different applied pressures. The linear relationship between the heave strain and the logarithm of the applied stress is assumed, and the *percent heave* and *heave pressure* can be read by extrapolating the curve

to the intersections with the axes.

2) Heave-Consolidation Method

The sample is wet at a low applied pressure as for establishing the percent heave. It is then consolidated by increasing the applied load until the sample reaches the same void ratio as before wetting (ie the overall strain is zero). The applied pressure at which this occurs is considered to be the *heave pressure*.

3) Double Oedometer Method

This method was first proposed by Jennings and Knight¹². Two samples of heaving clay are tested simultaneously in adjacent oedometers. The samples are loaded to a small nominal load (as for establishing the *percent heave* of the sample) and one is then saturated by flooding the oedometer reservoir to cause heave. The other is maintained at its initial moisture content throughout the test. After heaving of the first sample is complete both samples are loaded with additional load increments until the e - $\log p$ relationship for each sample shows that the normal consolidation is taking place.

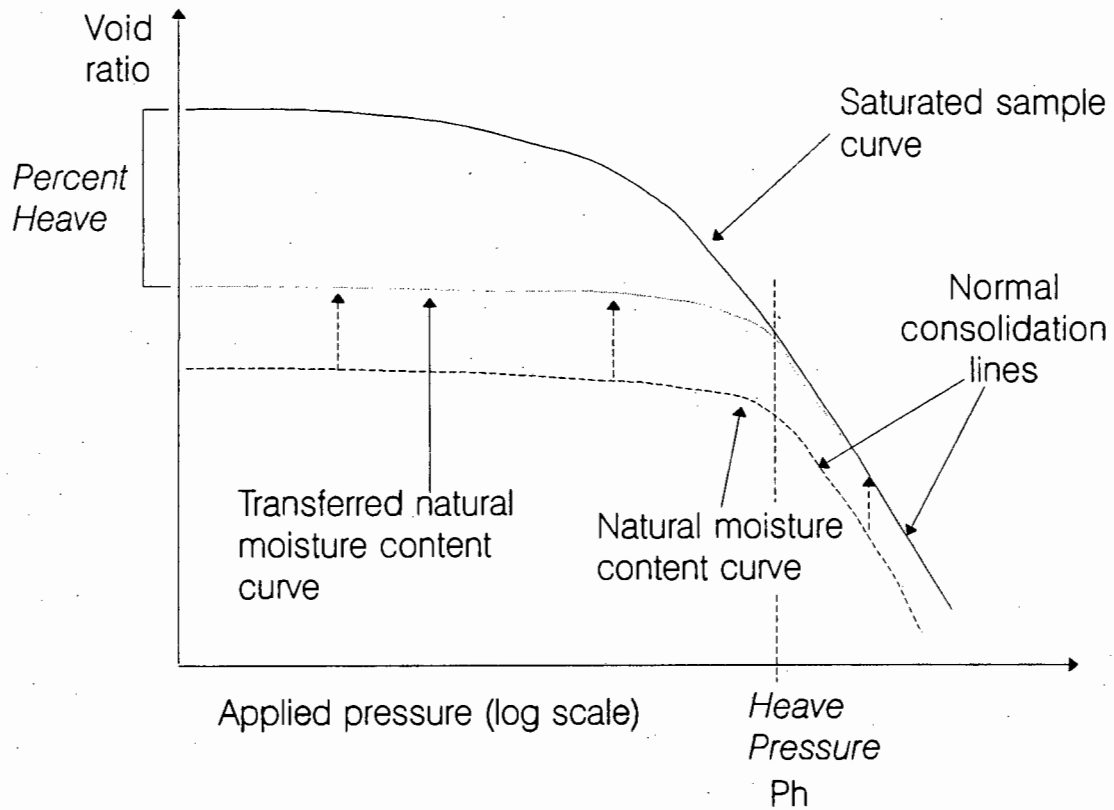


Figure 3.4 *e-logp* relationship for samples in a double oedometer test (After Jennings and Knight¹²)

Figure 3.4 shows a schematic representation of a double oedometer test. The *percent heave* and the *heave pressure* can be established from the graph as indicated. The heave of the clay at any applied stress up to the *heave pressure* will be the strain difference between the curves.

An important aspect of heaving clay behaviour noted by Jennings and Knight¹² and upon which the double oedometer test is based is that both the fully saturated sample and the sample at natural moisture content will eventually reach the same normal consolidation line. This makes it possible to relate the two oedometer tests to one another by shifting their *e-logp* curves until

their normal consolidation lines coincide, as shown in Figure 3.4.

4) Constant Volume Method

The constant volume method is used for establishing the *heave pressure*^{2,13}. A sample of heaving clay is placed in an oedometer at natural moisture content and is then wetted. As the sample heaves a normal stress is applied to confine the sample to zero volume increase. The ultimate value of the heave pressure takes some time to develop fully and thus the sample must be continually monitored and the confining stress adjusted in order to keep the sample volume constant.

3.2.3 Influence of Test Stress Path on the Value of Heave Parameters

Several researchers have noted that these four test methods yield different values of the *percent heave* and *heave pressure* for the same soil^{9,11,13}. This is illustrated by Brackley⁹ who presents the different stress paths of the different test methods on a single plot.

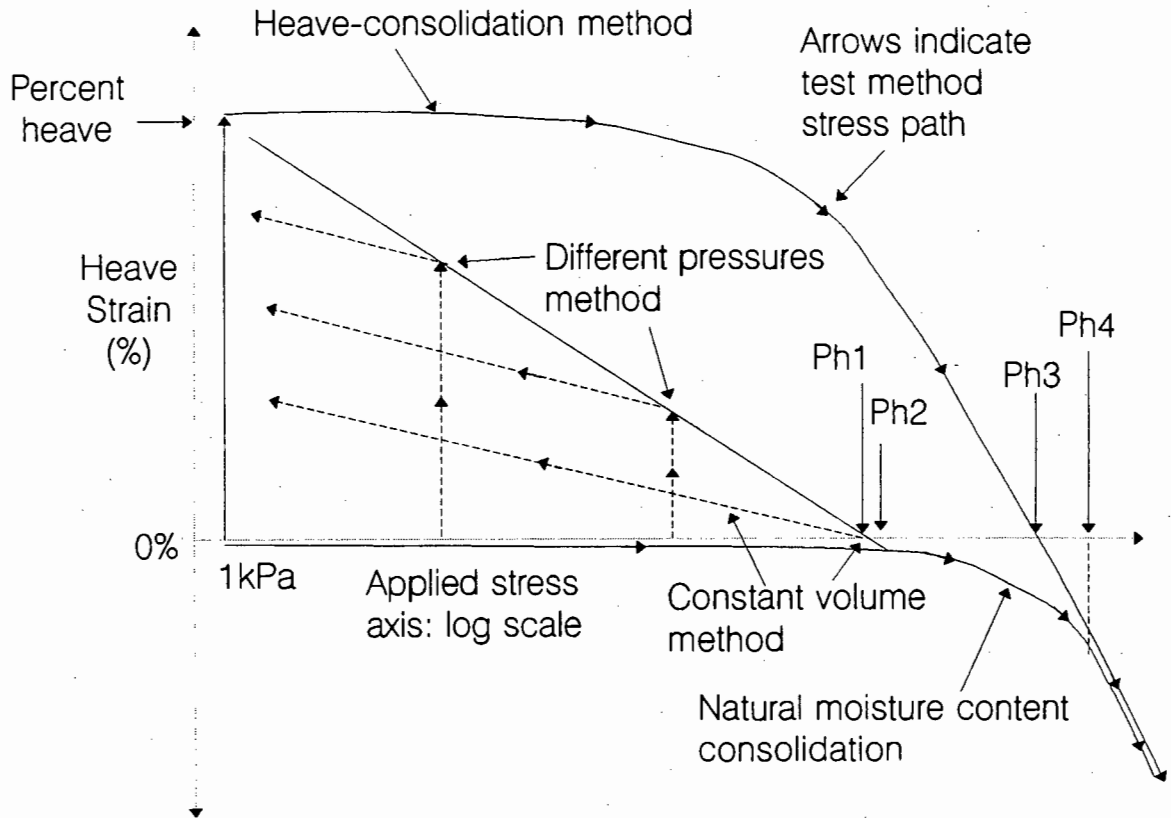


Figure 3.5 Schematic diagram of the test stress paths of different tests for establishing the heave parameters (after Brackley⁷).

Figure 3.5 shows that there may be a significant difference between the value of the *heave pressure* indicated by the different pressures method (p_{h2} in the figure) and the heave-consolidation method (p_{h3}). El-Sohby and Mazen¹³ performed both the heave-consolidation test and the different pressures test on several clays with different clay mineral compositions. The heave-consolidation method established a *heave pressure* that was between 10% and 100% higher than the value observed in the different pressures method. They concluded that the difference was related to the type of clay mineral present in the soil. The lowest difference in the values was where potassium montmorillonite-vermiculite was the dominant clay mineral, and the highest where sodium montmorillonite was the dominant mineral.

The stress path to which a heaving clay is subjected is therefore important. The explanation for the observed difference in values is that the two test methods measure two different types of behaviour. The different pressures method incorporates only the volumetric behaviour of heaving clay as it expands due to the chemical interaction between the clay minerals and free water while under load. The heave-consolidation method incorporates the heave behaviour during wetting at a nominal load and then the mechanical consolidation behaviour of the fully saturated clay.

The four test methods described can be grouped into two categories. The first category, which incorporates only heave behaviour, includes the different pressures and constant volume methods. Although these tests involve the same aspect of expansive clay behaviour the value of the *heave pressure* yielded by each test may still be different. This is because in the different pressures method the sample is first compacted to a higher density by consolidation under the applied load before water is added (compare p_{h1} and p_{h2} on Figure 3.5).

The second category of tests, incorporating heave and mechanical consolidation, includes the double oedometer and heave-consolidation test. The exact values of the *heave pressure* yielded by the two tests may also differ because in the heave consolidation test the reference volume is the initial volume of the sample, while in the double oedometer test the reference volume is the volume of a sample at natural moisture content subjected to the same applied stress (compare p_{h3} and p_{h4} on Figure 3.5).

The volume of a heaving clay is significantly effected by the loading-wetting procedure imposed on it. It is important that the mechanical behaviour of clay under changes in loading and constant saturation be

distinguished from the mechanical-chemical behaviour of clay under changes in saturation. For the purposes of this research work the different pressures method from the first test category was chosen for evaluating the heave parameters because it measures only heave effects and does not include mechanical consolidation effects.

3.2.4 Factors which Influence Heave Parameters

Once a consistent method of testing the *percent heave* and *heave pressure* parameters has been established other factors which effect the values of these parameters for a particular heaving clay can be investigated. Two major factors have been found to influence the degree of expansion of heaving clay. These are soil moisture conditions, and soil density and placement conditions.

1) Soil Moisture Conditions

Heaving occurs when a partially saturated potentially expansive clay is exposed to free water. The degree of heave will be dependent on the magnitude of the saturation changes. If two samples at the same initial moisture content are wetted to different final saturations then the sample with the highest final saturation will expand more and show a greater *percent heave*. If two samples are wetted to full saturation from different initial moisture contents then the sample with the lower initial moisture content will heave more. In both cases the sample that undergoes the greatest saturation change shows the greatest heave.

Proper description of the influence of changes in soil moisture conditions on the heave of the sample is considered with

reference to the soil suction, u_c , which is chosen because it represents a more definitive measure of soil desiccation and potential expansiveness than the moisture content^{4,14}. A detailed discussion about the choice of this variable, rather than the saturation is given in Chapter 4.

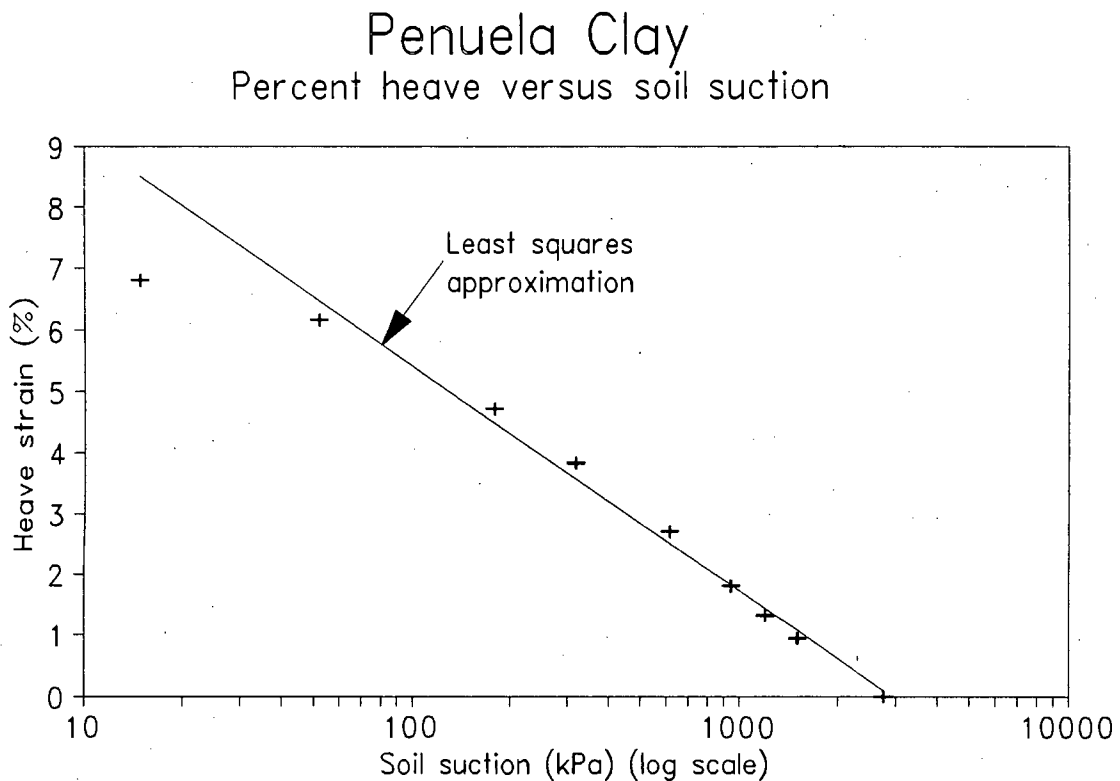


Figure 3.6 Percent heave versus logarithm of soil suction (data from Escario¹⁵)

Testing of heaving clay under conditions of controlled soil suction has not been widely reported. However, relationships have been established for the effect of suction on *percent heave* and *heave pressure*. Figure 3.6 shows a relationship between the *percent heave* and the soil suction. The results were taken from Escario¹⁵, but have been replotted with the soil suction drawn to a logarithmic scale. Most of the points on the graph lie very close

to a straight line, with the divergence from a linear relationship occurring as the soil suction approaches zero. Escario and Sáez¹⁶ also performed tests for clay samples under various external applied pressures and measured the heave strain at different values of the soil suction. The results are presented in Figure 3.7, which has also been replotted to a semi-logarithmic scale. Pellissier and Maree⁶ also report a linear relationship between heave strain and the logarithm of the soil suction.

Figure 3.7 shows linear relationships between the heave strain and the logarithm of the soil suction for samples subject to applied loads of 50, 100, 250 and 400 kPa. Lines representing a different value of the applied stress have different gradients, because the

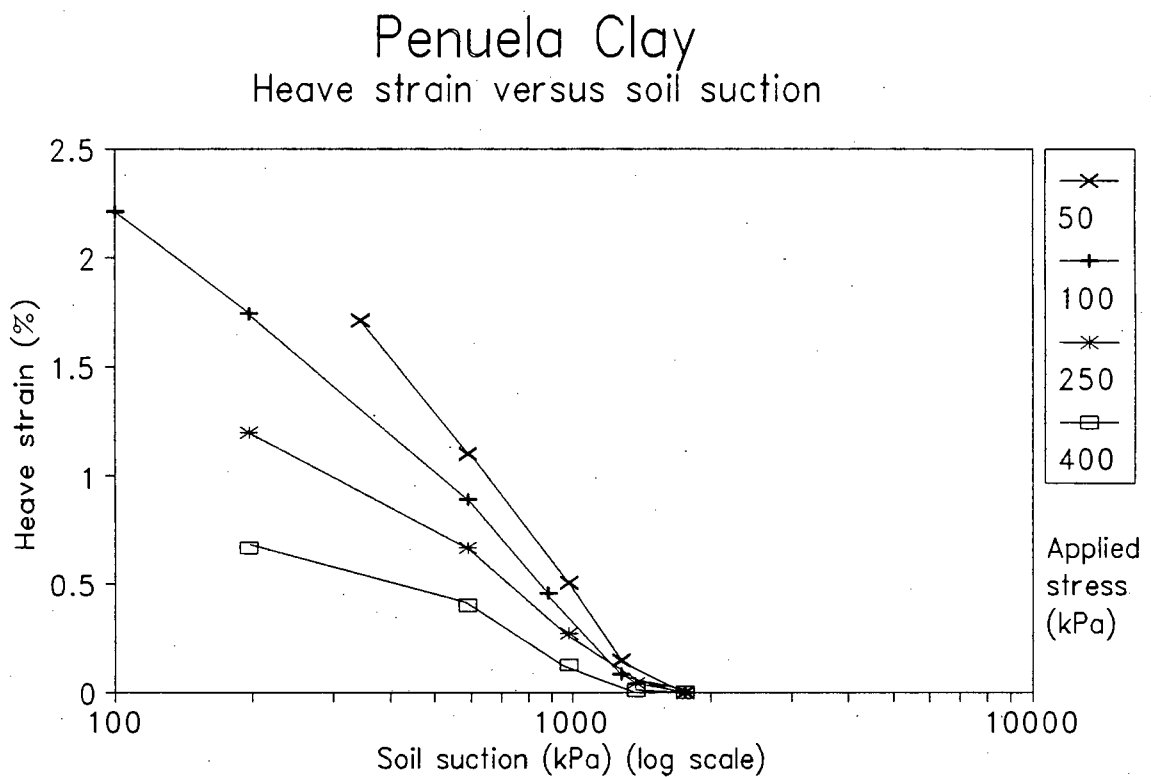


Figure 3.7 Heave strain versus logarithm of soil suction for heaving clays subjected to different applied stresses (data from Escario and Sáez¹⁶).

heave strain decreases in proportion to the logarithm of the applied stress (see also Figure 3.3).

Escario¹⁵ tested the relationship between heave pressure and suction, using the constant volume method discussed in §?. Figure 3.8 shows Escario's data replotted to a log-log scale. Initially a decrease in suction (increase in soil moisture) leads to a very rapid increase in the heave pressure that develops. The heave pressure reaches its maximum value after a small reduction in suction and then remains constant. This relationship is confirmed by Kassiff and Ben Shalom¹⁷, who also report a very sharp rise in the heave pressure to its maximum value during early stages in wetting.

For the purpose of simplifying the relationship between soil suction and heave pressure a bilinear approximation to the curve was made, and is shown superimposed on the figure.

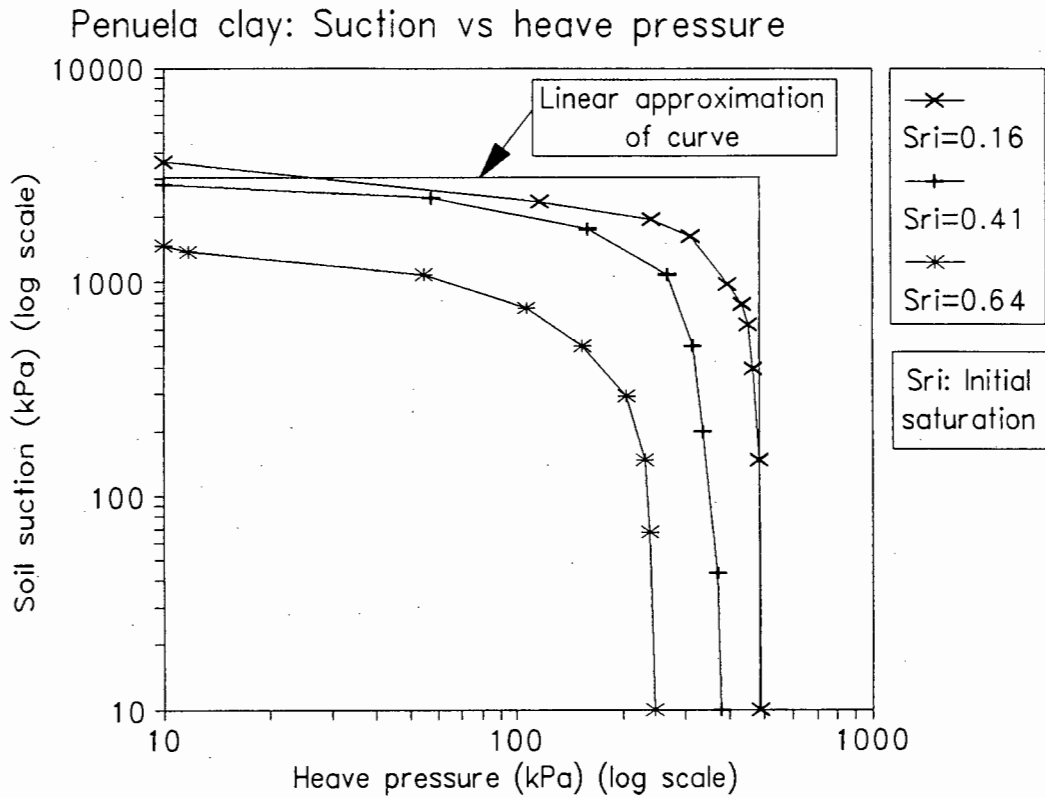


Figure 3.8 Relationship between heave pressure and logarithm of soil suction (after Escario¹⁵)

Relationships between heave strain and soil suction, heave strain and applied stress and heave pressure and soil suction. These can be represented graphically on a single three dimensional plot with the dependent axes being the heave strain and the independent axes being the applied stress and soil suction. Straight line relationships exist when the applied stress and soil suction axes are drawn to a logarithmic scale. Figure 3.9 shows the heave strain surface in a schematic three dimensional

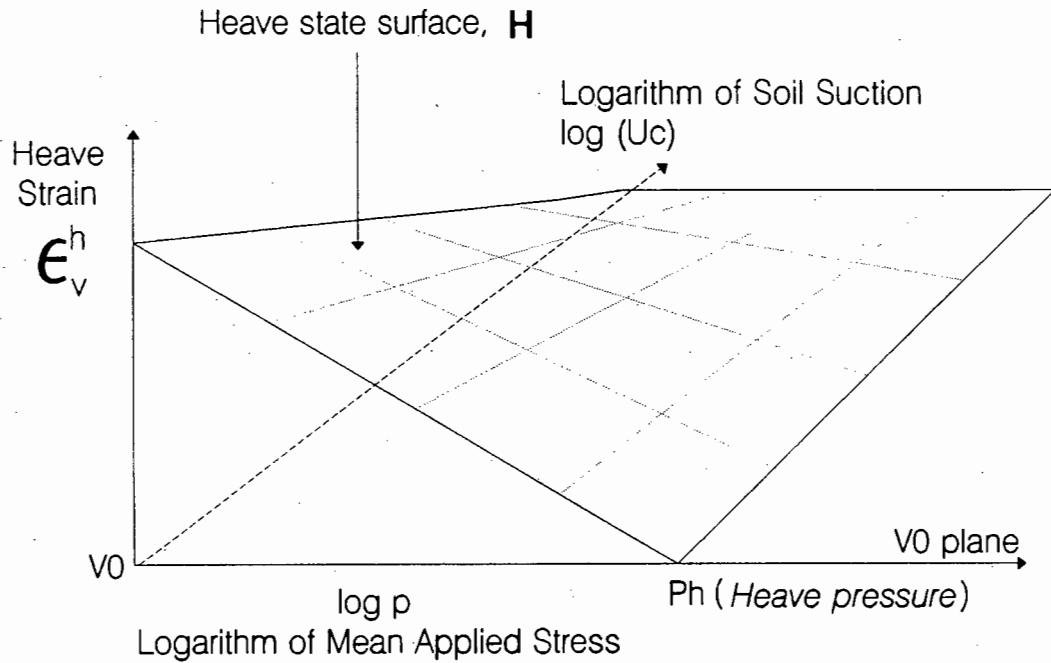


Figure 3.9 Heave strain surface plotted as a function of the logarithm of the applied stress and logarithm of the soil suction

diagram. The heave pressure is represented by the symbol p_h .

2) Dry Density and Placement Conditions

A heaving clay sample with a higher dry density will exhibit a greater *percent heave* and *heave pressure* than a sample with a lower density. The dry density is inversely related to the void ratio by the formula $\rho_d = \rho_s / (1 + e)$, where ρ_d , ρ_s are the dry and solids densities respectively and e is the void ratio. A higher void ratio implies a lower the dry density and a lower the *percent heave* and *heave pressure*. Figure 3.10 shows the relationship between initial void ratio e_0 and the resulting heave strain plotted from test data

Rosebank Clay Samples Heave versus void ratio

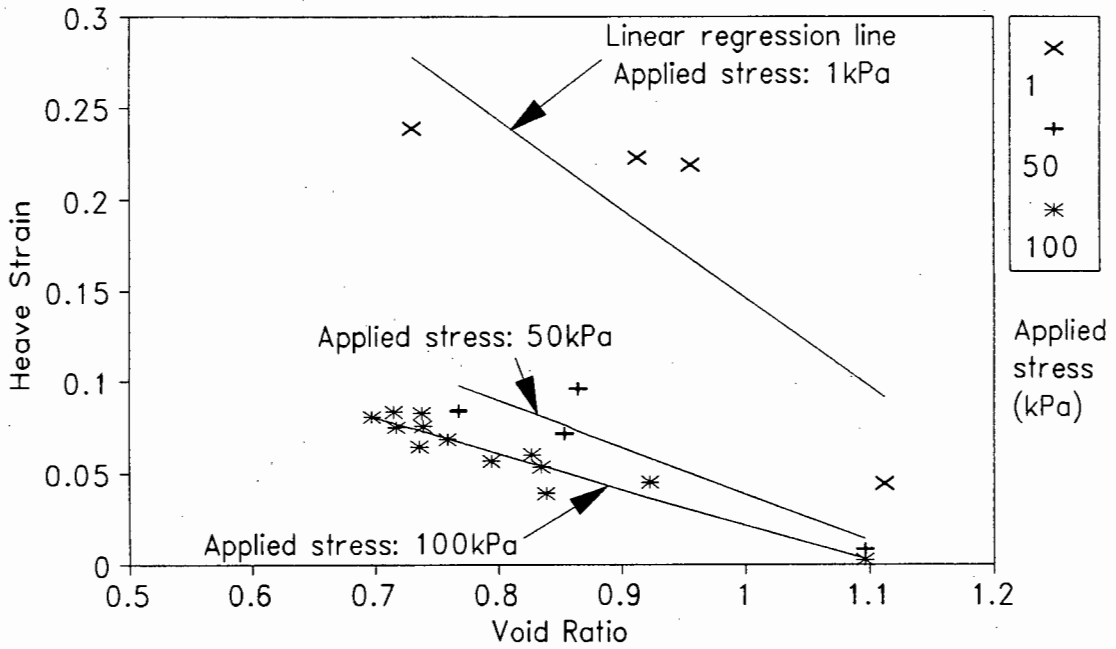


Figure 3.10 Relationship between heave strain and initial void ratio for remoulded Rosebank clay samples (test data from de Sousa Vinagre⁷)

from de Sousa Vinagre⁷. The data points shown are for heave under different applied loads, which must be considered when analysing the data. A sample wetted at a higher applied load will show less heave than another sample with the same dry density but a lower applied load. There is a high scatter in the data points, but the plot shows that the heave strain increases as the void ratio decreases.

The greatest number of data points in Figure 3.10 is for heaving at 100 kPa, and they suggest a linear relationship between heave strain and void ratio. The best fit line for each set of data points is plotted. The value the gradient of the heave strain/void ratio line will be different for each applied stress value. Figure 3.11 shows

the gradient of each line plotted against the logarithm of the applied stress associated it. The points on the figure indicate a possible linear relationship between the applied pressure and the gradient of the heave strain/void ratio line. It can be shown analytically that the relationship between these variables will be linear. The deviation from the line in this case can be attributed to a poor estimate of the 400 kPa gradient as a result of too few data points.

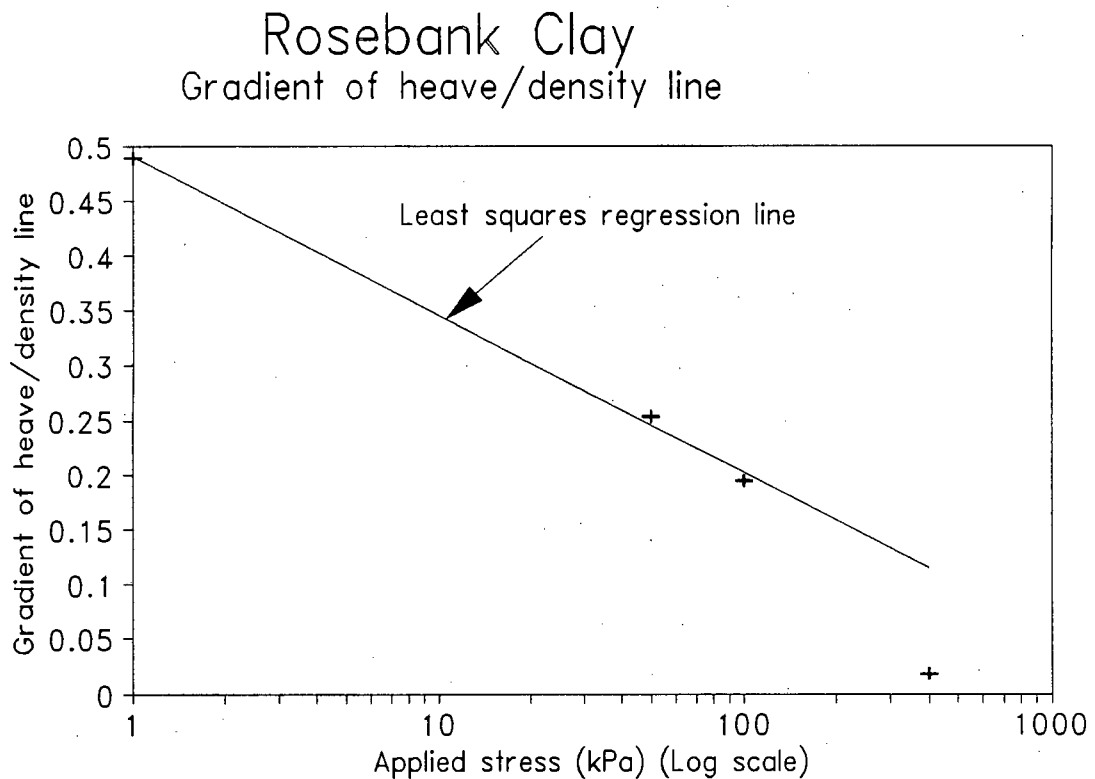


Figure 3.11 Gradient of heave strain-void ratio line plotted against logarithm of applied stress (test data from de Sousa Vinagre⁷)

The relationship between dry density, *percent heave* and *heave pressure* has been noted by other researchers⁸. It has further been noted that significant differences in heave parameters are

found for undisturbed, remoulded and compacted samples. A *percent heave* of 24% was observed for remoulded samples of Rosebank clay⁷, while experiments conducted on undisturbed samples of the same material as part of this research work gave a *percent heave* of 5.7%. If compacted samples are prepared then the degree of compaction, moisture content at compaction and method of compaction all influence the heave parameters of the compacted clay^{2,18}.

3.2.5 Lateral Expansion of Heaving Clay

Heaving clays consist of layered sheets of clay minerals. Depending on placement conditions (undisturbed or compacted) and method of deposition, the clay mineral layers may become aligned in a particular orientation which means that lateral heave parameters will be different to vertical parameters. Fourie¹⁹ measured lateral *heave pressures* in a triaxial apparatus and found them to be approximately twice the vertical *heave pressures* measured in an oedometer test. Brackley and Sanders²⁰ performed *in-situ* measurements of horizontal stresses due to heave and found them to be between two and four times the overburden stresses.

3.3 Shear Resistance of Heaving Clay

Free water moving into a heaving clay causes a volumetric expansion but also changes the shear resistance of the clay. Holtz and Gibbs² report reductions of unconfined compressive strength in an Arizona heaving clay from 1070 kPa to virtually zero. Stark and Duncan²¹ tested clay from the San Luis Dam (California) and found a reduction of the cohesion and friction angle of from 263 kPa and

39° to 0 kPa and 25° respectively upon wetting. Losses in shear resistance can thus be severe and must be considered when developing a constitutive model for heaving clay.

Escario²² and Escario and Saez²³ present the results of shear box tests performed under controlled suction on several different types of potentially expansive clays from Spain. The data has been used to establish the shear strength parameters of cohesion, c , and friction angle ϕ . These are plotted against the soil suction in Figure 3.12 and 30.

Shear resistance of unsaturated clay Cohesion versus Soil Suction

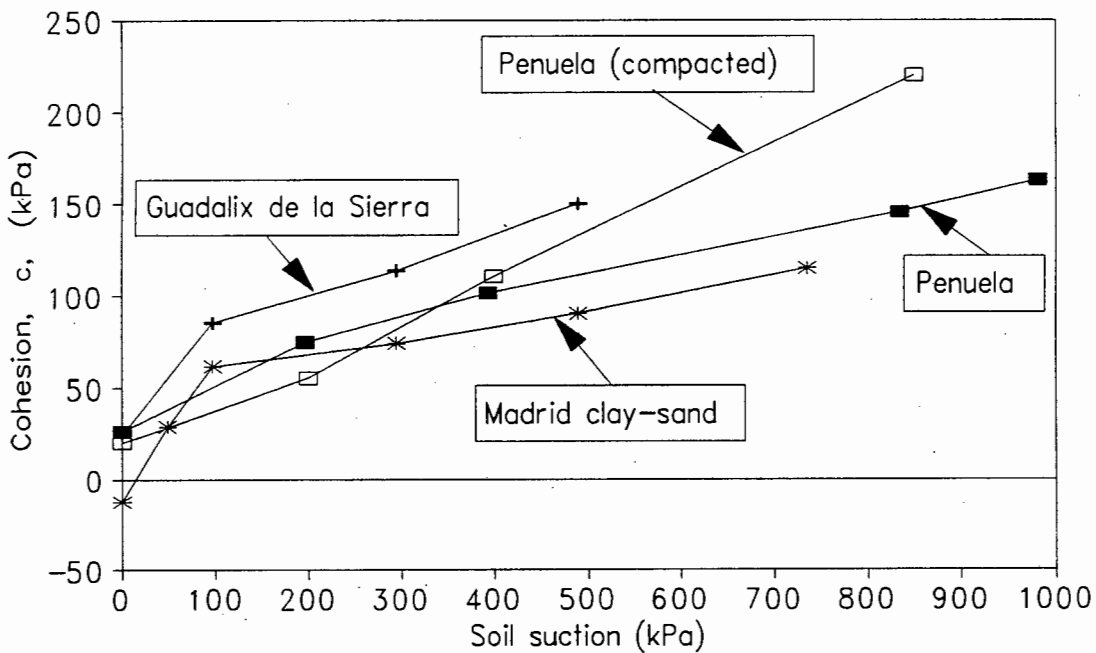


Figure 3.12 Relationship between soil cohesion c and soil suction u_c (Data taken from Escario²² and Escario and Sáez²³).

Figure 3.12 shows that there is a decrease in the cohesion of the clay as the soil suction decreases (ie as the soil moisture increases). It appears that for suctions above 100 kPa the relationship between cohesion and suction is

approximately linear. As the suction decreases from 100 to 0 kPa the decrease cohesion is more rapid, but there are insufficient data points to establish an exact relationship. Thus approximating the graph by a single line will lead to inaccuracies in shear strength prediction, and a bilinear approximation would be better.

Shear resistance of unsaturated clay Friction angle versus Soil Suction

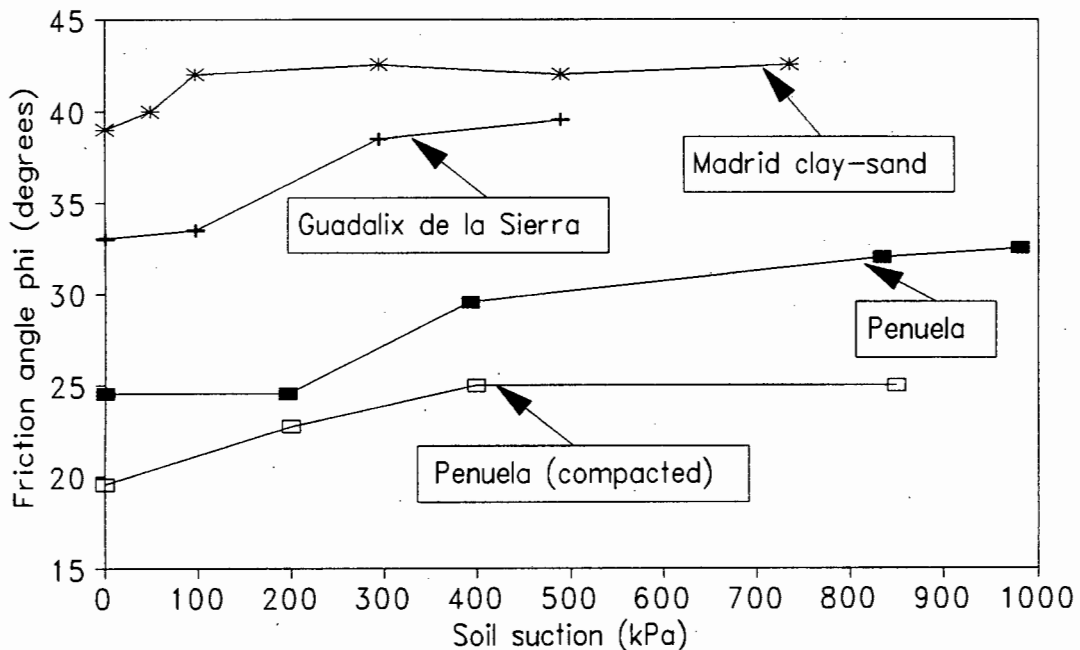


Figure 3.13 Relationship between friction angle ϕ and soil suction u_c (Data from Escario²² and Escario and Sáez²³).

Figure 3.13 indicates the relationship between the angle of friction ϕ and the soil suction. The general trend is that ϕ decreases as the suction decreases (soil becomes more saturated). However, the figure shows that the variation in friction angle for these particular clays is relatively small, hence a linear approximation of the relationship between ϕ and u_c would be reasonably accurate. A constant value approximation of ϕ at its minimum observed value of ϕ could be used in an analysis, giving conservative results.

Other researchers (such as Toll²⁴) have studied the shear strength of partially saturated soils and also concluded that the shear strength of soils decreases as the degree of saturation increases. However, no generalised relationship between shear strength and suction has been developed, and it appears that the precise nature of the relationship varies from soil to soil. The soil type appears to have an important influence on the relationship. Wheeler²⁵, for example, developed a cubic polynomial expression for the shear strength of partially saturated gravel as a function of the soil suction, which is different to trends for clays presented in the figures above.

In this Chapter some fundamental aspects of the volumetric and shear behaviour of heaving clay has been reviewed. The clay minerals present in a soil determine whether it will exhibit significant heave behaviour. It is the chemical behaviour of the clay minerals present in heaving clays that explains the volumetric expansion that these soils display when wet with free water.

The volumetric behaviour of expansive soils can be characterised by the parameters *heave pressure* and *percent heave*. When measuring these parameters it is important to consider the dependence of the volume of the clay upon the stress path, and to measure the parameters in a consistent way.

Both the volumetric and shear behaviour of expansive clays is related to the soil moisture conditions of the soil, and heave is triggered by changes in the moisture content of the soil. In order to implement a constitutive model for heaving clay it is thus necessary to develop a measure for quantifying soil moisture conditions, and a framework for incorporating this measure into the constitutive equations. This process is discussed in detail in the following Chapter.

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CHAPTER 4

REVIEW OF METHODS FOR INCORPORATING SOIL MOISTURE CONDITIONS IN THE CONSTITUTIVE RELATIONS FOR HEAVING CLAYS

The material state of fully saturated soils has been successfully described by the principle of effective stress proposed by Terzaghi¹ in 1936. The effective stress combines the stress in the soil due to applied loads and the pore water pressure into a single parameter. The volume and shear strength can be expressed as of the effective stress alone. A similar method for combining applied stress and soil moisture conditions in partially saturated soils is required so that changes in volume and shear strength can be evaluated.

Initially it seemed logical to extend the principle of effective stress to describe the state of partially saturated soils as well. This was not completely successful, however, and researchers investigated other methods of describing the material state of the soil. This led to the state variable approach which incorporates the applied stress and soil suction as independent variables which effect the volume and shear strength of the soil.

This Chapter begins by describing the principle of effective stress for partially saturated soils in order to introduce the parameters which effect the material state of such soils. The problems associated with the use of the effective stress are then discussed in order to motivate the need for a different approach for describing the material state. This leads into a description of the state variable approach and demonstrates how this can be applied to describe heaving clays. The Chapter concludes with a discussion of the relationship between moisture content, saturation and soil suction.

4.1 Principle of Effective Stress for Partially Saturated Soils

Terzaghi¹ defined the effective stress σ' of a fully saturated soil as the excess of the total applied stress σ over the pore pressure u . This is shown as equation (4.1):

$$\sigma' = \sigma - u \quad (4.1)$$

The material state variables for fully saturated soil such as void ratio and shear strength are exclusively related to the effective stress. If the effective stress changes then the void ratio and shear resistance will also change, and these changes are the same irrespective of whether the effective stress changed because of a change in applied stress or in pore pressure.

In partially saturated soils there is a negative pore pressure present, indicating that water has been "sucked out" of the soil causing it to be partially desiccated. As water is drawn out of the soil air enters the void spaces. If a horizontal plane through the soil is considered, the negative pore pressure will not act over the full area of the plane but only over a fraction of it. In 1960 Aitchison and Bishop² proposed that the effective stress for partially saturated soils be defined as the total applied stress less some fraction of the pore pressure. Both the applied stress and pore pressure were referenced to the pore air pressure as a datum. Hence the proposed expression is:

$$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) \quad (4.2)$$

where:

- σ is the total applied stress,
- u_a is the pore air pressure,
- u_w is the pore water pressure, and
- χ is the fraction of the soil suction which contributes to the effective stress.

The value of χ varies from 0 for a fully desiccated soil to 1 for a fully saturated soil. In particular, when the soil is saturated and $\chi = 1$ then equation (4.2) reduces to equation (4.1).

Equation (4.2) also contains a term $(u_a - u_w)$ which is referred to as the *soil suction*, denoted by u_c . The soil suction will be positive for negative pore water pressures, so a high soil suction implies a high negative pore water pressure and highly desiccated conditions.

The principle of effective stress has the potential to model the expansive behaviour of heaving clays because these have high initial soil suctions due to their desiccated condition. Equation (4.2) implies that there will be a high compressive stress in the soil initially. When water is added the suction eventually reduces to zero and the suction induced compressive stresses will disappear. This will cause the soil to expand - unless sufficient external load is applied to the sample to constrain the volume increase to zero.

The most important term in the effective stress equation is the parameter χ . This will vary from 0 to 1 with the saturation of the soil. However, research by Bishop and Blight³ and Blight⁴ suggested that χ is not a constant related to the saturation but a function of the stress path experienced by the soil that is influenced by the hysteresis effects of cyclic wetting and drying and loading and unloading. Bishop and Blight³ concluded that the effective stress path could not be considered on its own, but the separate paths of the two components ($\sigma - u_a$) and $(u_a - u_w)$ had to be considered as well.

Jennings and Burland² showed that the principle of effective stress would predict a volume increase when water is added to partially saturated soils. However, in collapsing sands adding water causes a decrease in volume, and thus the principle of effective stress is incapable of predicting the behaviour of such soils. Jennings and Burland concluded that there are limitations to the

usefulness of the principle of effective stress.

An effective stress formulation is desirable firstly in its ability to express the volume and shear strength in terms of a single effective stress value, and secondly in the fact that geotechnical engineers are familiar with the concept of effective stress. However, it has become apparent that for partially saturated soils it is not possible to use a single effective stress value only. The total stress ($\sigma - u_a$) and suction ($u_a - u_w$) paths must be considered as well. It is also difficult to establish values for χ because it is stress path dependent and varies widely from soil to soil, and anomalous values are frequently measured^{2,5}. This means that the effective stress value of partially saturated soils is heavily influenced by the complex nature of the function χ which makes changes in volume and shear strength due to changes in effective stress difficult to comprehend. This offsets the advantage of geotechnical engineers' familiarity with effective stress values.

The principle is also difficult to implement in a constitutive model. Its most severe disadvantage is the difficulty of establishing the precise nature of the function χ , which must be done experimentally for each individual soil and which requires testing which is either impractical or impossible. The principle has a major intrinsic weakness in that it is based purely on mechanical stress considerations and provides no way for taking into account the fact that heave is a chemical rather than a mechanical phenomenon.

4.2 State Variable Model

The work performed by researchers while studying the principle of effective stress for partially saturated soils highlighted some weaknesses in the principle shown in equation (4.2). However, the variables of *applied stress*, ($\sigma - u_a$) and soil suction ($u_a - u_w$) were identified as parameters effecting the material state

ratio e_0 .

The surface **H** is only valid for monotonic decreases in the soil suction because cycles of wetting and drying of the soil cause hysteresis effects which cause the state point to move off the state surface defined by **H**. Also, as stated in §3.2, **H** describes only heave movements under load and not straining under the mechanical process of consolidation under loading or unloading while the soil suction is zero.

The state variable approach as presented by Matyas and Radhakrishna⁵ has been illustrated by developing the state surface **H** for the void ratio of a heaving clay in terms its state variables. It is possible to consider any path of changes in applied stress and soil suction by reference to the surface **H**. The surface has the soil suction and applied stress as independent (perpendicular) axes. The plot of the surface presented in Figure 4.1 also assists in visualising the heave surface directly, without confusion due to the complex and often poorly defined parameter χ . This demonstrates the advantages of the state variable approach over the principle of effective stress in considering different test paths and clarity of visualisation of volume changes. The disadvantage compared to the effective stress approach is that geotechnical engineers are more familiar with the concept of effective stress.

A more significant disadvantage is that the formulation presented by Matyas and Radhakrishna⁵ does not describe states where reference to the previous stress or suction history is required. This means that any cyclic strains or suctions cannot be described because they introduce non-uniqueness into the surface. It is desirable that the concept be extended by the incorporation of further state variables such as a yield stress and suction hysteresis parameters which include the effect of stress and suction history on the soil volume and incorporate non-uniqueness in the state surface.

4.3 Relation between Moisture Content, Saturation and Soil Suction

In §3.2.4 it was stated that soil suction, u_c , represents a more definitive measure of soil desiccation and potential expansiveness than the moisture content (O'Neill & Poormoayed⁶, Brackley⁷). It is now possible to clarify this statement and discuss the relationship between moisture content, soil suction and saturation by consideration of the state variable concept.

The moisture content of a soil merely gives the ratio of the mass of water in a soil to the mass of dry soil solids. It gives no indication of whether the soil has the potential to absorb more water or not. It is not possible to tell from the moisture content alone what the saturation a heaving clay is and whether it has the potential to take up more water expand.

The soil saturation S_r is the fraction of the void spaces which are filled with pore water. If the saturation of a soil is less than 1 it means that the void spaces are not completely filled then the soil can potentially absorb more water and expand. The saturation is a more useful parameter than the moisture content because it will indicate whether a soil is partly desiccated or not. However, the saturation value gives no indication of the amount of energy required to lower the saturation from 1 to its reduced value. Lowering the saturation of a clay it will require more energy than reducing the saturation of a sand or gravel to the same value. A further aspect of the saturation is that it is dependent on the void ratio of the soil, and therefore on the applied stress. An increase the stress will decrease the void spaces and thus the saturation will increase (assuming that no water moves out of the soil during compression).

The soil suction is related to the negative pore water pressure in the soil. As such it is a dependent on the suction boundary conditions of the soil sample such as the desiccating action of the sun or plants. It also gives representation

of the "suction pressure" required to bring the soil to its current state of desiccation. It is thus a more useful parameter than the saturation.

There is, however, a relationship between the soil suction and saturation which varies from soil to soil. A typical plot of the relationship (at zero applied stress) can be found in Jennings and Burland². The state variable method can be used to express the relationship. The state variables relating to the soil saturation conditions are the applied stress ($\sigma - u_a$), soil suction u_c , initial void ratio e_0 and initial saturation S_{r0} . The state function is thus

$$S_r = \phi(\sigma - u_a, u_c, e_0, S_{r0}) \quad (4.6)$$

The function ϕ can be plotted in the same way as the other state functions in a $(S_r, \sigma - u_a, u_c)$ space (see Matyas and Radhakrishna⁵).

Two possible methods of incorporating the soil moisture conditions into the material constitutive relations have been discussed in this Chapter. The first extended the principle of effective stress to partially saturated soils. While this method was found to have some shortcomings it successfully identified the soil suction u_c as a state variable for expressing the moisture conditions of a soil. The second method was the state variable approach which uses the same state parameters as the effective stress but in such a way that they remain independent and the paths followed by each variable can be considered separately. In this Chapter it was shown that the state variable approach can be used to describe the volume of a heaving clay undergoing expansion. The soil suction rather than saturation and moisture content was chosen as the variable for describing the soil moisture state because it provides the most definitive measure of the state of desiccation of the soil and hence of its potential expansiveness if the soil is a heaving clay. However, the saturation, moisture content and soil suction are related to one another and can be described by a state surface using the state variable approach.

of a partially saturated soil. The principle of effective stress incorporated both of these parameters into a single stress function, but Bishop and Blight³ concluded that the behaviour of partially saturated soil was dependent on the individual paths followed by both parameters. Consequently a formulation was needed which considered changes in these parameters separately from one another. This was the motivation for the state variable approach proposed by Matyas and Radhakrishna⁵.

The state variable approach involves the identification of *state parameters* of a soil, and then the development of *state functions* which express the relationship between the different parameters. The state variables of a soil are defined as "the physical variants of the soil which are sufficient to completely describe the state of soil" (Matyas and Radhakrishna⁵). In §3.2 the various factors which influence the heave strain of a heaving clay are presented. The void ratio e of the clay will increase as the clay heaves, and thus the change in void ratio is related to the heave strain by the formula

$$\epsilon_v^h = \frac{\Delta e}{(1 + e_0)} \quad (4.3)$$

where ϵ_v^h is the volumetric heave strain,
 Δe is the change in void ratio due to heave, and
 e_0 is the initial void ratio.

It was shown that the final heave strain, and hence the final void ratio was dependent on the initial and final moisture conditions, the initial dry density of the sample (which is a function of the initial void ratio), and the external stress applied to the clay. Hence the various state parameters, or state variables, relating to the volume of heaving clay under changing moisture conditions are:

e the final void ratio;
 $(\rho - u_a)$ the external applied stress;
 $(u_a - u_w)$ the final soil suction, u_c ;

e_0 the initial void ratio; and
 u_{c0} the initial soil suction.
 p is the mean all round pressure. $p = 1/3(\sigma_{11} + \sigma_{22} + \sigma_{33}) = 1/3 I_{1,\sigma}$. The symbol $I_{1,\sigma}$ is the first invariant of stress.

Now that the state variables have been defined it is necessary to define a state surface which relates the state parameters uniquely to one another. This will take the form

$$e = \mathbf{H}(p-u_a, u_c, e_0, u_{c0}) \quad (4.4)$$

The function \mathbf{H} defines a unique surface and can hence be plotted in three dimensional space. This will in fact give the heave surface that has already been presented in figure Figure 3.9. The figure is re-plotted here as Figure 4.1.

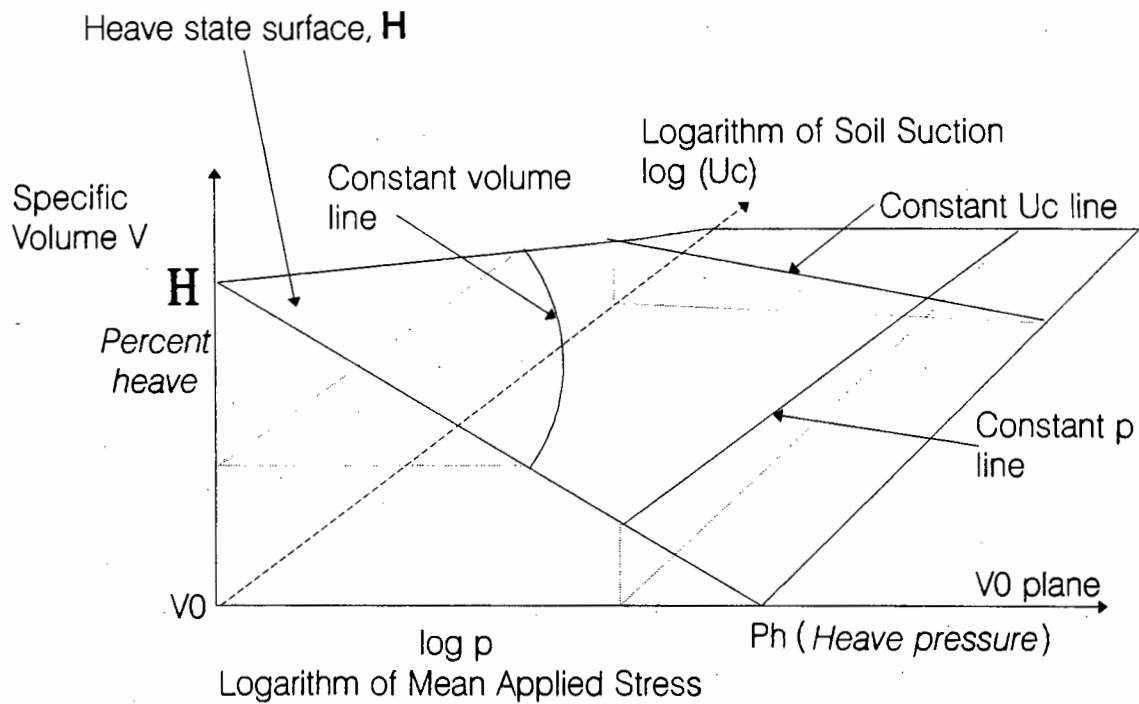


Figure 4.1 State surface function H for the expansion of a heaving clay under changes in soil moisture conditions

Figure 4.1 shows the uniqueness of the state surface H , and indicates how the state of a soil moves about on the surface for different stress, suction and void ratio paths. It is also possible to develop an expression for H based on the representation in the figure. The function is

$$e = H(1 + e_0) \left(1 - \frac{\log u_c}{\log u_{c0}} \right) \left(1 - \frac{\log(p - u_d)}{\log(p_h - u_d)} \right) + e_0 \tag{4.5}$$

where H is the *percent heave*, expressed as a strain value rather than as a percentage, and p_h is the *heave pressure*.

Equation (4.5) is not quite the full expression for the final void ratio because the *percent heave*, H and the *heave pressure*, p_h are also functions of the initial void

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CHAPTER 5

LABORATORY TESTING OF ROSEBANK CLAY

The expansion of heaving clay under wetting has been investigated in some detail by several researchers. However, the focus of most investigations has been the heave expansion, and little attention has been given to the mechanical consolidation behaviour of such clays before and after wetting. In particular there has been minimal research into the response of heaving clay to cyclic loading and unloading, and the effect of cyclic loading on the magnitude of heave. A constitutive model for heaving clay should be as general as possible, which requires an understanding of the volume changes of heaving clays under different stress paths and cyclic loading. The lack of information in the literature meant that a laboratory investigation into these aspects of heaving clay behaviour was necessary.

A potentially expansive clay from Rosebank in Cape Town, South Africa, was selected for the experimental programme because of the availability of test data for the clay from the work of de Sousa Vinagre¹. The samples were taken from the same location as the samples tested by Vinagre in order to ensure consistency in type of material being tested. In its *in situ* condition the clay has a very low potential for expansion because of its high water content. The samples were collected during the rainy season and were fully saturated with a moisture content of approximately 40%. However, after drying to a moisture content of about 10 and 15% the clay heaves when wetted.

Vinagre tested remoulded samples of the clay, but his results showed a high variability because of the difficulty of remoulding samples to the same dry density. For this reason testing was performed on undisturbed samples to try and minimise the variation in test results due to different sample preparation conditions.

Tests were performed in four oedometer testing devices in the geotechnical section of

the Department of Civil Engineering at the University of Cape Town. Oedometers are one dimensional testing devices which measure the vertical strain of a horizontally constrained soil sample under the application of vertical loads. Oedometer testing was chosen because the test procedure and apparatus are relatively simple (in comparison with triaxial testing, for example), and oedometer tests have been the basis of most other research work done on heaving clay.

Six series of tests were performed in order to compare the behaviour of the clay under different loading and unloading paths, with wetting of the sample occurring at some point during the loading cycle to trigger heave. Each series of tests was implemented on all four machines simultaneously, so that each series consists four tests.

This Chapter contains a description of the test programme, including details of the soil tested, a discussion of the test procedure and the different test loading paths followed. Finally, the test results are presented and discussed.

5.1 Characteristics of Rosebank Clay

As part of his research programme de Sousa Vinagre¹ performed a wide range of indicator tests, X-ray diffraction analyses and other tests on the clay. A summary of some of these results is presented in Table 5.1, to show some of the fundamental engineering properties of the clay.

Table 5.1 Fundamental engineering indices and mineralogy of Rosebank clay (after de Sousa Vinagre¹).

General Description	
Fine silty clay, with a small amount of fine sand. Light red colour	
General Engineering Indices	
Liquid Limit (LL)	78.9%
Plastic Limit (PL)	36.9%
Plasticity Index (PI)	42%
Linear Shrinkage	12.2%
Particle Size Distribution	
Clay	59%
Silt	29%
Sand	12%
Clay Minerals Present	
Kaolinite, Muscovite, Montmorillonite	

The properties shown in the table indicate that the soil has a high potential for expansion. This is suggested by the high clay fraction and plasticity index, and by the presence of the montmorillonite clay mineral which belongs to the group of clay minerals which exhibit heave.

5.2 Test Procedure Followed while Using the Oedometer Devices

The basic procedure for oedometer testing was adopted from **BS 1377: Part 5: (1990)**². This describes the procedures for sample preparation, loading of the machines, recording of observations and so forth. A detailed testing procedure

is also given by de Sousa Vinagre¹.

The major differences between the procedure laid out in **BS 1377: Part 5: (1990)**² and the procedure followed in these tests related to the control of soil moisture. In the standard testing procedure the oedometer reservoir in surrounding the soil sample is filled with water from the start of the test, so it is conducted under fully saturated conditions. This, however, would trigger the heaving of the sample at the start of the test, which was undesirable. The samples were thus loaded into the oedometers in a dry condition and the top of the reservoir was sealed by plastic sheeting and elastic bands to keep the humidity in the reservoir constant and prevent changes in the soil moisture content. At the point during the test path when saturation of the sample was required a hole was made in the plastic covering and the reservoir was filled with distilled water. This water moved through the porous stones above and below the sample and was absorbed by the clay, triggering heave. After wetting the integrity of the plastic seal no longer mattered and the sample was maintained at full saturation by keeping the water in the oedometer reservoir topped up.

It was important to ensure that the process of heaving was fully complete before applying the next load increment. This was done by measuring the height of the sample at various time intervals after the water was added. The height was plotted against the square root of the elapsed time to check when the rate of change of height had reduced to a negligible value. A period of 24 hours was found to be sufficient for completion of heaving.

The clay in its *in situ* condition was fully saturated and would not normally heave, so the samples were dried out before. Consideration of the results of de Sousa Vinagre¹ suggested that the soil would be below its shrinkage limit for a moisture content of 17% or less, and therefore the exact value of the moisture content of this order would not influence the final expansion of the clay. The

samples were dried in a drying oven at room temperature, with the oven fan rather than heat promoting drying.

The preceding discussion indicates some of the assumptions about the soil moisture content that were made in the absence of apparatus for accurately measuring and controlling the moisture conditions during the test. It was assumed that as long as the samples were initially drier than their shrinkage limit moisture content the test results would not be influenced by slight variations in initial moisture contents between different samples. It was also assumed that the samples would be fully saturated after by flooding the reservoir with free water and allowing 24 hours for completion of the heave process.

5.3 Description of the Different Test Series

This section describes the loading and wetting path followed in each of the six different test series, and shows a plot of the results for one of the four tests from each series. The complete set of test data and plots of all the test results can be found in Appendix I. The results have been plotted as a graph of the volumetric strain of the sample versus the logarithm of the applied vertical load. The semi-logarithmic scale has been used because linear relationships can be found between the strain and the logarithm of the applied stress.

It must be emphasised that the stress values shown in the plots refer to the vertical loads applied to the test device, and are thus total stress values rather than effective stress values as defined in equation (4.2). It is only after the soil has become fully saturated that the total and effective stress values become equal (refer to equation (4.2)).

5.3.1 Test Series 1

The soil is loaded to an applied stress of 100 kPa and then water is added to saturate the soil and trigger heave. The load on the sample is then increased to 1700 kPa before unloading to zero and removal from the testing device.

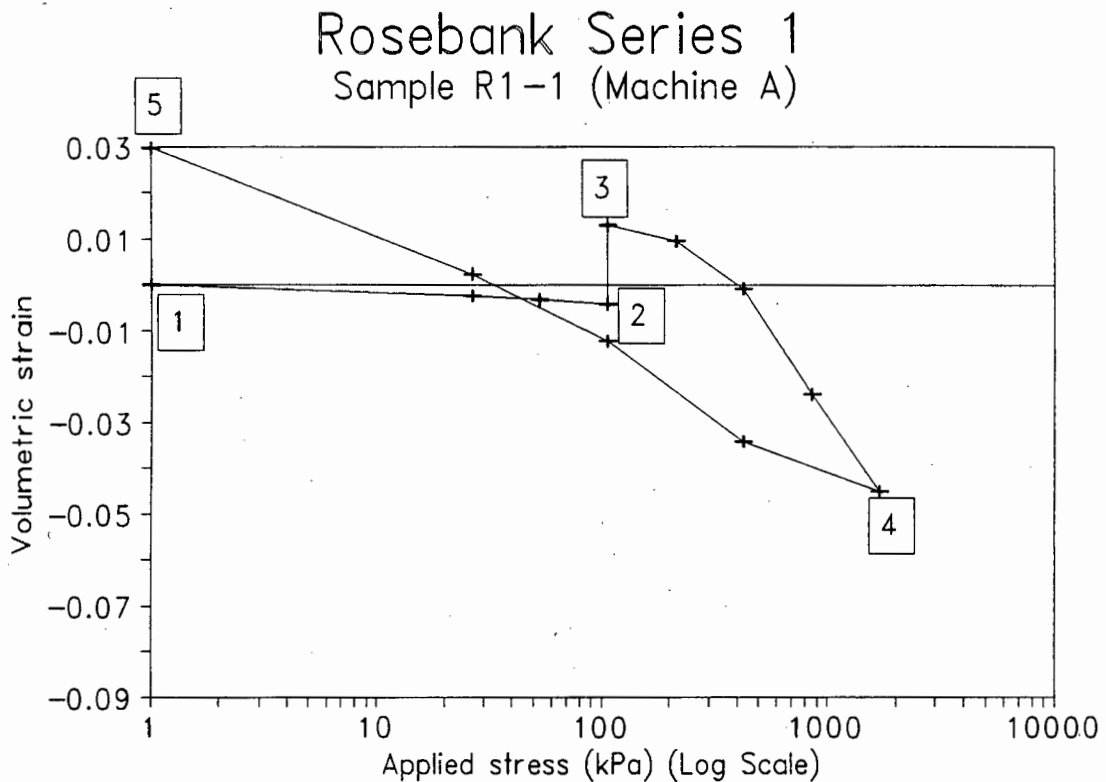


Figure 5.1 Volumetric strain versus logarithm of applied stress for a test from series 1

Figure 5.1 shows a typical test result for the series. The test begins at zero load at the point marked ① and is loaded to 100 kPa (point ②). The addition of water causes heaving to point ③. Further loading causes the sample volume to decrease until point ④, after which the sample swells upon unloading to point ⑤.

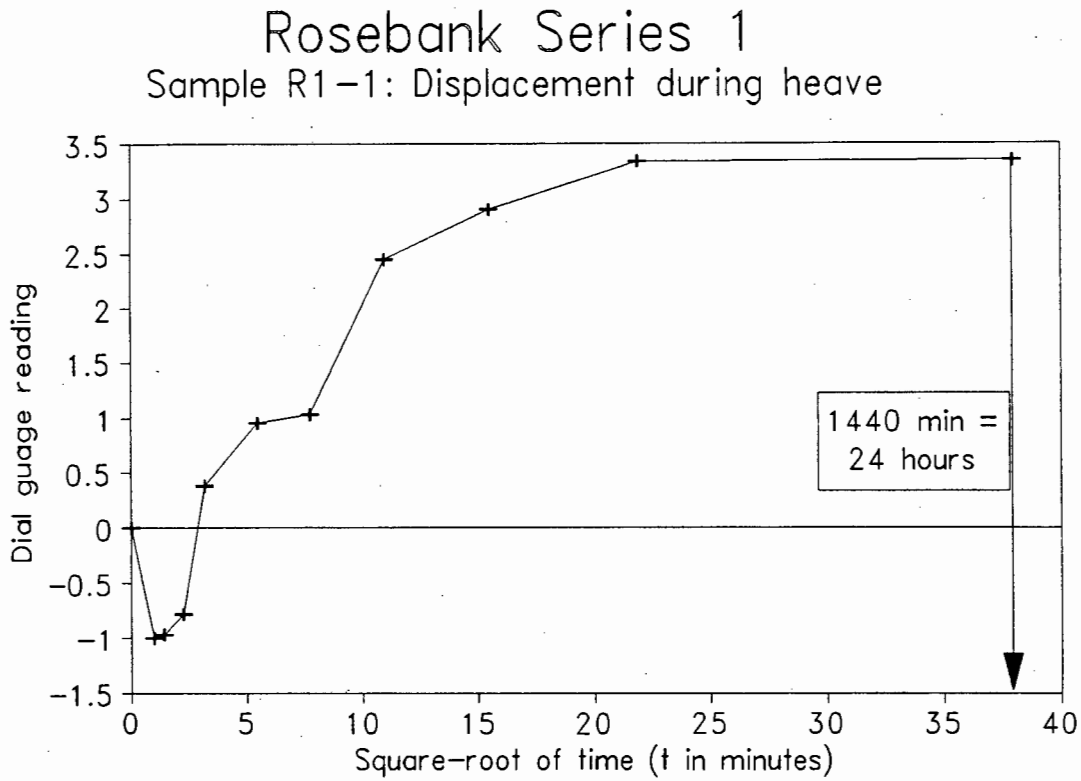


Figure 5.2 \sqrt{t} fitting of changes in sample height during heave

Figure 5.2 shows a plot of the change in sample height during heaving versus the square root of the time (in minutes) since water was added. The final value on the graph represents 24 hours, after which the shape of the curve is almost flat and hence the rate of change of volume is very low. The next load increment can therefore be added to the sample after 24 hours.

5.3.2 Test Series 2

This series investigated the effect of load cycles on the soil displacement. A typical test result is plotted in Figure 5.3.

Rosebank Series 2 Sample R2-2 (Machine B)

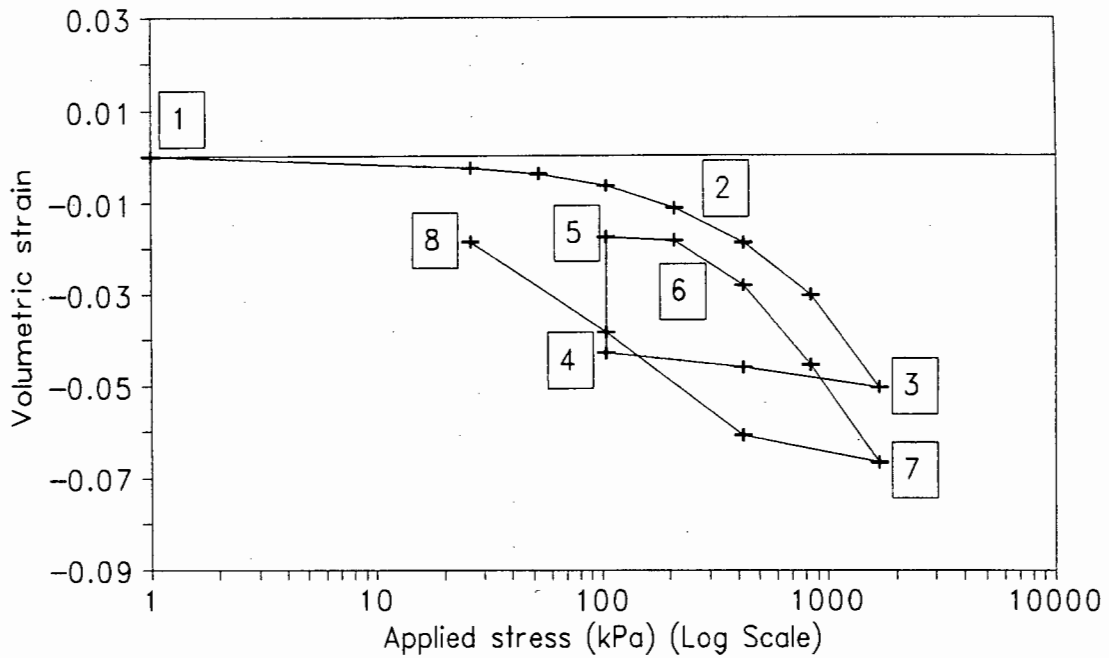


Figure 5.3 Volumetric strain versus logarithm of applied stress plot for a test from series 2

The test begins at a zero load and strain at point ①. The sample is then loaded incrementally to 1700 kPa at point ③. During this loading sequence the material response changes from a relatively flat curve to a steeper curve at point ②. It seems that in the vicinity of ② the material yields and the response changes from elastic to elastic-plastic. This is confirmed by consideration of ③→④, along which the sample swells under unloading along a line roughly parallel to ①→②. The slope of line ③→④ therefore represents the elastic bulk modulus of the clay.

At ④ the clay is wetted by flooding the oedometer reservoir, which triggers heaving to ⑤. The sample is then reloaded along ⑤→⑥→⑦ to 1700 kPa. Again there is a change in the material behaviour from an elastic to an elastic-plastic response at point ⑥. From ⑥ to ⑦ the slope of the curve

is roughly parallel to ②→③, and thus the elastic-plastic modulus for the partially saturated sample is approximately the same as for the fully saturated sample.

From ⑦ to ⑧ the sample is unloaded and swells. This is an elastic unloading line similar to ③→④, but the slope is greater, implying that the value of the elastic bulk modulus for the partially saturated soil differs from its value in the fully saturated soil.

5.3.3 Test Series 3

Series 3 is similar to test series 2, with two fundamental differences. Firstly, the soil samples were cut from the soil block in a lateral direction rather than a vertical direction as for all the other tests. Secondly, during loading along path ①→②→③ the sample is loaded to 800 kPa rather than 1700 kPa.

A typical test result is shown in Figure 5.4. The material response follows the same pattern as the results for series 2. One significant difference is that the yield point ② is at a lower stress value for the lateral samples (series 3) than the vertical samples (series 2). This can be attributed to the different horizontal and vertical stresses that are experienced by the clay in the field.

Rosebank Series 3 Sample R3-1 (Machine A)

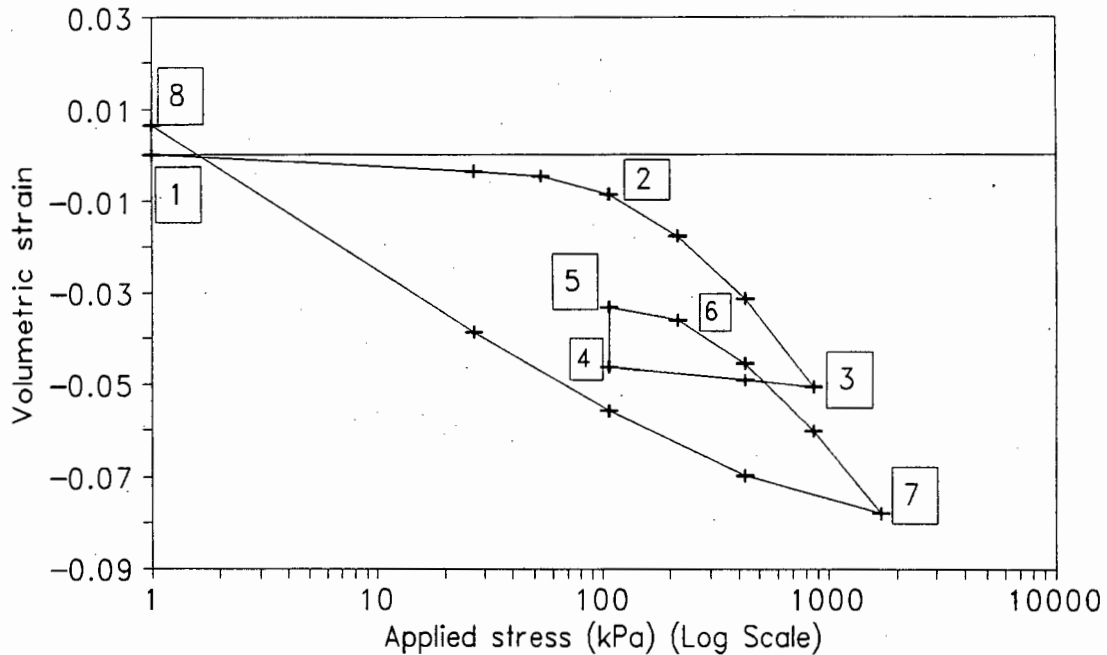


Figure 5.4 Volumetric strain versus logarithm of applied stress for a test from series 3

5.3.4 Test Series 4

These tests examined the response of the clay when wetting occurred under an applied load of 25 kPa as opposed to 100 kPa for the first three series. It also investigates the effects of cyclic loading on a sample after wetting. A typical result is shown in Figure 5.5.

Rosebank Series 4 Sample R4-2 (Machine B)

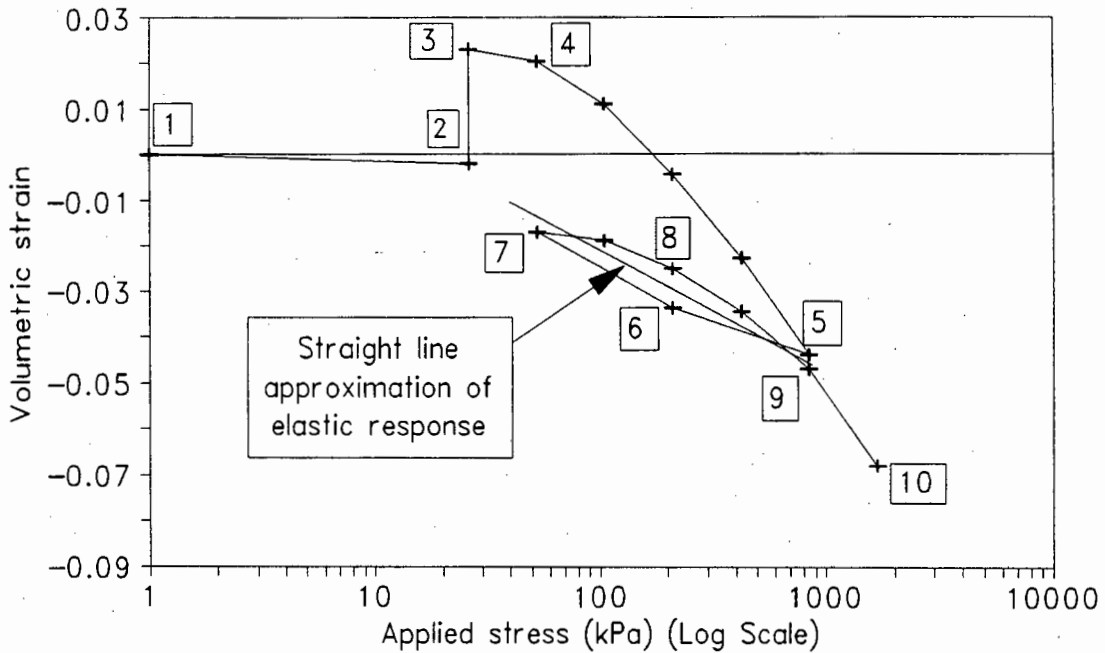


Figure 5.5 Volumetric strain versus logarithm of applied stress for a test from series 4

From point ① to ② the sample is loaded to 25 kPa, after which it is wetted and heaves to ③. The load is then increased to 800 kPa along ③→④→⑤. The sample yields at point ④ and shows an elastic-plastic response to ⑤. At ⑤ the sample is unloaded to 50 kPa and swells along path ⑤→⑥→⑦. Reloading begins at ⑦ and follows path ⑦→⑧→⑨→⑩. The test was terminated at ⑩.

The effect of cycling the load is that the sample undergoes a hysteresis during the cycle ⑤→⑥→⑦→⑧→⑨. However, the points are so close to the same straight line that all the points can be considered to be part of the same elastic response. At ⑨ material returns to the elastic-plastic deformation line ④→⑤→⑨→⑩.

5.3.5 Test Series 6

The series is discussed here because it is related to series 4, examining the response of the material to wetting at 25 kPa after the material has been exposed to a load cycle and plastic deformation. The test path is shown in Figure 5.6.

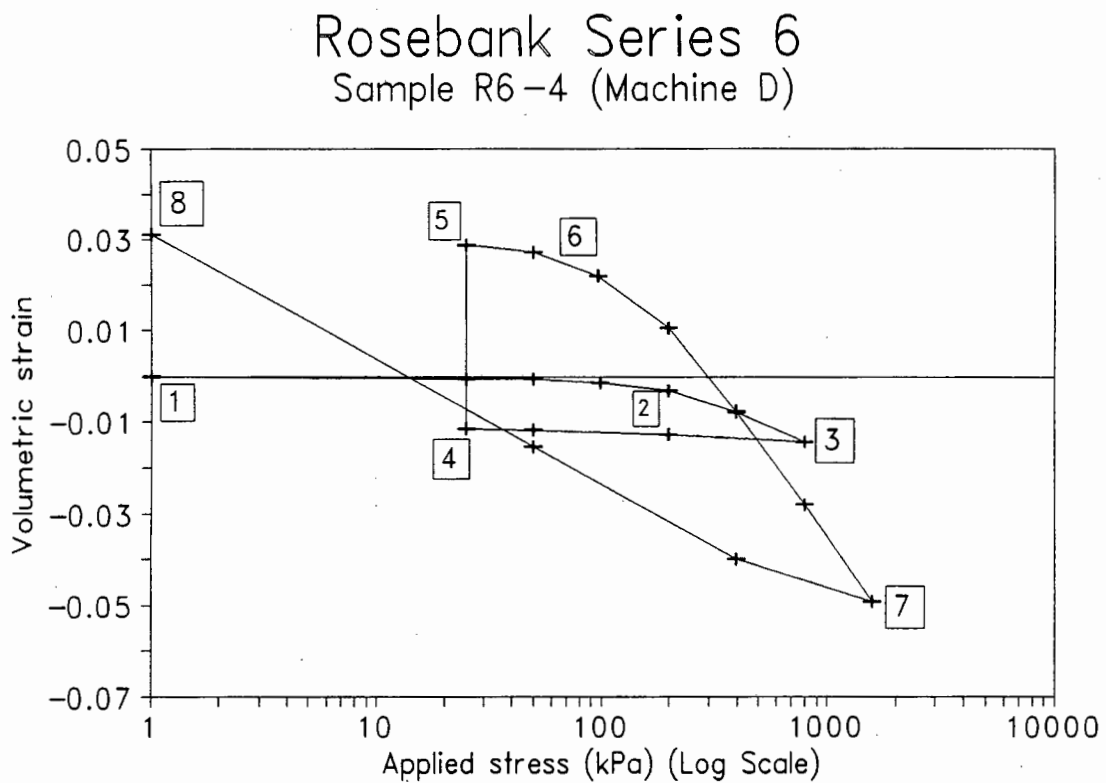


Figure 5.6 Volumetric strain versus logarithm of applied stress for a test from series 6

The loading cycle is similar to series 2. The sample is loaded along ①→②→③ to 800 kPa, exhibiting yielding at ②. The load is reduced to 25 kPa along ③→④, during which the sample exhibits elastic swelling. Water is added and the sample heaves to ⑤. The sample is then reloaded to 1700 kPa at ⑦, yielding at ⑥ and deforming elasto-plastically along ⑥→⑦. Unloading commences at ⑦ and the sample swells elastically along ⑦→⑧. The elastic

bulk modulus for the sample when fully saturated is different to the modulus for the sample when unsaturated, confirming observations made for test series 2.

5.3.6 Test Series 5

The series examines the response of the soil when wetted under a nominal load of approximately 1 kPa.

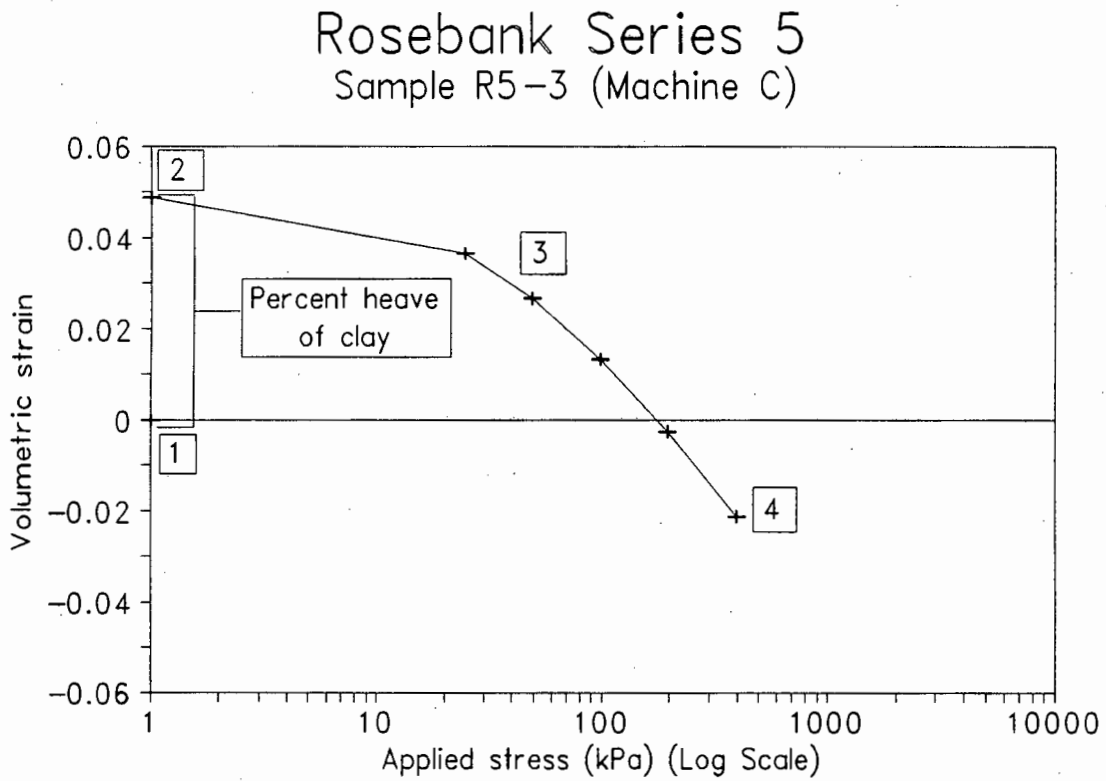


Figure 5.7 Volumetric strain versus logarithm of applied stress for a test from series 5

Figure 5.7 shows a typical result from series 5. Water is added to the sample immediately after placement in the oedometer causing the sample to heave from ① to ②. This heave strain represents the percent heave of

the clay. The sample is then loaded to 400 kPa at ④, showing yielding at ③ and elastic-plastic response along ③→④.

5.4 Analysis of Test Results

Some of the trends suggested by the test results are discussed in the description of the test series in §5.3. In order to confirm these trends and establish values for the parameters which characterise heaving clay behaviour a more detailed numerical investigation of the results was performed. This section describes how the numerical data were obtained from the test results. The data are then plotted and tabulated to demonstrate the characteristics of the material behaviour.

5.4.1 Generation of Numerical Data

The aim of the test programme was to examine the heave and mechanical consolidation behaviour of the clay. The test results had to be processed to give information on both of these aspects. The heave parameters that could be measured are the heave strain shown by the sample upon wetting, the applied load under which heave occurred and the *heave pressure* from the "heave-consolidation method" (see §3.2.2).

The consolidation of the samples under changes of applied load is referred to as "mechanical consolidation". This is done to distinguish between this process and heaving due to wetting, which is a chemical process. The results of consolidation tests performed in an oedometer are traditionally plotted to a semi-logarithmic scale because straight line relationships exist between the volumetric strain and the logarithm of the

applied stress. The bulk modulus is defined as the ratio of change of hydrostatic stress to change to volumetric strain according to a natural scale. The slopes of the lines in an oedometer test result plot are thus inversely related to the bulk modulus.

Two linear relationships between volumetric strain and logarithm of applied stress can be identified for oedometer test results. The slopes of the two lines are given the symbols λ and κ . The line with the slope λ applies to the condition where the soil is deforming elasto-plastically, and can be calculated from the formula

$$\lambda = \frac{\Delta\epsilon_v}{\log\left(\frac{\rho_1}{\rho_0}\right)} \quad (5.1)$$

where $\Delta\epsilon_v$ is the change in total volumetric strain;
 ρ_1 is the final applied stress value;
 ρ_0 is the initial applied stress value.

The κ line applies to the condition where elastic deformation is occurring, and is calculated in a similar way to λ . They are both defined as positive values even though the slopes of the lines are negative. In order to distinguish these parameters from the standard bulk modulus they are referred to as "logarithmic bulk moduli". λ is the "elastic-plastic logarithmic bulk modulus" and κ is the "elastic logarithmic bulk modulus."

The change in material response from elastic (κ line) to elastic-plastic (λ line) occurs at a stress value termed the *preconsolidation pressure*, which will be given the symbol p_c . This is essentially identical to the yield stress of the clay, and the terms *preconsolidation pressure* and *yield stress* are used interchangeably in this thesis.

The methods used for measuring λ , κ and p_c are described below. For heaving clay the values of these parameters may be influenced by the saturation of the soil, and hence values before and after wetting were measured.

Rosebank Series 4 Sample R4-4 (Machine D)

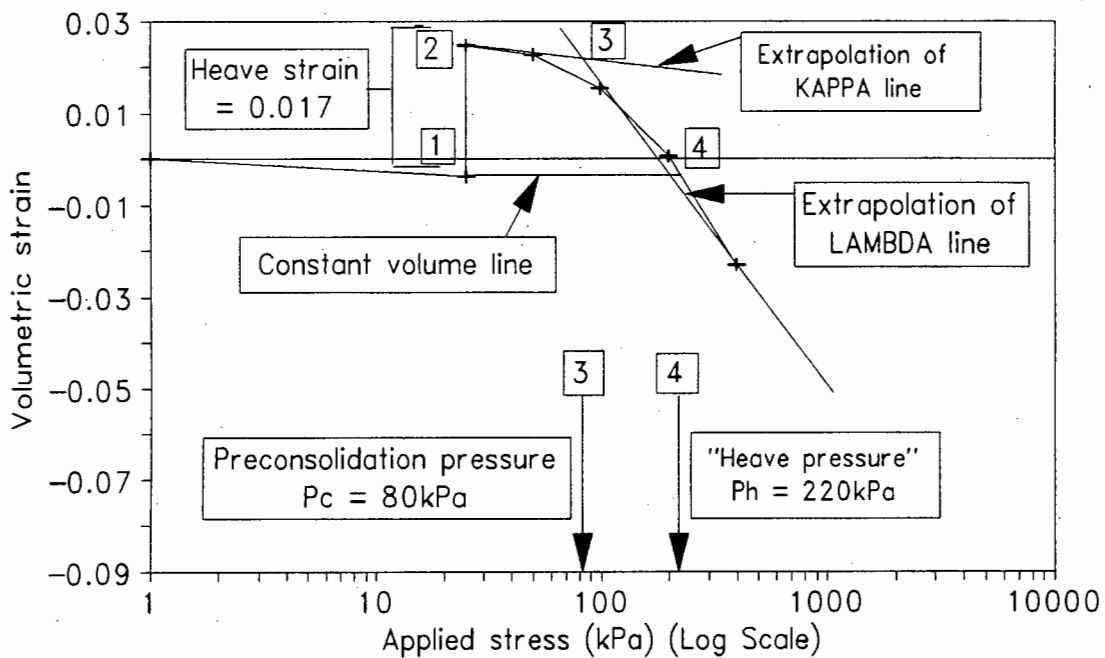


Figure 5.8 Establishing the heave strain, *heave pressure* and preconsolidation pressure p_c from the test results.

Figure 5.8 shows how the heave strain, *heave pressure* and *preconsolidation pressure* (yield stress) can be measured from a typical test. The heave strain is the strain difference between points ① and ②. For this particular test the heave strain was 0.017 and occurred under an applied stress of 25 kPa. The heave pressure (heave-consolidation method) is read from point ④ which is at the same volume as point ①. The yield pressure was determined by extrapolating the λ and κ lines (elastic and elastic-plastic response lines) to their intersection at point ③.

Rosebank Series 3 Sample R3-3 (Machine C)

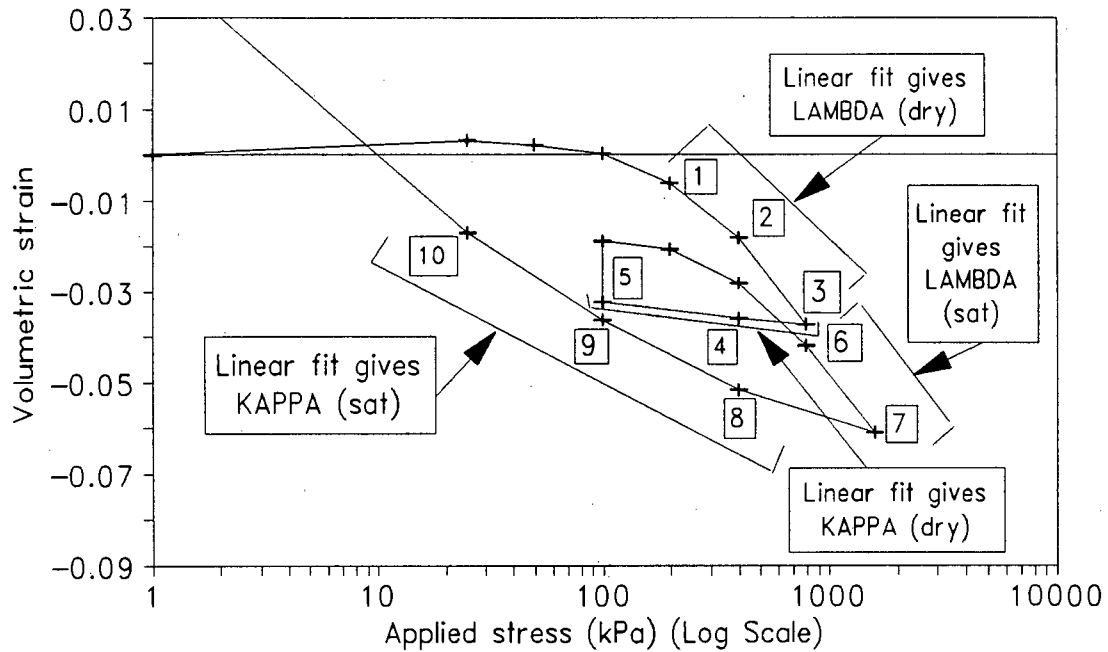


Figure 5.9 Establishing the values of λ and κ for partially and fully saturated (dry and wet) conditions

The method of calculating λ and κ is shown in Figure 5.9. Two values for each of λ and κ are possible, one evaluated before and the other after wetting. A linear regression fit of data points ①→②→③ will give the value of λ_{dry} . A linear fit through data points ⑥ and ⑦ will give λ_{sat} . The value of κ_{dry} comes from a regression of the data points represented by ③→④→⑤, while κ_{sat} can be obtained from a regression of the data for ⑦→⑧→⑨→⑩.

5.4.2 Quantification of Heave Parameters

Series 1, 4 and 5 gave data for derivation of the heave parameters *percent heave* and *heave pressure*. The heave strains measured in each test were averaged and this was plotted against the stress value applied

to the sample at wetting, as shown in Figure 5.10.

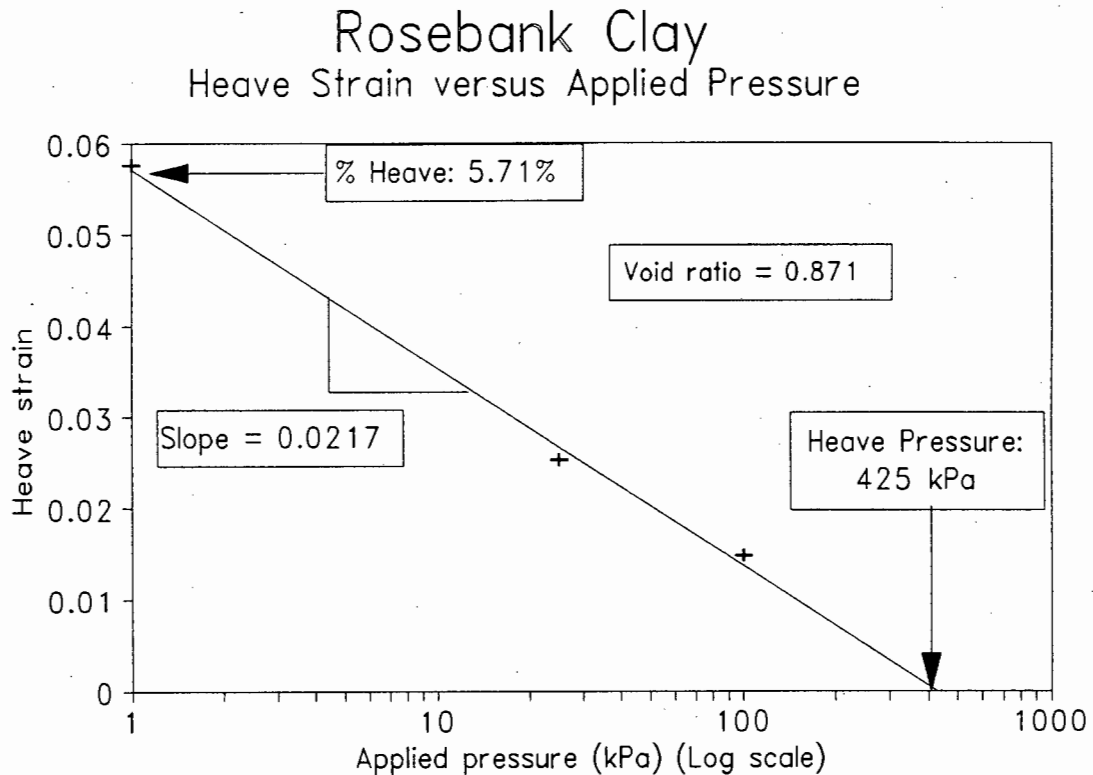


Figure 5.10 Heave strain versus logarithm of applied stress for Rosebank clay (undisturbed samples)

The slope of the graph is linear when the applied stress is plotted to a logarithmic scale. It can be used to derive the heave parameters by the "different pressures method" (see §3.2.2). The *percent heave* is the heave strain for an applied stress of 1 kPa and can be read directly from the graph to be 5.7%. The *heave pressure* is the applied stress value at which heave strain is constrained to be zero and is measured by extrapolation of the graph to be 425 kPa. This curve has been generated for samples that have not been subjected to a cycle of loading and unloading before wetting. The average initial void ratio of the different samples tested was 0.871.

The *heave pressures* measured by the "different pressures method" can now be compared with the values measured by the "heave-consolidation" method. This is done in Table 5.2, which shows the average value of the *heave pressure* given by the "heave-consolidation method" for each of the series 1, 4 and 5.

Table 5.2 Comparison of *heave pressures* measured by two different methods

"Heave-Consolidation Method"	
Test Series	Average <i>heave pressure</i> (kPa)
1	386
4	169
5	162
Average	239
Value from "different pressures method": 425 kPa	

The table shows that there is a significant discrepancy between the *heave pressure* value for series 1 and the values for series 4 and 5 using the heave-consolidation method. The average value is about half the value obtained for the different pressures method. This confirms the dependence of the heave-consolidation method upon the stress path that the sample is subjected to. The different pressures method gives more consistent results.

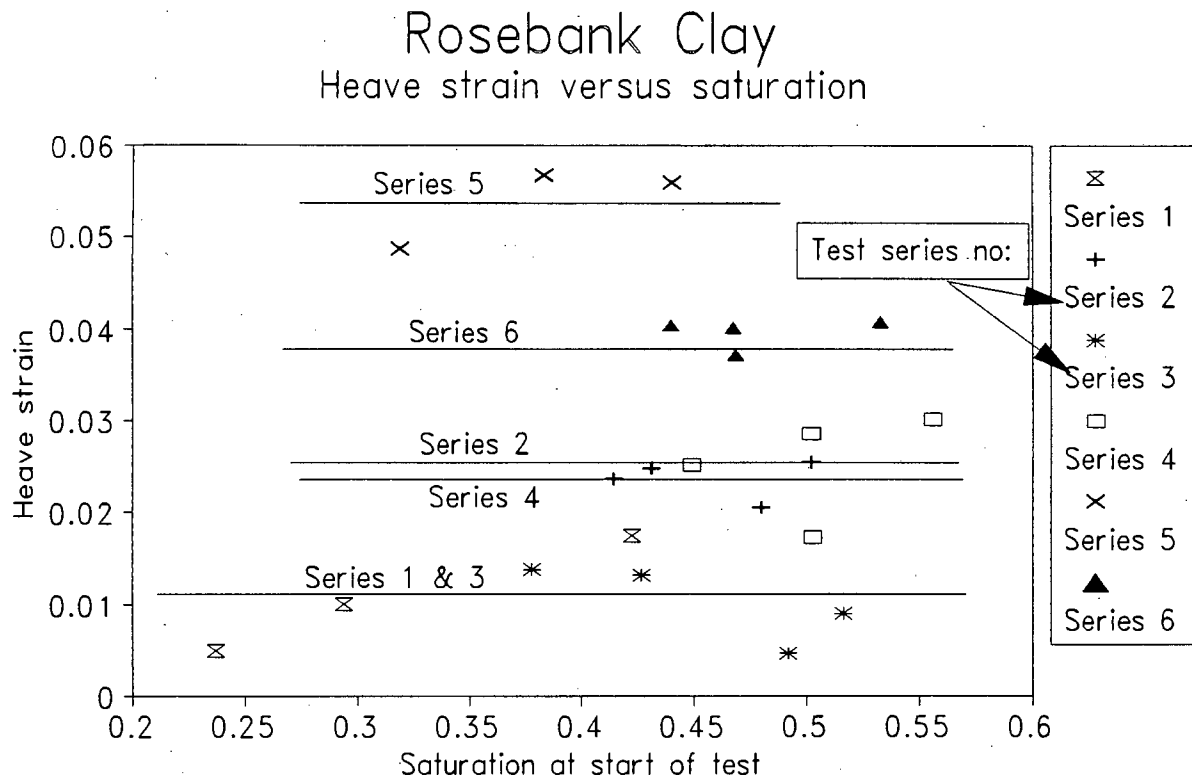


Figure 5.11 Relationship between initial saturation and heave of samples

The influence of soil moisture conditions and sample densities on heaving was also investigated. Figure 5.11 shows the relationship between the heave of the samples against their initial saturations. There is a degree of scatter in the plotted points but it appears that in general the relationship between heave and initial saturation is flat, confirming that the moisture content of the samples is below the shrinkage limit value³ and therefore the exact value of the initial moisture content is not a factor which has to be considered when analysing the results³.

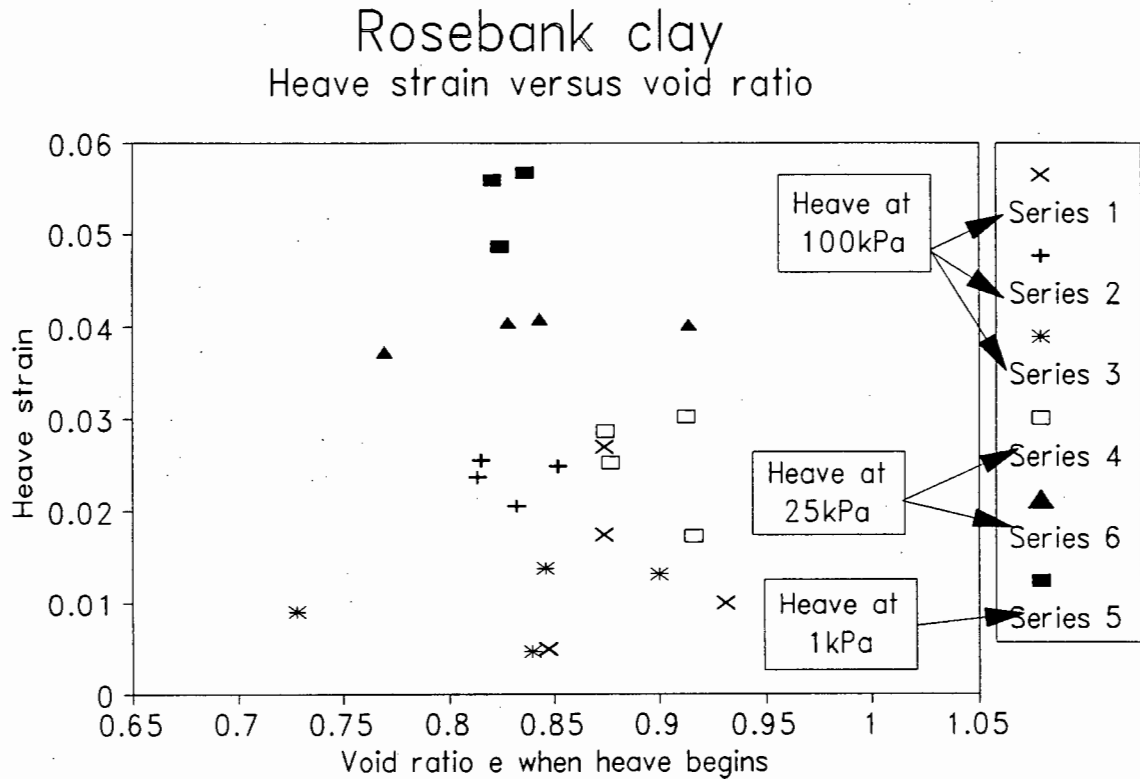


Figure 5.12 Relationship between heave and void ratio when water is added to sample

A plot of the heave versus the void ratio of the sample when heave begins is shown in Figure 5.12. It is difficult to identify clear trends on the plot. The influence of the applied stress on the heave must also be considered. The high degree of scatter in the results can to some extent be attributed to inaccuracy in measuring the sample volume in the oedometer ring before the test, and also to the varying nature of samples tested in their undisturbed state. Plots of the average values of the heaves and void ratios are shown in Figure 5.13, and can be interpreted more meaningfully than the data in Figure 5.12.

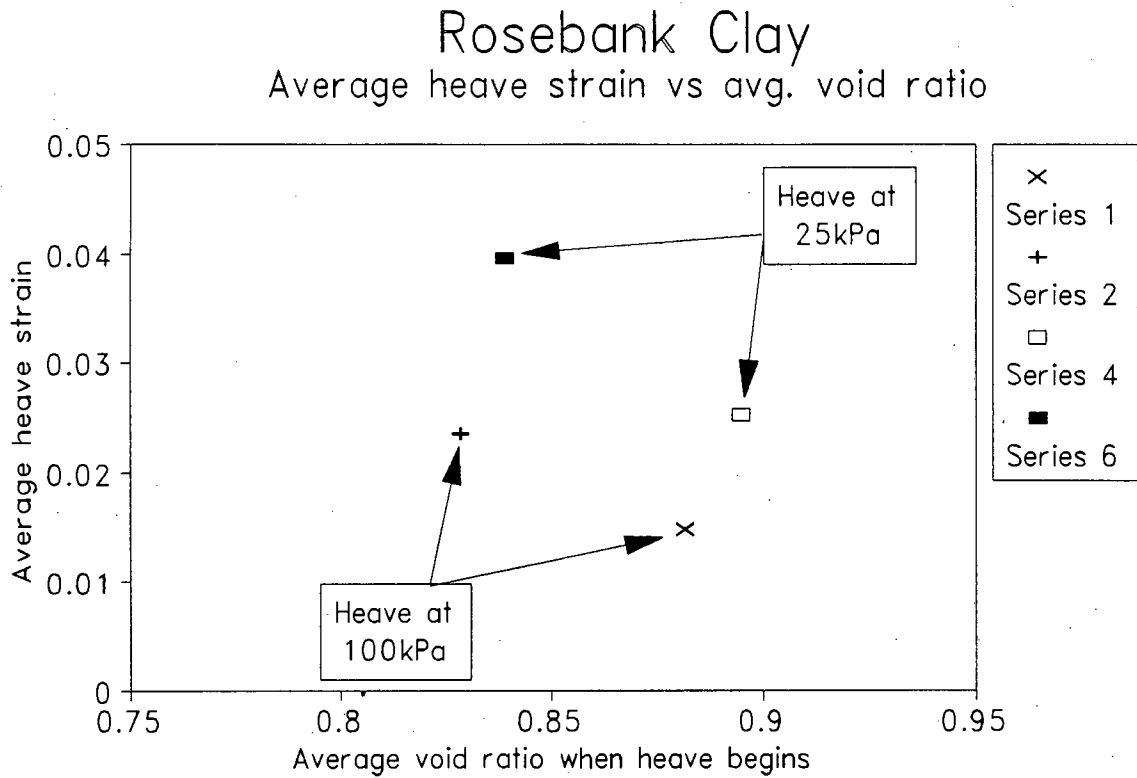


Figure 5.13 Averaged heave versus averaged void ratio when heave begins

Only series 1, 2, 4 and 6 are considered in Figure 5.13. In series 1 and 4 the samples are wetted without applying a loading/unloading cycle to the sample first, while in series 2 and 4 the samples are subject an initial load cycle. The average void ratio of the samples that have been cycled are lower than the samples that have not, indicating that irreversible plastic deformation has occurred. The heave of the cycled samples is greater than that for the uncycled samples, suggesting that the process of plastic deformation influences the heave behaviour of the clay.

5.4.3 Mechanical Consolidation Behaviour

The changes in volume of the clay due to changes in loading rather than changes in saturation is characterised by the logarithmic bulk moduli λ

and κ and the yield stress p_c at which the behaviour changes from elastic to elastic-plastic.

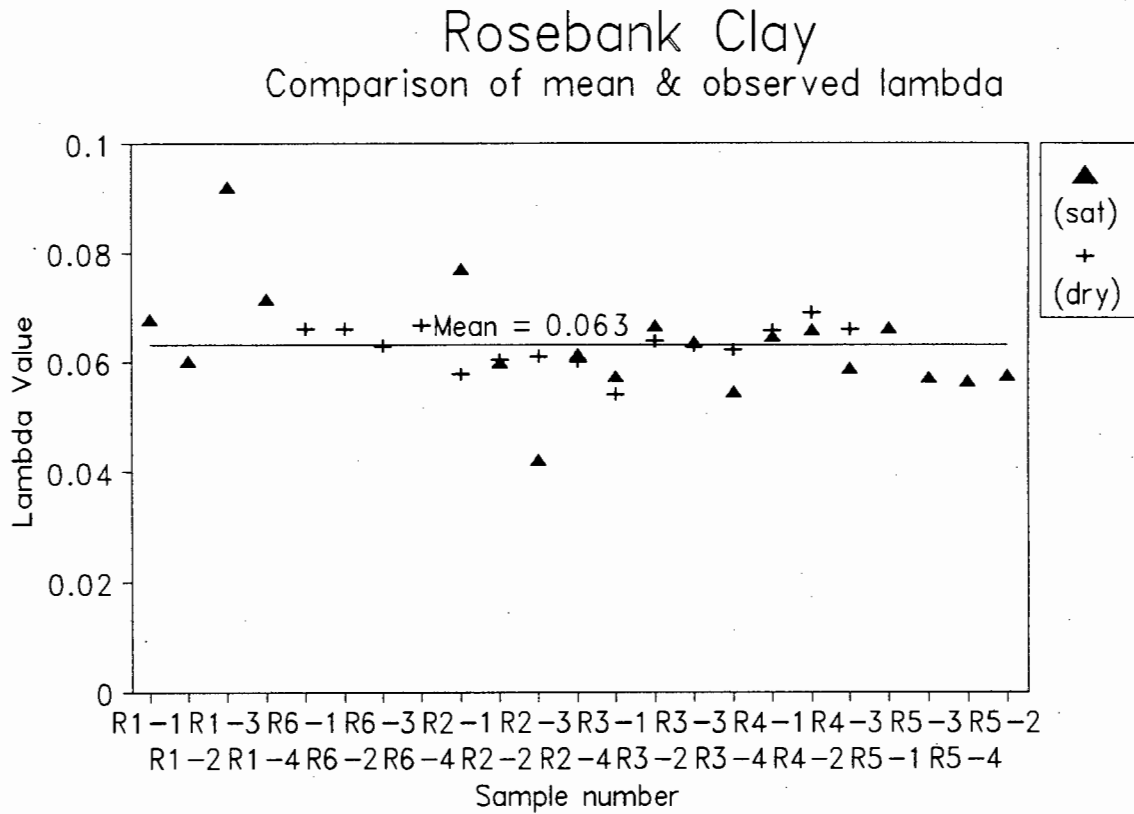


Figure 5.14 Comparison of observed λ values for dry and fully saturated Rosebank clay with mean λ value

Figure 5.14 shows a plot of the mean value of the value of λ , the elastic-plastic bulk modulus, along with the values measured for each individual test. The values of λ for both the dry and fully saturated conditions are so similar that it has been assumed that they are the same. The mean value of λ is 0.063.

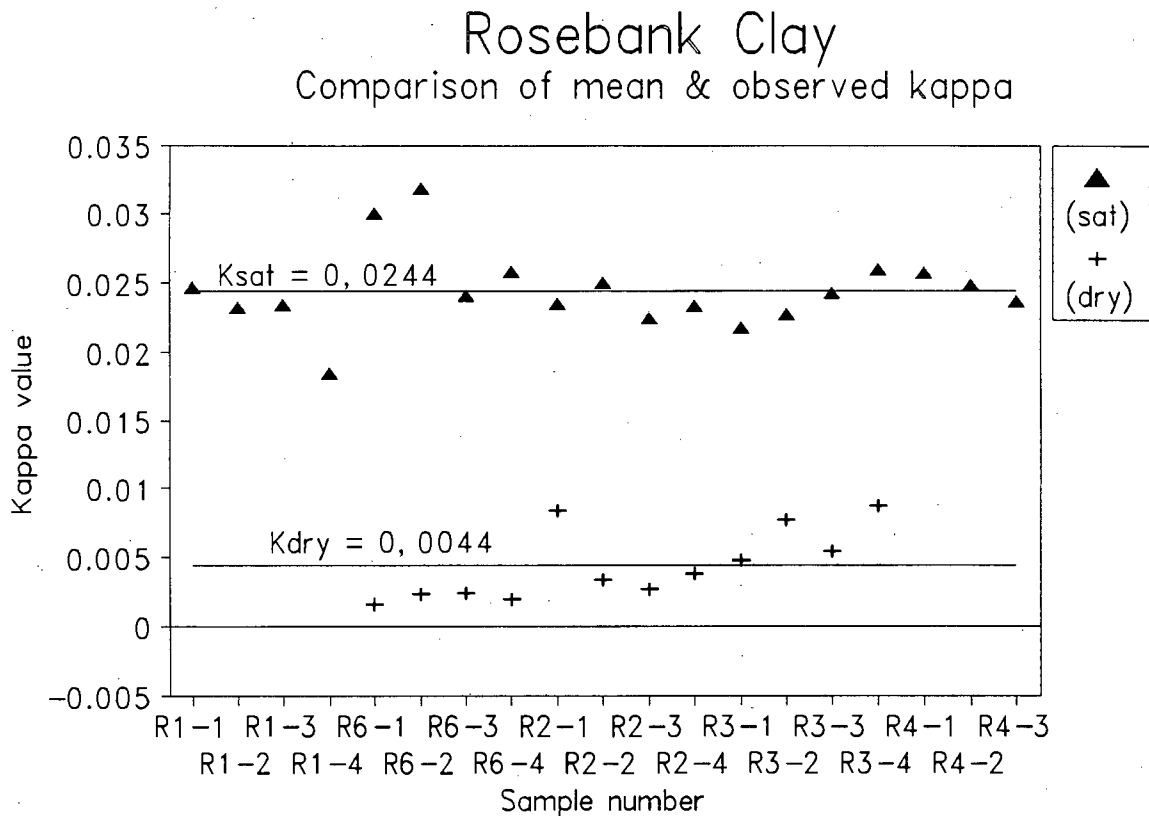


Figure 5.15 Comparison of observed κ values for dry and fully saturated Rosebank clay with mean κ values

A similar scatter plot for the elastic logarithmic bulk modulus κ is shown in Figure 5.15. The individual values lie very close to their means, but in this case there is a significant difference between the values for dry and fully saturated conditions, with the κ_{sat} value being about 5.5 times higher than κ_{dry} .

Location of the yield pressure, p_c is important for defining the material response. However, the value of the yield pressure appears to be effected by the process of heave. Test series 2 shows (see Figure 5.3 on page 53). In this series the samples were loaded at their initial moisture content to 1700 kPa. Consideration of the test result plots indicated that the average initial yield pressure p_c was 452 kPa (point ② on Figure 5.3). The load applied to the samples was therefore in excess of the initial p_c .

The yield stress of the material at the end of this loading process would therefore have increased to 1700 kPa (③ on Figure 5.3). After unloading to 100 kPa, wetting, heaving and reloading, the average p_c observed from the plots was 310 kPa (⑥ on Figure 5.3), which represents a substantial reduction in the yield stress.

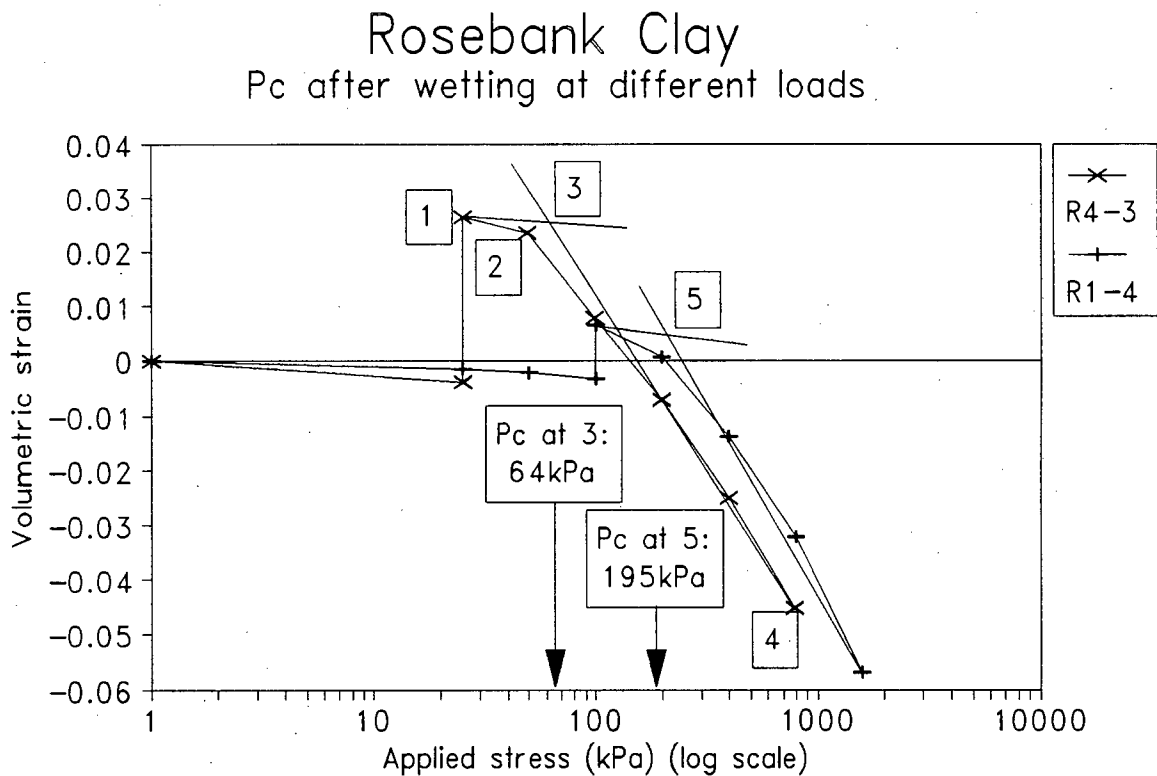


Figure 5.16 Yield stresses p_c for samples wetted at different applied loads

A second observation of the response of p_c was that its value after wetting was related to the applied stress imposed on the clay during wetting. Figure 5.16 illustrates this. Sample R4-3 is wet at 25 kPa and has a p_c of 64 kPa (③), whereas sample R1-4 which is wetted at 100 kPa has a p_c of 195 kPa (⑤). The figure also shows that the general shape of the curves for both the samples after wetting is the same but they are displaced relative to one-another. This trend is confirmed by the consideration of other results. A possible explanation for this proposes

that the curve ①→②→④ traced by the samples after heaving represents a slow transition from elastic to elastic-plastic behaviour rather than a sudden change at a precisely defined yield pressure p_c (point ③ on the figure). Thus plastic deformation actually begins at ① and not at ③.

In order to test this proposal a means of quantifying the shape of the curve was sought. The simplest means of doing this was by using the value of p_c obtained for each curve and dividing this by the value of the applied stress at wetting, p_{wet} . The value of λ and κ_{sat} have been shown to be consistent for all the samples. Therefore if the ratio of the yield stress (representing the transition from the κ line to the λ line) to the stress at wetting is constant then the shape of the curves for all samples after wetting will be roughly the same. The average values of the ratio of p_c to p_{wet} is presented in Figure 5.16, Table 5.3.

Table 5.3 Relationship between p_c and applied stress at wetting

Test Series	p_{cdry} (p_c before wetting) (kPa)	p_{csat} (p_c after wetting) (kPa)	p_{wet} (kPa)	p_{csat}/p_{wet}	p_{cdry}/p_{csat}
R1	451.5*	270	100	2.7	1.67
R2	1700	310	100	3.1	5.48
R3	800	312	100	3.12	2.56
R4	451.5*	79.5	25	3.18	5.68
R5	451.5*	20.5	1	20.5	22.0
R6	800	129	25	5.15	6.21

*These p_{cdry} values are assumed from the p_c value for the initial loading/unloading curve for series 2. The other p_{cdry} values are taken as the value of the highest applied load in the loading/unloading cycle that preceded wetting.

The value of the ratio p_{cdry}/p_{csat} is different for every test series, which suggests that there is no clear relationship between the values of the yield stress p_c before and after wetting. Values of the ratio between these two results varies for every test series. However, for the first four of the six series the ratio p_{csat}/p_{wet} is very close to 3.1, which suggests that the shape of the curve after wetting is similar for all these test series. For series 5 the ratio is 20.5 which is significantly different from the other ratios. The overburden pressure on the sample in the field would be about 20 kPa because the samples were taken from a depth of 0.6m below the soil surface. The p_{csat} value for these samples was 20.5 kPa which is close to the overburden pressure. This may be a reason why the observed p_{csat} value and hence the high ratio p_{csat}/p_{wet} for the series.

The value of the p_{csat}/p_{wet} ratio for series 6 is significantly different to the value for the first four samples. There is insufficient test data available to establish whether this result is merely a statistical outlier or whether the change in plastic deformation of the samples during the initial loading/unloading cycle in series 6 caused some increase in the value of p_{csat} relative to the value for series 4 where there was no initial stress cycle. The ratio of the two p_{csat} values is 1.62. Series 2 has also undergone a loading/unloading cycle relative to series 1 and ratio of the p_{csat} values for these two series is 1.15 which also indicates an increase in the value of p_{csat} after a stress cycle, although by a lower factor.

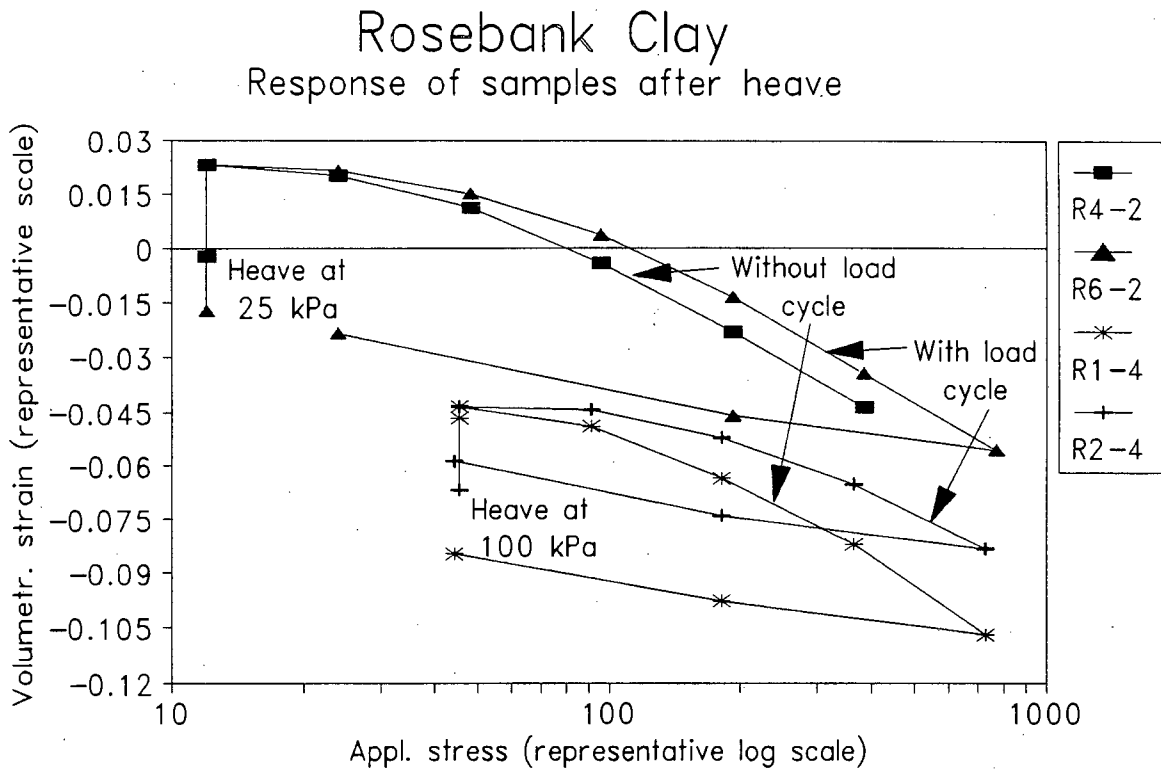


Figure 5.17 Volumetric response (after heave) of samples that have not experienced a load cycle before heave compared with volumetric response (after heave) of samples that have experienced a load cycle before heave.

Figure 5.17 shows a plot of the logarithm of applied stress versus the strain for the behaviour of the samples from the point where water is added. Samples where a load cycle was applied before wetting are compared with samples that were not cycled. Two comparisons are made: one where the samples were wetted at 25 kPa and the other where the cycles are wetted at 100 kPa. The stress and strain values have been adjusted so that the points representing the strain of the samples at the end of the heave process coincide. This is done so that the shape of the curves for loading after heaving can be compared.

For samples wetted at 100 kPa and samples wetted at 25 kPa the general shape of the loading curves after wetting is similar. However, the

curves for those samples that were subjected to a load cycle and plastic strain before wetting are displaced relative to those that were not pre-loaded. The difference in the magnitude of the heave can also be seen, with pre-loaded samples heaving more. This illustrates that pre-loading and plastic deformation have an effect on the heave magnitude and the yield stress of the sample after wetting (p_{csat}), but there is insufficient test data to quantify these effects.

This Chapter has described the test programme performed as part of this research project to investigate the effect of cyclic loading and wetting on the magnitude of heave and the mechanical consolidation behaviour of an expansive clay. The means of analysing the test data has been described and a discussion of the results has presented the aspects of the behaviour of the clay that could be identified from the test data. The aspects of the behaviour identified are summarised below:

1) Heave Behaviour

There is a straight line relationship between the heave strain and the logarithm of the applied load at which the sample was wetted. This relationship was used to establish the parameters of *percent heave* and *heave pressure*. The magnitude of the heave strain is increased when the clay is first loaded and unloaded to cause plastic deformation, thereby increasing the dry density of the sample.

2) Mechanical Consolidation Behaviour

The elastic-plastic logarithmic bulk modulus λ of the clay appears to be the same for partially and fully saturated conditions. However, the elastic logarithmic bulk modulus κ varies with the degree of saturation. The value of κ increases as

the sample changes from a partially saturated to a fully saturated state.

The value of the yield stress p_{csat} of the clay after heave is effected by the load applied to the sample during heaving and also by the plastic strain experienced by the sample prior to wetting.

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2. British Standard 1377:(1990): *Methods of Test for Soils for Civil Engineering Purposes*, Part 5. (British Standards Institution, London, 1990).
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CHAPTER 6

DEVELOPMENT OF A CONSTITUTIVE MODEL FOR HEAVING CLAY FROM ROSEBANK

Constitutive models to describe the behaviour of a material are the basis of analysis techniques like the finite element method. The development of a constitutive model for heaving clay would make powerful analysis methods available to designers so that structures founded on an heaving clay can be designed with greater confidence. In order to develop a model, however, a good understanding of the material response to changes in loading and wetting is required. A survey of available literature has established a reasonable understanding the factors that cause heave and influence its magnitude. This information has been presented in Chapter 3. The testing programme on samples of expansive clay from Rosebank has led to an understanding of the response of expansive clays to loading and unloading cycles before and after wetting. There is now sufficient information available to proceed with the development of a constitutive model for the volumetric behaviour of Rosebank clay under loading and wetting.

This Chapter discusses the model development in detail. It begins with an idealisation of the volumetric behaviour of the clay based on the results of the test programme. This generalised material behaviour can be subdivided into three different aspects of constitutive response. The rationale for the division is presented, after which each aspect is discussed. This involves a theoretical description of each aspect, followed by the mathematical formulations which model the constitutive behaviour. The Chapter concludes with some evidence from literature which supports the proposal.

6.1 Idealisation of the Generalised Behaviour of Rosebank Clay

The test results presented in Chapter 5 show that linear relationships exist between the volumetric strain and the applied stress. Two linear responses can be seen in the test results: an elastic one represented by the logarithmic bulk modulus κ and an elastic-plastic one represented by the logarithmic bulk modulus λ . The stress at which the response changes from elastic to elastic-plastic is the yield stress or preconsolidation pressure, p_c . The strain/log stress relationships for heaving clay can be generalised by a series of straight lines in a strain/log stress space.

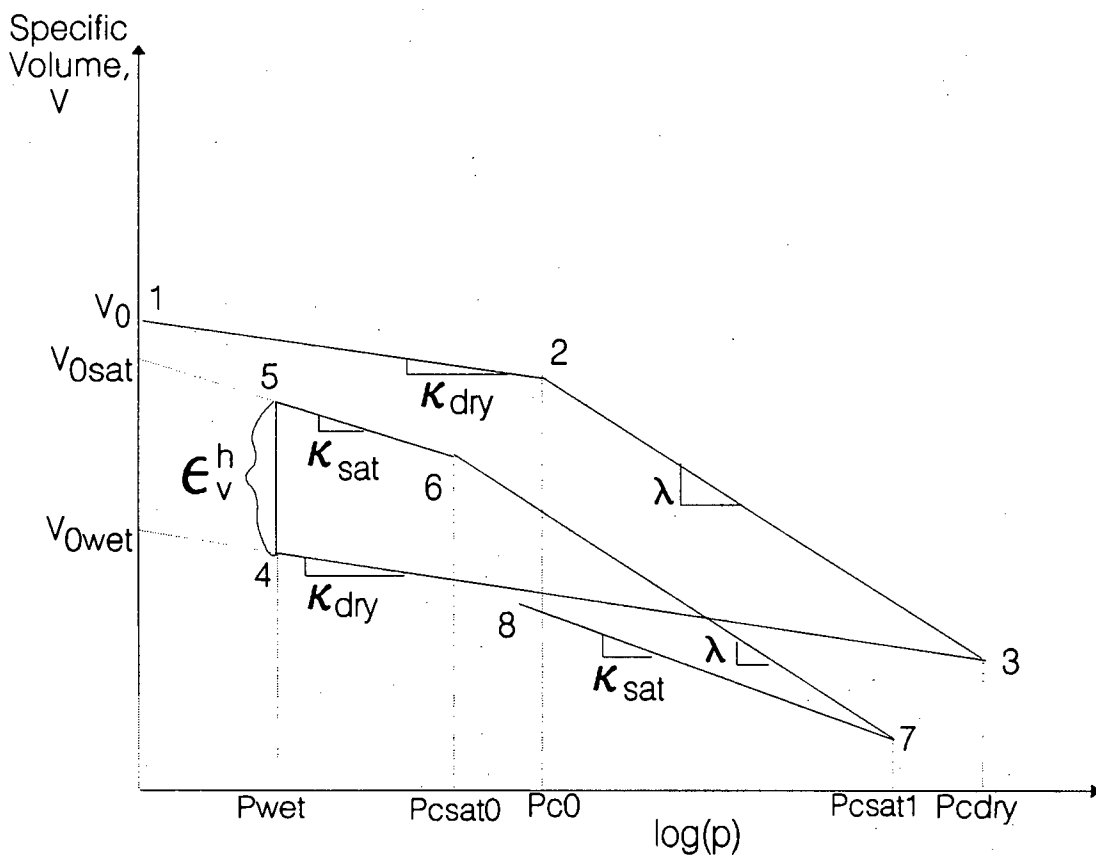


Figure 6.1 Generalisation of the volumetric behaviour of Rosebank clay under loading and wetting

The specific volume, v , of a soil is defined as the total volume of the soil which

contains a unit volume of solids¹¹, and is expressed by the formula $v = 1 + e$. Figure 6.1 shows a generalisation of material response in specific volume-log stress space. The initially sample state is at point ①, with an initial specific volume of v_0 at a stress of 1 ($\log(p) = 0$). It is initially partially saturated ("dry" of full saturation). As the sample is loaded the specific volume reduces along the elastic response line κ_{dry} until point ②, which is at the initial yield stress p_{c0} , is reached. The elastic-plastic response begins along the line ②→③ of slope λ . During this process the yield stress p_c increases along with the applied stress. At ③ unloading commences, and thus the sample unloads elastically along line ③→④ of slope κ_{dry} . During plastic deformation strain hardening of the material occurred causing an increase in the yield stress from p_{c0} to the maximum value of the stress applied in the cycle ①→②→③→④. The final yield stress after hardening is p_{cdry} .

At ④ water is added to saturate the clay. This triggers heave and a heave strain ϵ_v^h occurs to point ⑤. If the sample is then reloaded it again follows an elastic loading line to point ⑥ where it reaches a new p_c value, p_{csat0} . This value is different to p_{c0} and p_{cdry} . It is assumed that the elastic line ⑤→⑥ has a slope of κ_{sat} , the saturated elastic modulus. The assumption is made to simplify the description of the material behaviour, because then all elastic strains after wetting follow κ_{sat} , and all elastic strains before wetting follow κ_{dry} . Experimental evidence from the test results suggests that the samples may yield at a load very close to the applied load at wetting p_{wet} , and thus ⑤→⑥ is actually an elastic-plastic transition curve and not an elastic response. Also, for many of the test results the slope of ⑤→⑥ is initially between κ_{dry} and κ_{sat} . The assumption of elastic behaviour at slope κ_{sat} is merely to simplify the formulation of constitutive equations.

From ⑥ to ⑦ the sample deforms elasto-plastically and hence the yield stress increases with the applied stress until unloading commences at ⑦, after which the yield stress is p_{csat1} . Unloading along ⑦→⑧ will be elastic and the line will have

the slope κ_{sat} .

The clay follows the same general pattern, with phases of elastic straining, yielding, plastic deformation and hardening under loading, and elastic unloading before and after wetting. This can be presented conceptually in a log stress/ volumetric strain diagram which shows cyclic loading and unloading for a dry and a saturated sample. Figure 6.2 shows the idealised behaviour of the clay under cyclic loading. This is an analogy to the presentation of the stress strain curves for an ideal soil-like material presented by Atkinson and Bransby¹².

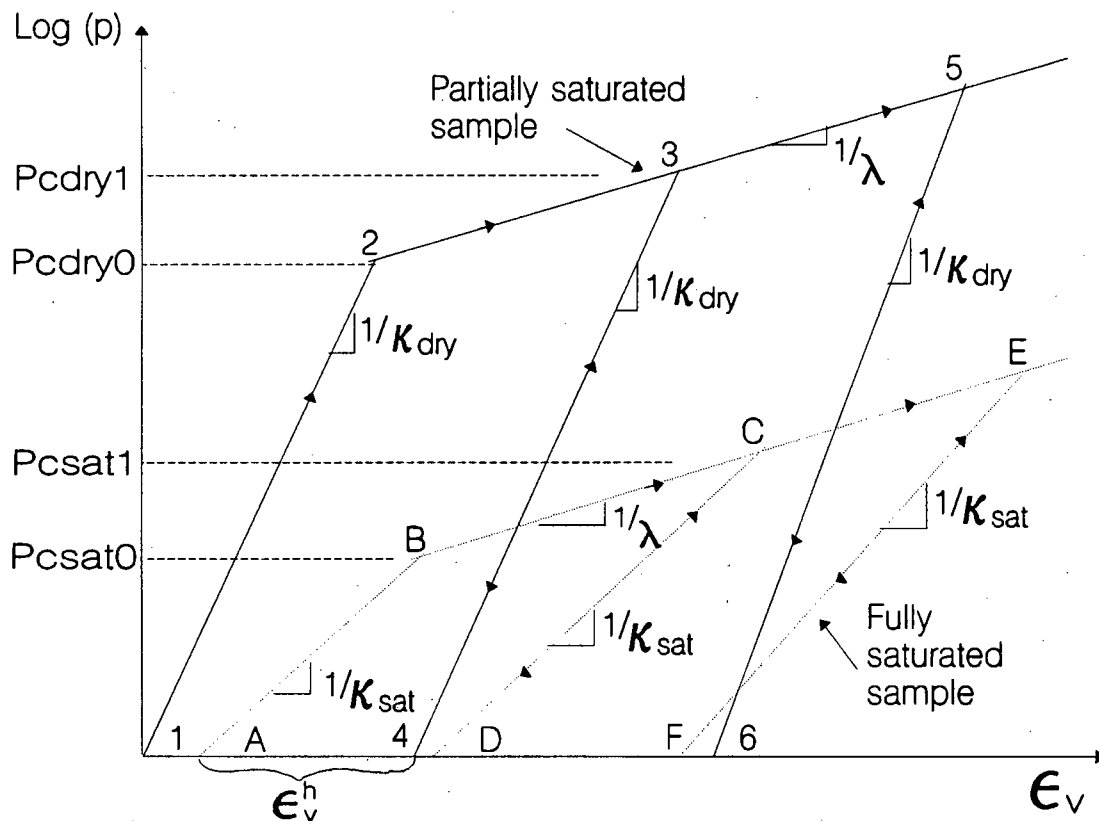


Figure 6.2 Idealisation of the response of partially and fully saturated clay under cyclic loading

The partially saturated sample undergoes elastic loading from ① to ② at which point it yields at a stress of p_{cdry0} (the initial preconsolidation pressure). It then deforms plastically and hardens from ② to ③, which establishes the new yield

stress at p_{cdry1} . Unloading occurs along ③→④. The unrecoverable deformation incurred during the cycle is the distance ①→④. A second cycle ④→③→⑤→⑥ then commences with reloading along ④→③.

For the fully saturated sample a typical cycle is represented by A→B→C→D. The difference between the responses is that the elastic logarithmic bulk modulus κ_{sat} is different to κ_{dry} and the initial yield stress p_{csat0} is lower than p_{cdry0} . Fundamentally, however, the type of material response is the same and only the values of κ and p_c are different.

An unsaturated and a saturated material response to cyclic loading has been identified. Heave represents a volumetric strain change experienced by the sample as it is wetted and moves from the family of "dry" cyclic loading response curves to the family of "wet" cyclic response curves. If a "dry" sample undergoes one load cycle ①→②→③→④ and is wet at ④ then the distance ④→A is the heave strain, ϵ_v^h , which "moves" the sample from the "dry" to the "wet" response.

Conceptualising the material behaviour in this way suggests that the constitutive behaviour of the clay can be split into two different aspects. The first is the behaviour of either saturated or unsaturated clay under "mechanical consolidation" (consolidation under applied load or strains at constant moisture content). The second is the heave behaviour of the soil under a change in the soil moisture conditions. Separation of these two aspects of the material behaviour is made for two reasons. Firstly, consideration of the idealisations of the material response in ? and Figure 6.2 suggests that this is a convenient way to analyse the constitutive behaviour of the soil. Secondly, observations of heaving clay behaviour^{1,8} and consideration of clay mineralogy suggests that consolidation under changes in load is a mechanical effect while the heave under changes in moisture is a chemical effect. Heave can be regarded as a chemical reaction between the clay and water and is therefore a totally different

type of material behaviour to the response to loading.

A rational basis exists for considering the mechanical consolidation and heave responses of the clay as essentially different constitutive behaviours. However, the difference in behaviour does not imply that they can be considered in isolation. It has been noted above that the yield stress of the clay during mechanical consolidation after wetting is effected by the value of the applied load at wetting. Also, analysis of experimental data from this research work and the work of de Sousa Vinagre² shows that the initial sample dry density effects the magnitude of the heave. The initial dry density of a sample is changed by the amount of plastic deformation that a sample has experienced, which means that the heave is influenced by mechanical consolidation.

Three aspects of the constitutive behaviour must therefore be considered. These are the mechanical behaviour under load changes, the heave under soil moisture changes, and the effects of consolidation on heave and heave on consolidation. These three aspects of the constitutive behaviour are discussed in the sections below and constitutive equations describing each aspect are derived.

6.2 Volumetric Response to Mechanical Consolidation

The mechanical consolidation behaviour of the clay involves elastic loading and unloading, yielding at the yield stress (preconsolidation pressure) and plastic deformation and hardening for loading beyond the initial value of the yield point. This behaviour is presented schematically in Figure 6.3.

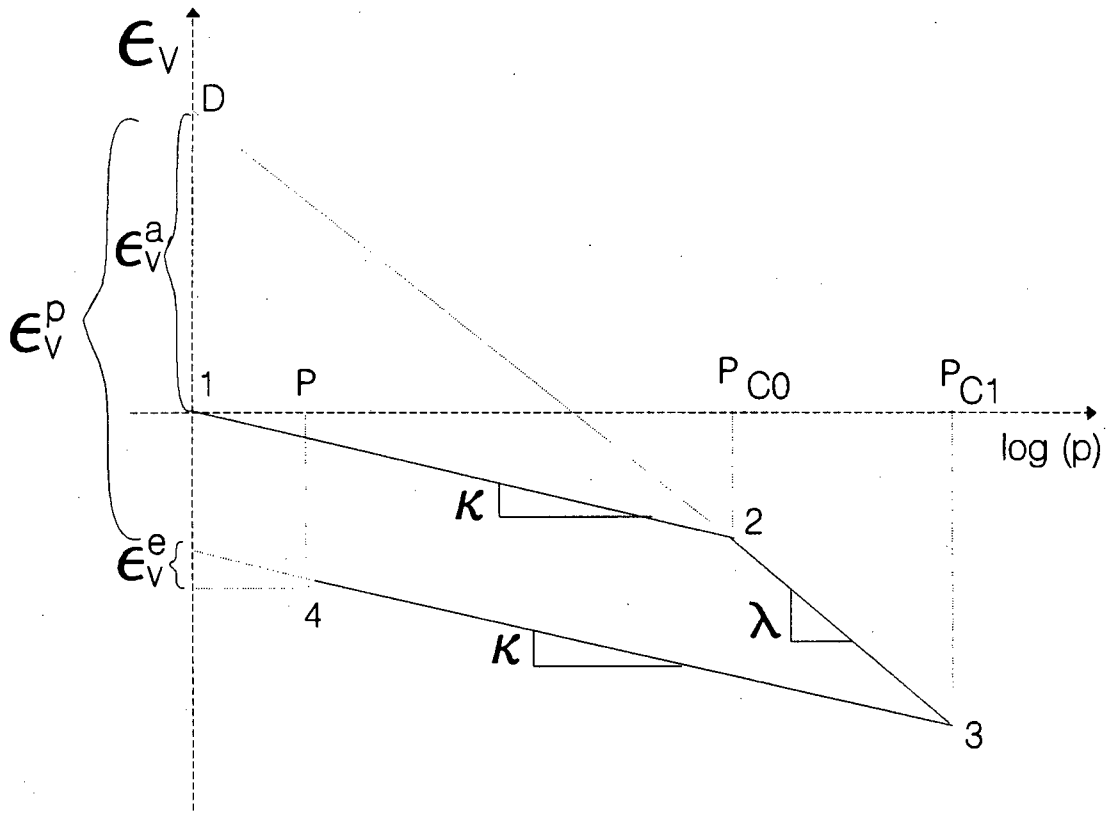


Figure 6.3 Schematic plot of mechanical consolidation behaviour

The plot shows the response of a sample that has been loaded from point ① and undergoes elastic strain until point ②, where it yields and starts to undergo elastic-plastic deformation until ③. The initial yield stress was p_{c0} corresponding to ②, but due to hardening under plastic deformation it increases to p_{c1} at ③. From ③ the sample is unloaded to ④ which represents its current material state.

The plot shown in Figure 6.3 describes the volumetric behaviour of the soil and is thus formulated in terms of the volumetric strain ϵ_v and the hydrostatic stress p . These two parameters are related to the first strain and stress invariants respectively and are defined by the equations below:

$$\epsilon_v = I_{1,\epsilon} = (\epsilon_{11} + \epsilon_{22} + \epsilon_{33}) \tag{6.1}$$

where ϵ_v is the total volumetric strain;
 $I_{1,\epsilon}$ is the first invariant of strain;
 ϵ_{ii} are the direct strain components of the strain tensor.

$$p = \frac{1}{3} I_{1,\sigma} = \frac{1}{3} (\sigma_{11} + \sigma_{22} + \sigma_{33}) \quad (6.2)$$

where $I_{1,\sigma}$ is the first invariant of stress;
 σ_{ii} are the direct stress components.

The choice of the hydrostatic stress as the stress measure for the constitutive formulation was based on continuum mechanics considerations³, rather than on the test methods used. Oedometer machines are not isotropic testing devices, and therefore the formulation of the constitutive equations in terms of hydrostatic stresses is based on the assumption that the test results can be manipulated to give the hydrostatic stresses. One means of doing this is to assume a Poisson's ratio ν for the clay. The horizontal stresses σ_h in the soil in an oedometer test are therefore functions of the vertical stress σ_v and the Poisson's ratio ν :

$$\sigma_h = \nu \sigma_v \quad (6.3)$$

This leads to an expression for the hydrostatic stress:

$$p = \frac{(1 + 2\nu)}{3} \sigma_v \quad (6.4)$$

Three expressions are required to complete the constitutive relations for the material response under mechanical consolidation. These are a constitutive equation, a yield function and elastic and elastic-plastic bulk moduli which relate changes in stress to changes in strain.

6.2.1 Constitutive Equation for Mechanical Consolidation

The constitutive equation can be derived directly from Figure 6.3 (after Britto and Gunn³, §2.5.2). The figure shows that the total strain of the sample ϵ_v , relative to point ① is:

$$\epsilon_v = \epsilon_v^a + \epsilon_v^p + \epsilon_v^e \quad (6.5)$$

where ϵ_v^a is an "adjustment" strain to shift the strain datum (origin) from its current position to a "historic" position at point D (see Figure 6.3);
 ϵ_v^p is the plastic strain referenced to the "historic" strain datum D;
 ϵ_v^e is the elastic strain.

This expression for the total strain is based on the assumption that the total strain can be expressed as the sum of the elastic and plastic strains and is therefore only valid for infinitesimal strains.

Further consideration of Figure 6.3 leads to expressions for ϵ_v^p and ϵ_v^e :

$$\epsilon_v^p = -(\lambda - \kappa) \log_e(\rho) \quad (6.6)$$

and

$$\epsilon_v^e = -\kappa \log_e(\rho) \quad (6.7)$$

where λ is the elastic-plastic logarithmic bulk modulus and
 κ is the elastic logarithmic bulk modulus;
 (Both evaluated in terms of natural logarithms).

Equations (6.5), (6.6) and (6.7) can be combined to give an equation for the total volumetric strain ϵ_v as a function of the hydrostatic stress p and the yield stress p_c :

$$\epsilon_v = \epsilon_v^a - (\lambda - \kappa) \log_e(p_c) - \kappa \log_e(p) \quad (6.8)$$

This can be inverted directly to give the constitutive equation as:

$$p = \exp \left(\frac{\epsilon_v^a - \epsilon_v - (\lambda - \kappa) \log_e(p_c)}{\kappa} \right) \quad (6.9)$$

6.2.2 Yield Function for Mechanical Consolidation

The yield function f can be expressed as the difference between the current values of the hydrostatic stress p and the yield stress p_c .

$$f(p, p_c) = p - p_c \quad (6.10)$$

The yield stress p_c is a function of the volumetric plastic strain ϵ_v^p , as shown in equation (6.6). However, it has been found to be more convenient to express the yield function f in terms of the yield stress p_c rather than the plastic volumetric strain.

6.2.3 Elastic and Elastic-Plastic Moduli for Mechanical Consolidation

The relation between stress and strain during the mechanical consolidation of the soil has been shown as a straight line relationship.

This is only true when the stress is plotted to a logarithmic scale. When drawn to natural axes the stress-strain relationships are curves, which means that the elastic and elastic-plastic moduli are not constant but increase with the strain because the curves are upwardly convex when plotted to a natural scale. This is shown in Figure 6.4.

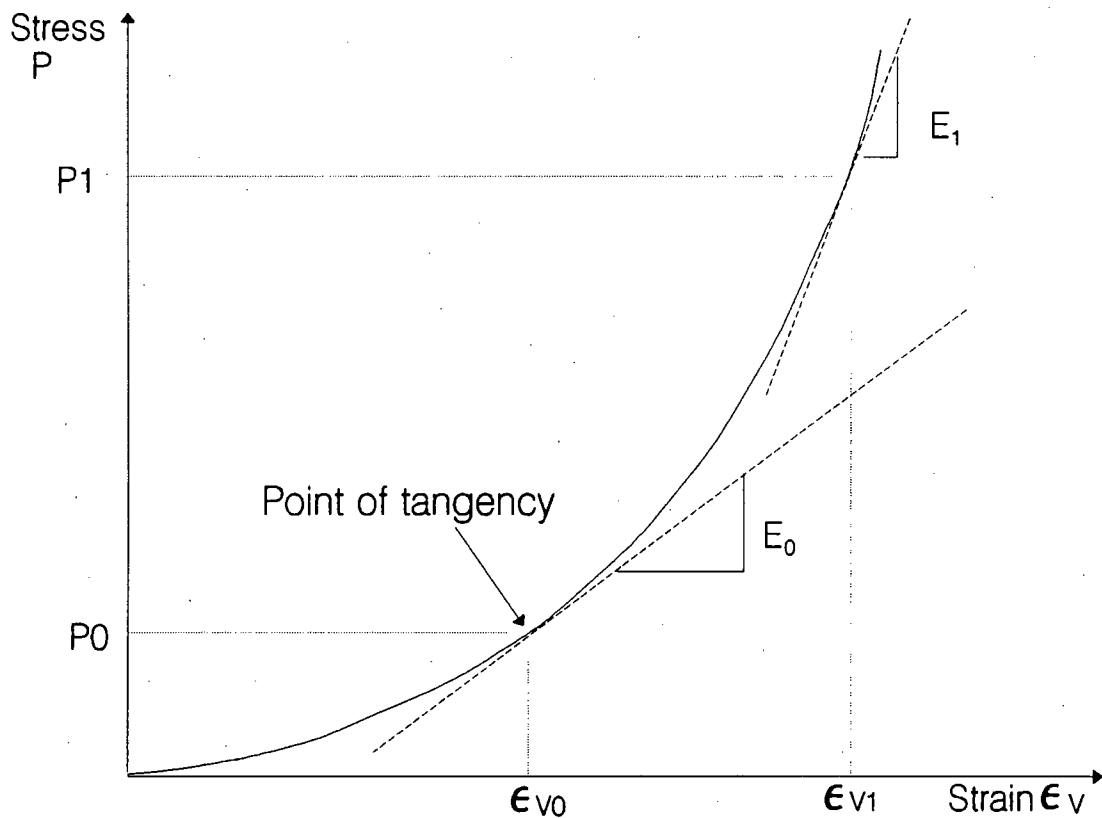


Figure 6.4 Elastic stress-strain behaviour of a soil under isotropic compression, drawn to natural axes

The elastic bulk modulus at a particular strain value is the slope of the tangent to the curve at that particular strain value. The value of the modulus increases monotonically with increases in stress and strain as long as the material remains elastic. A similar curve can be drawn for the elastic-plastic response.

Expressions for the moduli are obtained from derivatives of equation (6.8). When the response is elastic the yield stress p_c is a constant and

thus the derivative of p_c is zero. The elastic bulk modulus K_{el} at a hydrostatic stress of p is therefore

$$K_{el} = \left. \frac{dp}{d\epsilon_{v,el}} \right| = -\frac{p}{\kappa} \quad (6.11)$$

For the condition of loading at the yield stress (plastic deformation) the yield stress is no longer constant but is equal to the value of the applied stress. Thus the derivatives of p and p_c are equal, and the elastic-plastic bulk modulus K_{pl} at a hydrostatic stress p (where p is on the yield surface) is

$$K_{pl} = \left. \frac{dp}{d\epsilon_{v,pl}} \right| = -\frac{p}{\lambda} \quad (6.12)$$

The constitutive relations for the mechanical consolidation have been developed, with expressions derived for the constitutive equation, the yield function and the elastic and elastic-plastic bulk moduli. These equations are valid whether the clay is partially or fully saturated, as only the value of κ is effected by changes in soil moisture. The value of p_c is effected during heaving, but the constitutive equations for this are discussed in §6.4.

6.3 Volumetric Response during Heave

The literature review of the behaviour of heaving clay during wetting under load indicated that the relationship between the heave strain and the logarithm of the stress applied during wetting is linear. This observation was confirmed in the

experimental programme conducted on Rosebank clay. No testing was performed under conditions of controlled suction and therefore the relationship between the heave strain and soil suction identified in the literature cannot be confirmed but it is assumed to hold.

The expansion behaviour of heaving clays was expressed through the state variable approach as a state surface relating the heave strain of the clay to the applied stress and the soil suction. This state surface was plotted in Figure 4.1 on page 41, and the function H describing the surface was presented as:

$$\epsilon_v^h = H \left(1 - \frac{\log_e(u_c)}{\log_e(u_{c0})} \right) \left(1 - \frac{\log_e(p - u_a)}{\log_e(p_h - u_a)} \right) \quad (6.13)$$

where ϵ_v^h is the heave strain;
 H is the *percent heave* (expressed as a volumetric strain);
 u_c, u_{c0} are the initial and final soil suctions;
 u_a is the pore air pressure;
 p, p_h are the current hydrostatic stress and the *heave stress* respectively.

This function was originally presented as equation (4.5), which expressed the void ratio e , and here has been changed to an express the heave strain ϵ_v^h . The expression can be simplified using the assumption that the pore air pressure in an *in situ* clay will generally be equal to atmospheric pressure and hence the term u_a in equation (6.13) can be ignored.

Equation (6.13) is assumed to apply to the Rosebank clay tested in the experimental programme, and therefore forms the basis of the constitutive equation for heave strain of expansive clay. The expression can be inverted directly to give the constitutive equation for heave as

$$p = p_h \left(1 - \frac{\epsilon_v^h}{H \left(1 - \frac{\log_e(u_c)}{\log_e(u_{cd})} \right)} \right) \quad (6.14)$$

(assuming that $u_a = 0$).

No yielding and plastic deformation has been observed during heave and therefore no yield function is required and only a heave bulk modulus K_h is required to complete the constitutive relations. The stress-strain relationship will again be non-linear, but an instantaneous value for the modulus can be obtained from the slope of the tangent to the stress-strain curve for a particular stress/strain value. The expression can be obtained from partial differentiation of (6.13) with respect to the stress p , which yields the stress-strain modulus for conditions of constant soil suction. Similar expressions can be developed to relate changes in heave strain to changes in soil suction at constant stress, and for relating changes in stress to changes in suction at constant volume. The stress/strain relationship for constant soil suction is:

$$K_h = \frac{\partial p}{\partial \epsilon_v^h} = - \frac{p \log_e(p_h)}{H \left(1 - \frac{\log_e(u_c)}{\log_e(u_{cd})} \right)} \quad (6.15)$$

which completes the constitutive relations necessary to describe heave.

6.4 Constitutive Expressions Describing the Relationship between Heave and Mechanical Consolidation

The heave and mechanical consolidation behaviour of Rosebank clay are analysed separately in this proposed constitutive model, but each of these aspects of the behaviour will influence the other. This section proposes

expressions for the effect of mechanical consolidation on subsequent heave, and the effect of heave on subsequent mechanical consolidation. Most of the proposed relationships are inferred from test results or else suggested as "possible" relations, because there is a paucity of experimental data on which to base the model.

6.4.1 Effect of Mechanical Consolidation on Subsequent Heave

The tests performed on Rosebank Clay indicate that after a cycle of loading, yielding, plastic deformation and unloading the *heave pressure* and *percent heave* of the clay were increased. The plastic deformation experienced by the clay has the effect of increasing its dry density by reducing the volume. Analysis of the test data of de Sousa Vinagre² suggested a linear relationship between the void ratio (and hence the specific volume: $v = 1 + e$) of the clay and the heave strain shown by the sample under a specific applied stress. These observations are the basis for the relations between change in *heave stress* and *percent heave* with changes in dry density of the material.

The relations were developed on the assumption that the change in the heave strain is related to the irreversible (plastic) volumetric strain experienced by the soil during a load cycle, which permanently changes the specific volume and dry density and therefore influence the heave parameters. It is therefore presumed that the effect of elastic strain on the specific volume will not change the heave parameters. The assumption is valid if the different pressures method is used to evaluate the heave parameters. As the sample is loaded to the pressure at which the heave will occur elastic strain will take place. The test therefore implicitly measures the effect of elastic changes in volume on the heave. Consequently only plastic volume changes will be shown to change the

volume is obtained from the heave strains of samples wetted at a 1 kPa load. This relation can be expressed as

$$H = M_h(v_0 - v) + H_0 \quad (6.16)$$

where H, H_0 are the current and initial (reference) percent heaves;

M_h is the gradient of H with respect to specific volume (expressed as a positive value);

v, v_0 are the current and initial (reference) unstressed specific volumes.

The reference values of H_0 and v_0 refer to the values for a sample which has experienced no plastic deformation.

A similar expression will exist between heave of samples at the applied stress value which will show zero heave strain at the reference specific volume. This is the reference *heave pressure* p_{h0} of the clay. If the sample undergoes plastic strain then the density of the clay will increase and the soil will exhibit heave when wetted at this applied stress (showing that the *heave pressure* has increased). The gradient M_p for the relationship between heave strain and specific volume under applied stress p_{h0} can therefore be established. The value of the *heave pressure* p_h for a particular specific volume will be:

$$p_h = p_{h0} \left(\frac{(v_0 - v)M_h + H_0}{(v_0 - v)(M_h - M_p) + H_0} \right) \quad (6.17)$$

Thus if the reference values H_0, p_{h0} , and v_0 and the gradients M_h and M_p are known then the *percent heave* H and *heave pressure* p_h for any other unstressed specific volume v can be established from equations (6.16) and (6.17).

6.4.2 Effect of Heave on Subsequent Mechanical Consolidation

If a sample of Rosebank clay is loaded mechanically after it has heaved it will have a new yield stress p_c^* and a new elastic logarithmic bulk modulus κ^* . The framework for these changes is indicated in Figure 6.6.

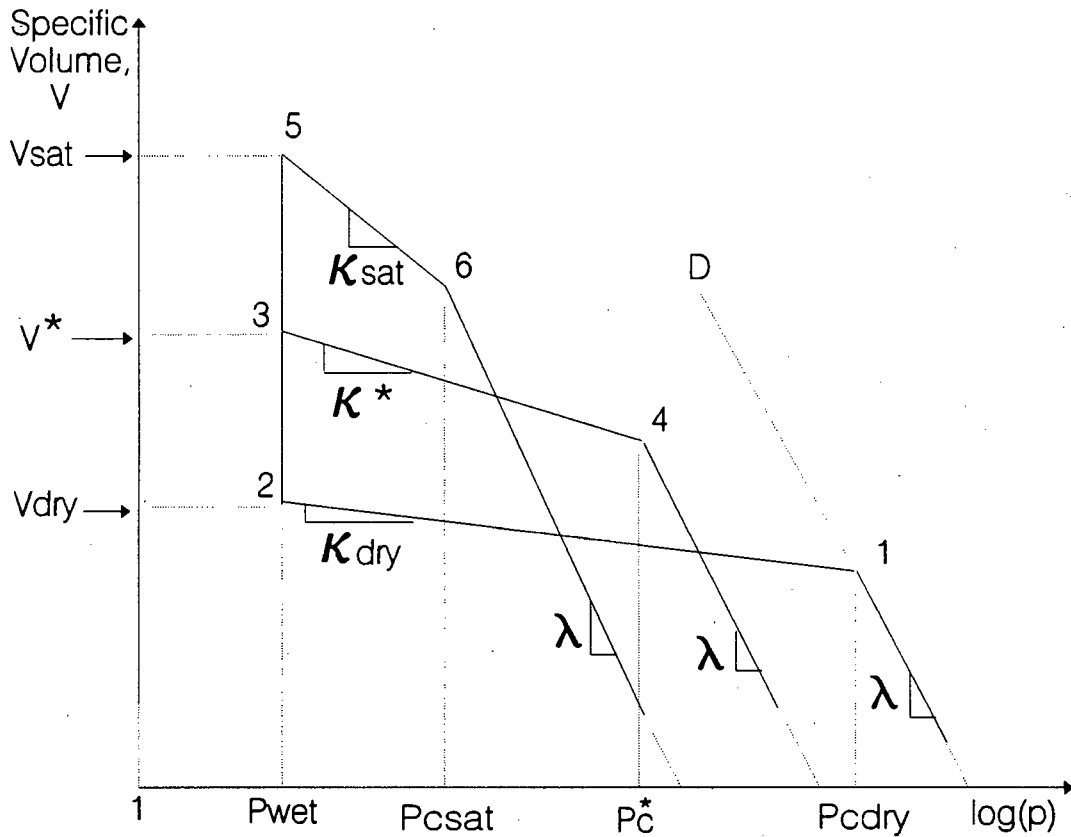


Figure 6.6 Effect of the degree of change of saturation on p_c and κ after heaving of a clay

The figure shows the changes in specific volume of a clay under three different unloading, wetting and reloading paths. The clay has been subject to loading at its unsaturated ("dry") moisture content along a path $D \rightarrow \textcircled{1}$ (not fully shown in Figure 6.6) which has ended at the material state defined by $\textcircled{1}$. The hydrostatic pressure in the clay is equal to the value of the yield stress p_{cdry} , and therefore the material state lies on the yield

surface. The clay is unloaded elastically along ①→② (slope κ_{dry}) to a stress p_{wet} where water is added and heave occurs. If sufficient water is added to fully saturate the soil the sample heaves to ⑤. Reloading of the sample occurs along an elastic loading line ⑤→⑥ with slope κ_{sat} to the new yield stress p_{csat} , after which the deformation is elastic-plastic and the line has a slope λ .

If the sample was unloaded to ② but not wetted at all before reloading then the values of κ and p_c would remain unchanged from κ_{dry} and p_{cdry} . Consequently the sample would retrace the path ②→① with a slope κ_{dry} to the original yield stress p_{cdry} where it would yield and then undergo elastic-plastic loading along a line with a slope λ .

These two cases reloading from point ② represent the limits of the material response under mechanical consolidation after a change in soil moisture at ②. The two limits are caused by the degree of change of moisture content at ②, being zero (path ②→①) and complete wetting (path ②→⑤→⑥). It is now necessary to consider an intermediate case where a clay sample is wet at ② but not sufficiently to reduce the soil suction to zero and fully saturate the soil. Consideration of the constitutive equation for heave developed above shows that the sample will heave from ② to ③ which is at a somewhat lower specific volume than ⑤. As the sample is loaded from ③ it is assumed to follow an elastic loading line with a slope κ^* which is intermediate between κ_{dry} and κ_{sat} . At some point ④ the sample will yield at a stress p_c^* which is also assumed to be intermediate between p_{csat} and p_{cdry} . No experimental data is available to indicate the nature of these two relationships and therefore they are both assumed to be logarithmically related to the soil suction. The assumed relationships are therefore

$$\kappa^* = \kappa_{dry} + (\kappa_{sat} - \kappa_{dry}) \left(1 - \frac{\log_e(u_c)}{\log_e(u_{c0})} \right) \quad (6.18)$$

and

$$p_c^* = p_{cdry} - (p_{cdry} - p_{csat}) \left(1 - \frac{\log_e(u_c)}{\log_e(u_{c0})} \right) \quad (6.19)$$

The factor

$$\left(1 - \frac{\log_e(u_c)}{\log_e(u_{c0})} \right) \quad (6.20)$$

is a scaling factor which adjusts the parameters to their boundary values according to a logarithmic rather than a linear scale. It thus necessary to establish the boundary values of the parameters κ_{dry} , κ_{sat} , p_{cdry} and p_{csat} . The modulus values can be established directly from the measurement of the slopes of the elastic loading lines on plots of test results. The "dry" yield stress p_{cdry} for an undisturbed sample can be measured directly from testing. If a loading/unloading stress path is applied to the sample then this may cause yielding and an increase in the yield stress which will be equal to the value of the highest stress applied. The parameter which is difficult to judge is p_{csat} , because experimental observations indicate that it is related to the stress p_{wet} at which water is added to the sample, and also to the degree of plastic deformation experienced by the sample during previous loading cycles. The number of tests performed on Rosebank clay was insufficient to establish the nature of the effect of plastic strain on p_{csat} . It does appear, however, that for Rosebank clay the ratio p_{csat}/p_{wet} is approximately 3.0 (for all cases except swelling at 1 kPa), and the ratio will increase if more plastic strain occurs before wetting.

6.5 Evidence in Literature which Supports the Proposed Model

The various constitutive relationships proposed above were developed to be consistent with the findings reported in literature and the experimental results. However, the fundamental assumption around which the model is based is that a clear distinction must be made between the volumetric response of the clay under wetting (heave), which is considered to be a chemical process, and the response under changes in load at constant moisture content (consolidation), which is considered to be a mechanical process. As such it is at variance with the test methods proposed by Chen⁴, Jennings and Knight⁵ and others, and with the assumptions of Bishop and Blight⁶ and Blight⁷ that the process is purely mechanical. In this section brief reference is made to some results reported in the literature which have not been discussed so far but which support the hypotheses of the model and confirm some of the test results.

1) Heave as a chemical process

Wittke⁸ discusses the swelling of rocks during wetting and refers to the heave of the clay mineral corrensite. The volumetric heave behaviour displayed by this clay while under load is similar to the behaviour of rocks which swell when water is added to them. These rocks do not contain clay minerals but swell because of a reaction between the rock material and the free water. Heave of the corrensite clay mineral is so closely similar to the chemical swelling of other rocks that Wittke considers the response of corrensite to free water to be essentially a chemical reaction.

2) Variation of bulk moduli before and after full wetting

The double oedometer test proposed by Jennings and Knight⁵ is based

on the observation that the elastic-plastic logarithmic bulk modulus λ is the same for partially saturated and fully saturated samples of the same clay. This result was confirmed in the tests on Rosebank clay, but it was noted that the elastic logarithmic bulk modulus κ had a significantly higher value in fully saturated material than in partially saturated material. A plot of several test results on a heaving clay from Vereeniging by Williams⁹ confirms that the elastic-plastic modulus λ is the same for saturated and unsaturated samples, but the elastic modulus κ is substantially higher for fully saturated samples than for unsaturated samples.

3) Response of clay to mechanical consolidation after wetting

The tests on Rosebank clay showed that wetting the clay to induce heave caused a significant change in the yield stress p_c . Stark and Duncan¹⁰ tested clay from the San Luis Dam, California. They noted that the value of the elastic-plastic modulus λ was the same for partially and fully saturated undisturbed samples and for saturated remoulded test specimens. They also noted that after soaking and heaving the value of the yield stress had changed from its value for unsaturated conditions to a value equal to the applied stress at wetting. The results of two of the tests are reproduced below in Figure 6.7.

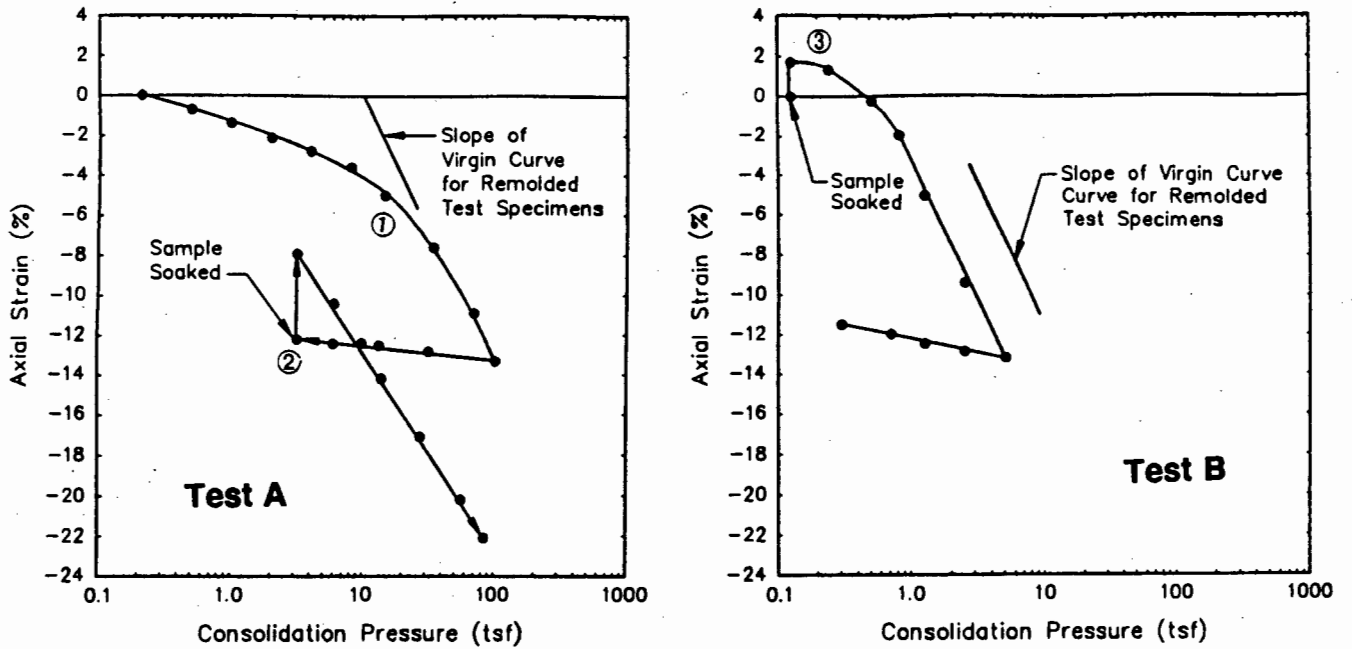


Figure 6.7 Test results on clay from San Luis Dam, California, by Stark and Duncan¹⁰

Their proposed explanation of the results is that the unsaturated clay exhibits a "psuedo yield stress" (at point ①). After unloading and wetting at point ② the memory for its stress history (ie p_c) disappears. Only when a sample was soaked at a very low load was there evidence of a discernable yield point (③), and the value of the yield stress corresponded with the overburden pressure that the clay had experienced in the field.

The tests on Rosebank clay also indicated that the "memory" of the clay for the yield stress before wetting was changed during heave. In the tests a yield stress after wetting was generally identifiable, unlike in Test A (see Figure 6.7). However, this could possibly be explained by the difference in load increment size used in these tests, where the maximum loading was 9580 kPa compared with 1700 kPa in the Rosebank tests.

This Chapter has presented the development of constitutive relations to describe the

volumetric behaviour of Rosebank clay during heave and mechanical consolidation. The basis of the model is that these two aspects of the behaviour are fundamentally different, the former being a chemical and the latter a mechanical process. Separate constitutive relations were developed for both processes, and then consideration was given to the effect of each process upon subsequent occurrences of the other. It was necessary to make certain unsupported assumptions about the material behaviour in order to complete the constitutive framework. Once this had been developed it was possible to proceed with numerical implementation of the model through the Finite Element Method, as discussed in the following Chapter.

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CHAPTER 7

NUMERICAL IMPLEMENTATION OF CONSTITUTIVE MODEL FOR ROSEBANK CLAY

This chapter describes the process of numerical implementation of the model developed in Chapter 6. Some aspects of the constitutive relations required modification in order to make them suitable for numerical implementation. Consideration had to be given to the numerical solution procedures that would be used to solve the constitutive relations.

The chapter begins with a discussion of the calculations required within a user defined constitutive model by the finite element package *ABAQUS*¹, which was the software used for implement the heaving clay model. Thereafter the user subroutine programmed for performing the constitutive calculations is described. Finally three test problems are presented in order to verify the numerical model. This gives an indication of the degree of success with which the model describes the experimentally observed behaviour of the clay.

7.1 Constitutive Calculations Required by *ABAQUS*

ABAQUS provides a facility for a user defined material constitutive model through a user programmed FORTRAN subroutine. Stresses, strains, temperatures and various solution dependent variables are passed into the subroutine containing their values at the end of the previous solution increment. Predicted values for the change in strain and temperature during the current solution increment are also provided. In the subroutine the user must define the value of the stresses and solution dependent variables at the end of the solution increment, based on the predicted strain and temperature changes passed into

the subroutine. The user must also define the "material Jacobian matrix" which is the array $\partial\Delta\sigma_{ij}/\partial\Delta\epsilon_{kl}$. This is used to form the next prediction of the change of strain during the increment.

An incremental solution procedure is used whereby the algorithm iterates within each solution increment until the force residual in the finite element mesh complies with an acceptable tolerance. This implies the use of a full or modified Newton-Raphson solution algorithm^{2,3}. The rate of convergence of the iterative procedure is dependent on the accuracy with which the material Jacobian matrix $\partial\Delta\sigma_{ij}/\partial\Delta\epsilon_{kl}$ is defined. Convergence rates for the material model are discussed in §7.3.

Implementation of the constitutive relations developed in Chapter 6 requires consideration of three factors. Firstly, all the constitutive relations developed involved hydrostatic stresses and volumetric strains, without consideration of the deviatoric components of the stress and strain tensors. A method for defining the deviatoric stresses in terms of the deviatoric strains is required to satisfy equilibrium requirements in a finite element model. Secondly, a means of defining values for the soil suction during the analysis is required. Thirdly, the state variable surface defining the heave strain must be expressed in a form which is compatible with the incremental approach adopted by the algorithm in generating the solution.

7.1.1 Constitutive Relations for Deviatoric Stresses and Strains

The total stress tensor σ_{ij} can be divided into hydrostatic and deviatoric components through the relation

$$s_{ij} = \sigma_{ij} - p\delta_{ij} \quad (7.1)$$

where s_{ij} is the deviatoric stress tensor;
 p is the hydrostatic stress: $p = 1/3 (\sigma_{11} + \sigma_{22} + \sigma_{33})$;
 δ_{ij} is the Kronecker delta.

Similarly, the strain tensor ϵ_{ij} can be divided up into deviatoric and volumetric parts through

$$e_{ij} = \epsilon_{ij} - \frac{1}{3}\epsilon_v \delta_{ij} \quad (7.2)$$

where e_{ij} is the deviatoric strain tensor;
 ϵ_v is the volumetric strain: $\epsilon_v = (\epsilon_{11} + \epsilon_{22} + \epsilon_{33})$;

Increments in the hydrostatic stress are related to increments in the volumetric strain by the bulk modulus \mathbf{K} , and increments in the deviatoric stresses and strains are related by the shear modulus \mathbf{G} .

$$\dot{p} = \mathbf{K} \dot{\epsilon}_v \quad (7.3)$$

$$\dot{s}_{ij} = 2\mathbf{G} \dot{e}_{ij} \quad (7.4)$$

The moduli \mathbf{K} and \mathbf{G} are hypo-operators which are dependent on whether the material is undergoing elastic or elastic-plastic deformation.

The shear modulus can be expressed in terms of the bulk modulus if a Poisson's ratio ν is specified for the material:

$$\mathbf{G} = \frac{3(1 - 2\nu)\mathbf{K}}{2(1 + \nu)} \quad (7.5)$$

Two methods of incorporating the deviatoric stress and strain into the analysis are therefore possible. The first involves specifying a constant value for \mathbf{G} and calculating all deviatoric stresses on the basis of this

constant. However, the value of \mathbf{K} is not constant but gets progressively stiffer as p increases. A constant value of \mathbf{G} will apply only to a limited range of stresses.

The second method of dealing with deviatoric stresses and strains is to specify a constant Poisson's ratio ν for the material, so that \mathbf{G} will change in constant proportion to \mathbf{K} . This was the method used for treating the deviatoric stresses and strains in the numerical model.

This is the simplest means of dealing with the deviatoric stresses and strains, and was implemented to satisfy equilibrium requirements in the numerical model. However, this method does not model the shear behaviour of expansive clay accurately. Heaving clay will exhibit a frictional resistance to shear⁵ and will undergo plastic shear failure at high shear stresses. Simulation of this kind of material response requires the use of some form of critical state model^{5,6}. The shear equations adopted in this model describe only elastic shear response and do not incorporate shear failure. Consequently the model cannot be used in problems involving a high degree of shearing, meaning that there is a significant limitation upon the type of problems that can be analysed.

7.1.2 Incorporation of Soil Suction into the Constitutive Relations

ABAQUS has a procedure for solving partially saturated flow problems in terms of pore pressure as the state variable and the saturation as the flux, making it possible to solve for soil suction. However, when this procedure is invoked the effective stresses are passed into the user constitutive subroutine. These are defined by the principle of effective stress for partially saturated soils (see equation (4.2) on page 36). A total stress formulation was required which meant choosing another state

variable to represent the suction. The temperature is a direct analogy of pore pressure state variable, so this was chosen to replace the soil suction u_c in the constitutive model.

For the experimental investigations of Rosebank clay no soil suction measurements were made during the analysis and there is no information available to define the relationship between pore pressure and soil saturation for the clay. If it were possible to use the pore pressure value defined by *ABAQUS* the quality of the parameters defining the response of the partially saturated soil would be very poor and would be representative only. Therefore use of the temperature as a substitute variable for soil suction is unlikely to effect the quality of the solution that will be obtained.

7.1.3 Incremental Formulation of the Heave Strain State Function

The incremental solution procedure used by *ABAQUS* requires a method for using the heave strain state function to solve incremental heave strains. This modification to the surface should result in consistent results for the total heave strain irrespective of the number of increments taken to reach the final solution.

The material state point on the heave surface is always defined by the values of the state variables p and u_c . The state point at the beginning of any solution increment n can be defined by the values of p_{n-1} and u_{cn-1} at the end of the previous increment.

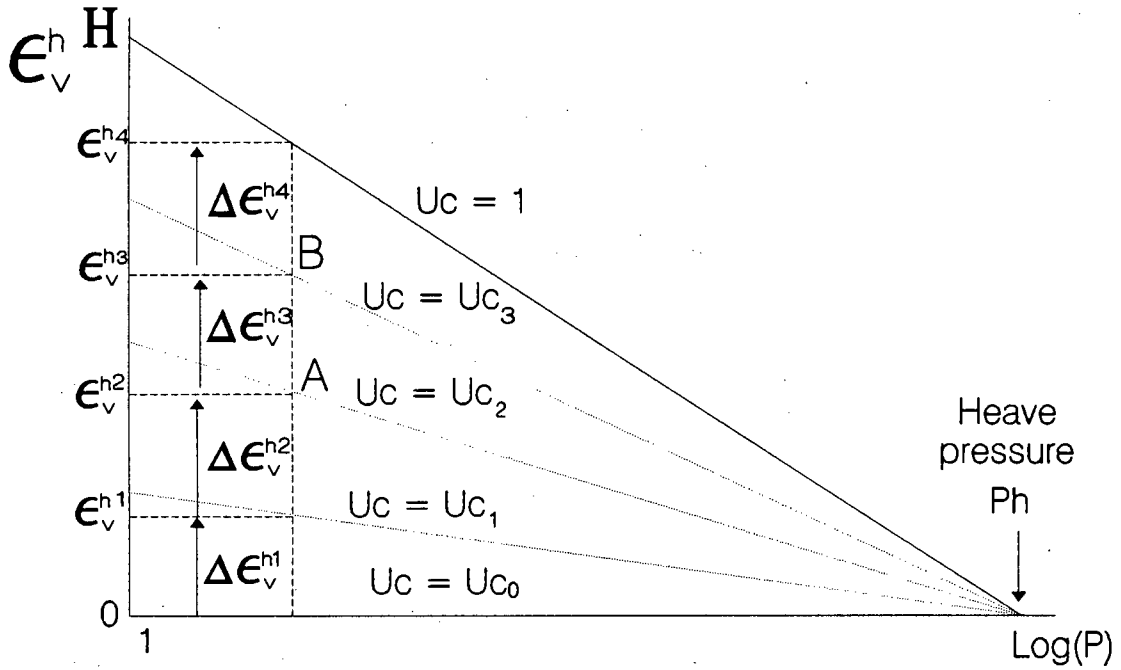


Figure 7.1 Method for solving the heave strain of a clay incrementally

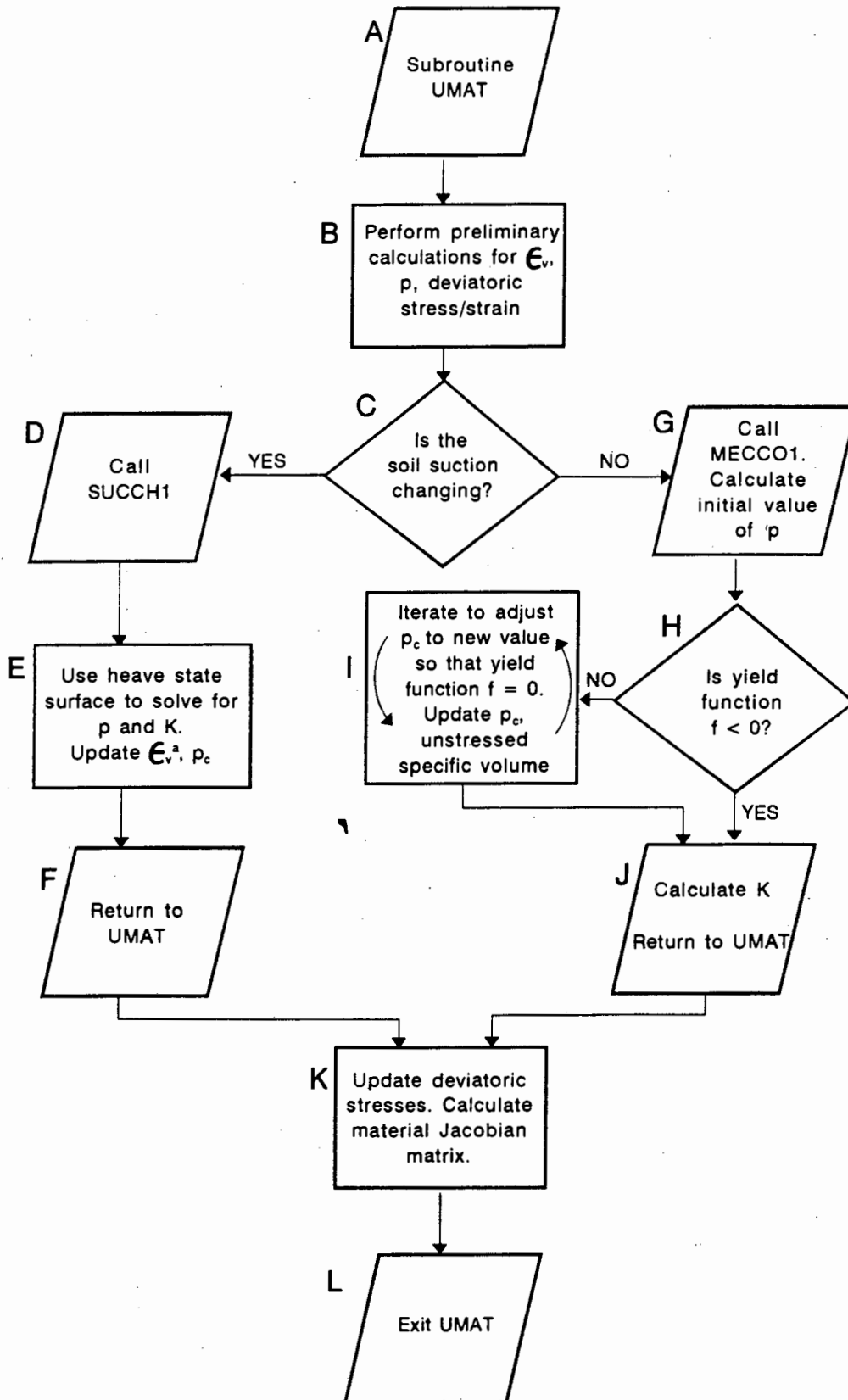
Figure 7.1 indicates the incremental solution procedure. A clay subjected to an applied stress p has undergone several incremental changes in soil suction. At the end of a previous increment the soil suction was, say, u_{c2} . The stress and suction conditions at the end of the increment defined the heave state point to be A, with a total heave strain of ϵ_v^{h2} . The soil suction at the end of the current increment is u_{c3} . The material state point at the end of the increment is therefore represented by point B, with an associated total heave strain of ϵ_v^{h3} . The current heave strain increment is therefore the difference between points A and B, and will be $\Delta\epsilon_v^{h3}$.

7.2 Description of the User Subroutine Defining the Heaving Clay Constitutive Behaviour

This section describes the structure and solution algorithm of the FORTRAN subroutine which defines the constitutive model of the clay behaviour for use by *ABAQUS*. The source code is included in Appendix II.

Box 1 contains a flowchart for the subroutine indicating its various parts. The subroutine UMAT is called from the main program which passes in the values of strain and stress tensors, the temperature and the solution dependent variables yield stress, p_c , "adjustment strain", ϵ_v^a , and "unstressed specific volume", v at the end of the previous increment. The predicted values for the change in strain and temperature in the current increment are also passed in. The invoking of the subroutine is shown by "A" in the flowchart.

After the subroutine is called it performs preliminary calculations ("B" in the flowchart) to establish the first stress and strain invariants from the tensors, from which the values of p and ϵ_v are derived. The routine then checks whether the soil suction (temperature) is changing during the increment (C). If YES, this implies a change of soil suction and thus heaving will occur. A further subroutine SUCCH1 (D→E→F) is called which performs the constitutive calculations for heave. It calculates the value of p based on the change in volumetric strain $\Delta\epsilon_v$ during the increment, and also returns a value of the bulk modulus $K = \partial\Delta p / \partial\Delta\epsilon_v^h$. The equations used in this routine are presented in §6.3.



Box 1: Flowchart for FORTRAN subroutine of Rosebank clay constitutive model

The parameters of *percent heave*, H , and *heave pressure*, p_h , are dependent on the degree of plastic strain that has been experienced by the material in previous mechanical consolidation steps. This plastic volumetric strain is represented by the current unstressed specific volume v which can be compared with the initial specific volume v_0 .

As heave occurs it alters the yield stress p_c for subsequent mechanical consolidation, as discussed in §6.4. The "adjustment strain", ϵ_v^a , must be altered as p_c changes. These parameters are updated in the subroutine.

If "C" detects that there is NO change in soil suction (temperature) during the increment then subroutine MECCO1 is called which performs the constitutive calculations for mechanical consolidation. At stage "G" the hydrostatic pressure p in the material is calculated from the predicted value of the volumetric strain for the current increment and the value of p_c for the end of the previous increment. This is used to calculate the value of the yield function f at "H". If this is less than or equal to zero then no violation of the yield criterion has occurred and the calculated value of p can be passed to stage "J" and back to the calling routine.

However, if the yield function f is greater than zero then the yield criterion has been violated. This means that yielding and plastic strain have taken place which will cause hardening. It is therefore necessary to solve for the new value of p_c . This is done by a full Newton-Raphson solution procedure which is formulated as follows:

The current value of the yield function is f^i . This is greater than zero and thus it is necessary to apply a correction to the values of p and p_c to update the value of f .

$$f^i(p, p_c) = p^i - p_c^i \quad [(6.10) \text{ bis}] \quad (7.6)$$

The incremental form of f is:

$$\dot{f}^i = \dot{p}^i - \dot{p}_c^i \quad (7.7)$$

The value of p is calculated from

$$p = \exp\left(\frac{\epsilon_v^a - \epsilon_v - (\lambda - \kappa) \log_e(p_c)}{\kappa}\right) \quad [(6.9) \text{ bis}] \quad (7.8)$$

and therefore the incremental form of p is:

$$\dot{p}^i = -\frac{(\lambda - \kappa) p^i}{\kappa p_c^i} \dot{p}_c^i \quad (7.9)$$

The current value of the yield function f^i is greater than zero. A second estimate of the yield function f^{i+1} is required such that $f^{i+1} = 0$. This second estimate for f is f^{i+1} :

$$f^{i+1} = f^i + \dot{f}^i \quad (7.10)$$

Equations (7.10), (7.9) and (7.7) give the new estimate of f as:

$$f^{i+1} = f^i - \left(\frac{(\lambda - \kappa) p^i}{\kappa p_c^i} + 1\right) \dot{p}_c^i \quad (7.11)$$

The value of f^{i+1} must be 0 to satisfy the yield criterion so p_c will be:

$$\dot{p}_c^i = \frac{f^i}{\left(\frac{(\lambda - \kappa) p_c^i}{\kappa} + 1 \right)} \quad (7.12)$$

The new estimate for p_c is p_c^{i+1} :

$$p_c^{i+1} = p_c^i + \dot{p}_c^i \quad (7.13)$$

Solving for p_c involves an iterative procedure during which successive estimates of p_c^i are made until the yield function becomes sufficiently close to zero and the yield criterion is satisfied.

An increase in p_c implies that there has been an increase in the plastic strain and a change in the unstressed specific volume v . This is updated in part "I" of the subroutine (see flowchart in Box 1).

Part "J" of the subroutine calculates the value of $\mathbf{K} = \partial \Delta p / \partial \Delta \epsilon_v$. This will be the elastic-plastic bulk modulus if plastic strain has occurred and the iterative procedure "I" was invoked, or the elastic bulk modulus if the yield function was found to be less than or equal to zero in "H".

Part "K" of the subroutine represents the final calculations by the subroutine UMAT in which the deviatoric stresses are updated based on the updated hydrostatic stresses. The material Jacobian matrix $\partial \Delta \sigma_{ij} / \partial \Delta \epsilon_{ij}$ is calculated from the bulk modulus \mathbf{K} returned from one of the constitutive subroutines. The subroutine UMAT then terminates, passing out values of the stress and solution dependent variables for the end of the increment and a new value of the material Jacobian matrix for making the next strain prediction.

7.3 Stability and Convergence of the Solution Procedure

No rigorous mathematical analyses were performed to check the stability of the constitutive model or the solution procedure. However, the material shows strain hardening and hence a monotonic increase in strain always gives a monotonic increase in stress. Also, for any given strain value the value of the elastic bulk modulus is greater than the elastic-plastic bulk modulus. These two criteria conform with Drucker's first and second stability postulates and indicate material stability.

The convergence of the solution procedure is influenced by the use of the consistent tangent modulus in the incremental solution procedure. Cook, Malkus and Plesha³ suggest that the strain hardening shown by the material during both elastic and plastic material response will make convergence of the iterative solution procedure slower and could result in failure of convergence. They suggest that the rate and likelihood of success of convergence will be improved by use of the full Newton-Raphson solution procedure rather than a modified method. This influenced the decision to use a full Newton Raphson solution procedure in the numerical implementation of the constitutive model.

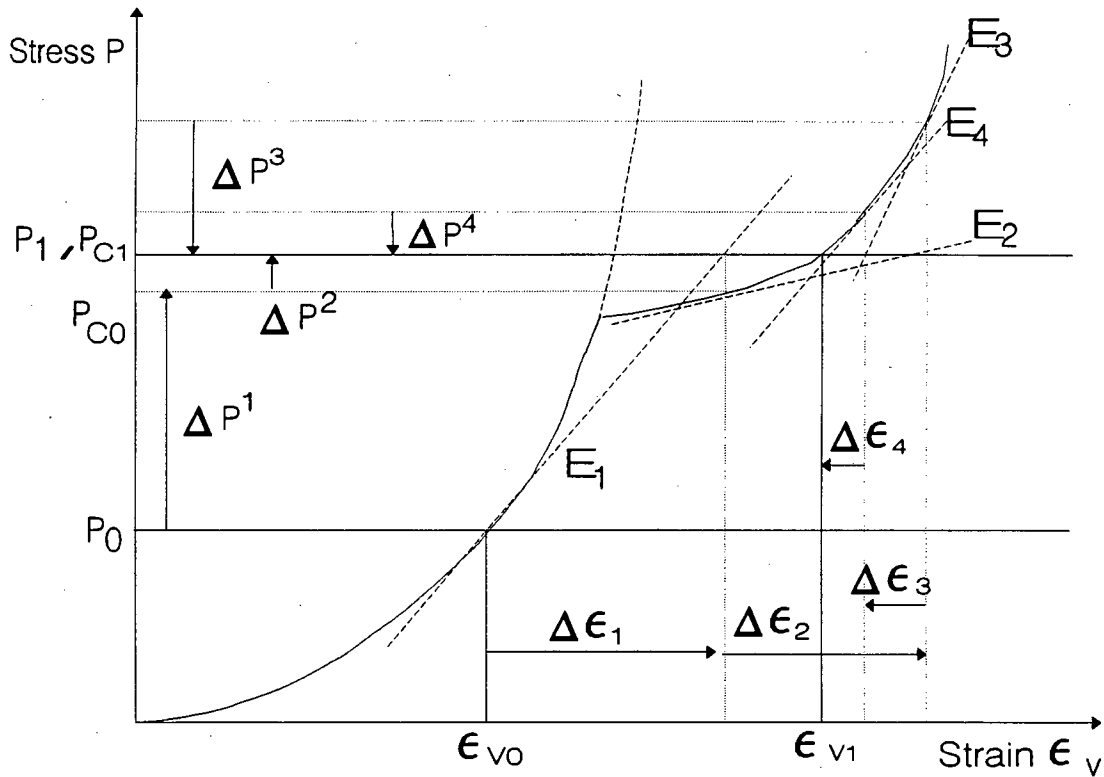


Figure 7.2 Newton-Raphson solution procedure for a stress increment during mechanical consolidation of a clay

Figure 7.2 shows the progress of the Newton-Raphson solution procedure in solving a stress increment from a stress p_0 and strain ϵ_{v0} to a stress p_1 and strain ϵ_{v1} . The initial yield stress p_{c0} is less than p_1 and therefore yielding will occur during the increment. The initial strain prediction is based on the elastic tangent bulk modulus E_1 and the stress difference Δp^1 . This gives an initial strain prediction of $\Delta \epsilon_1$. The stress value calculated from the constitutive equation shows that this strain increment gives an under-prediction of the stress by Δp^2 , and thus a second strain increment is predicted on the basis of Δp^2 and E_2 , which is the elastic-plastic tangent bulk modulus. The constitutive equation shows that the strain increment $\Delta \epsilon_2$ gives an over-prediction of the stress by Δp^3 . The solution then converges back to the correct solution through strain increments $\Delta \epsilon_3$, $\Delta \epsilon_4$ and so on.

The rate of convergence of the solution will be relatively slow because it involves an over-prediction of the stress and strain and convergence back to the correct solution. During the test problems run on *ABAQUS* the time incrementation controls had to be adjusted to force the algorithm to perform more iterations for each solution increment before checking for convergence. In a 144 element problem the solution converged fairly rapidly initially but the rate decreased as the force residual decreased. Convergence was eventually accepted on the basis of a less stringent tolerance on the residual.

The hardening of the material with strain decreased the rate of convergence of the solution and showed that the numerical implementation is relatively expensive computationally. However, ensuring rapid convergence rates was of secondary importance to the accuracy with which the numerically implemented constitutive relations modelled the experimentally observed behaviour.

7.4 Controls on the Numerical Solution Procedure

It is important to ensure that the operation of the computer model is sufficiently stable to prevent numerical errors from causing a breakdown in the solution procedure. Spurious numerical values sometimes develop at a few gauss points in a finite element mesh, and if the numerical algorithm is unstable then these few gauss points may make a solution to the problem impossible to generate, despite the fact that the contribution solution values at these points may be insignificant to the problem as a whole. For this reason several numerical checks are performed in the subroutine in order to improve the robustness of the constitutive model.

7.4.1 Warnings for Small or Negative Values

Many of the constitutive equations in the model are based on logarithmic relationships. This leads to potential instabilities when the values entered into the equations are very small or negative, because the logarithm of zero or negative values does not exist. It is thus important that the stress and suction (temperature) values in the finite element mesh remain non-zero and positive. When the numerical model detects a value of the stress or suction which is smaller than 0.333 then a warning message is given indicating that this has occurred. The execution of the subroutine continues, although this may lead to spurious results or numerical problems in the solution procedure. However, the user has been warned that the results of the analysis may be poor.

The bulk modulus K is related to the current hydrostatic stress value and therefore very low or negative values of this stress will lead to low or negative values of K and numerical problems during the solution of the global stiffness matrix. Hence the value of K is always calculated using a stress value of 1 or greater (even if the stress value at the material point is less than this). This prevents numerical problems due to poorly defined values of K , but does not ensure accuracy of the overall solution.

7.4.2 Warnings for Improper Suction Changes

The constitutive model has been developed only for monotonically decreasing values of the soil suction. If the subroutine detects that there has been an increase in the suction at some point then it gives a warning message indicating that an improper change in suction has occurred.

7.4.3 Values which Lie Outside the Limits of the Model Definition

The testing described in the literature has generally involved clay which is wetted under applied loads which are lower than the heave pressure. The constitutive equations hold for wetting at stresses greater than the heave pressure, but will predict a collapse in the soil under these conditions. This prediction may be reconcilable with observed material behaviour in some tests reported in literature⁷. It can be said, however, that if such conditions are experienced during a finite element analysis the algorithm will remain stable and a solution will still be obtained, but the accuracy with which the solution will reflect the true material response is uncertain.

Changes in soil suction alter the yield stress of the material for subsequent mechanical consolidation. Under certain stress and soil suction changes the new value of the yield stress that is calculated may be less than the current value of the stress at the point which is inadmissible because then the yield function f would evaluate to a positive value. A check is therefore made when adjusting the yield stress to ensure that the yield stress value is always greater than the current material stress value. If this is not true then the yield stress is set equal to the material stress.

7.5 Problems Analysed to Test the Constitutive Model

Three test problems are presented here to test the numerically implemented constitutive model. The first two are single element simulations of an oedometer test which were run to compare experimental oedometer results with computational results. The third is a larger problem which models the doming

of heaving clay that occurs under slabs, and is an example of a typical problem that would be analysed by a practising engineer dealing with heaving material.

The numerical model requires 12 material constants which are given by the user in the problem input. The values of the constants were derived from the test data for the experiments conducted on Rosebank clay. A description of each constant, along with the value used in the analyses, is given in Table 7.1.

Table 7.1 Material constants that must be specified by the user of the heaving clay constitutive model

Material Properties Required for Constitutive Model		
No	Description	Value
1	Logarithmic elastic-plastic bulk modulus λ	0.02736*
2	Dry logarithmic elastic bulk modulus κ_{dry}	0.001917*
3	Saturated logarithmic elastic bulk modulus κ_{sat}	0.01601*
4	Poisson's ratio ν (assumed)	0.3
5	Initial yield stress (must be a negative value)	-452.0 kPa
6	Initial soil suction (temperature)	1000.0 kPa
7	Percent heave strain at initial specific volume H_0	0.057
8	Heave pressure at initial specific volume p_{h0}	425.0 kPa
9	Gradient of percent heave versus ν : M_h	0.3123
10	Gradient of heave pressure versus ν : M_p	0.1138
11	Reference sample specific volume: v_0	1.9
12	Factor relating change in p_c on wetting to ν	0.0

NB: the values marked * have been calculated for natural base logarithms which is why they differ from the values indicated in chapter 5 where base 10 logarithms were used.

7.5.1 Oedometer Test Simulations using a Single Element

Two oedometer tests were simulated. The first involved loading the clay to 100 kPa and then saturating it to induce heave. The load was increased to 1700 kPa before unloading. This simulated test series 1 conducted on the Rosebank clay. Test series 2 was also simulated, involving loading the clay to 1700 kPa and unloading to 100 kPa before wetting to induce heave.

The finite element mesh consisted of a single 8-noded reduced-integration plane strain element of unit dimensions. It was constrained in the horizontal direction and in the vertical direction along the base. Loads were applied to the top of the element. These are the boundary conditions for an oedometer test. The input decks for both problems can be found in Appendix III.

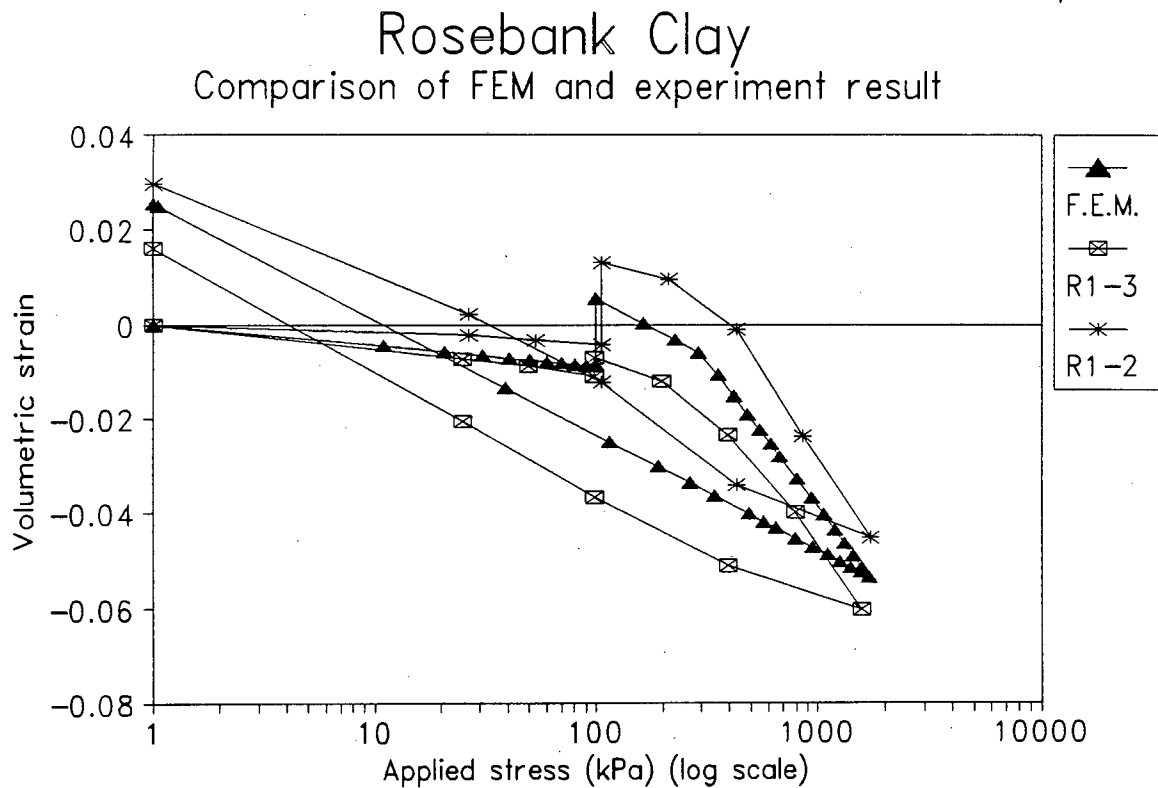


Figure 7.3 Comparison of experimental and finite element method (FEM) results for test series 1 on Rosebank clay

Figure 7.3 compares the results of the finite element method (FEM) analysis with the results of tests R1-1 and R1-2. The input parameters for the FEM analysis were based on average values taken over all six test series. However, the comparison of the general material behaviour is reasonable and the FEM results seem to agree with the experimental results both qualitatively and quantitatively.

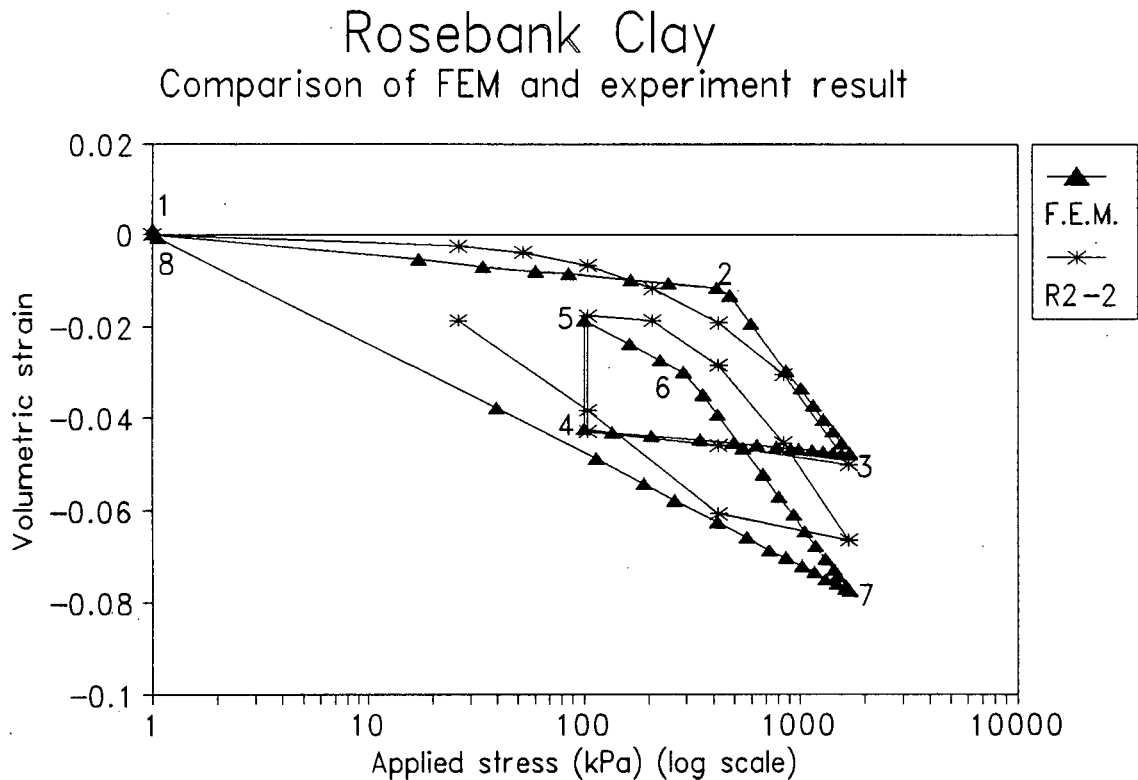


Figure 7.4 Comparison of finite element method (FEM) and experimental results for test series 2

Figure 7.4 compares the results for test number R2-2 and a finite element simulation for test series 2. The agreement between the experimental and actual results is very close along the first section of the curve (points ①, ②, ③, ④ and ⑤). Point ② represents the yield point, and this can be seen to be an idealisation of the behaviour because the experimental result shows a more gradual change from elastic to plastic behaviour than the abrupt change at ② in the numerical result.

At point ④ water is added and induces heave to ⑤. From ⑤ to ⑥ and ⑦ the curves diverge and the agreement is less close. This can be ascribed to the assumption that the material would deform elastically at the saturated elastic logarithmic bulk modulus κ_{sat} until a new yield point is reached. The plot shows that this assumption leads to a solution which is

reasonably close to the experimental solution but is an over-simplification of the material response and gives results which are less accurate than for other parts of the analysis.

7.5.2 Simulation of Doming of Heaving Clay under a Slab

In open field conditions the surface of a heaving clay profile is exposed to desiccation by the action of evaporation from the sun and wind and transpiration by plants. This causes the upper soil layers to become partially saturated and a soil suction develops. When a slab is laid over the soil this forms a membrane which prevents moisture loss. Water percolating up through the soil by capillary action becomes trapped under the slab and there is a build up of soil moisture and a reduction in suction, which results in heave. Some evaporation still occurs at the edges of the slab and therefore the greatest concentration of water develops under the centre of the slab, and the greatest amount of heave occurs here. The result is a the development of a characteristic dome under the slab, causing differential movements and possible cracking of building elements.

A finite element mesh of 144 elements was set up for a heaving clay profile with a water table 6 m below the surface. The soil suction at the surface was arbitrarily chosen as 1000 kPa. A slab of 6m width was analysed, although symmetry of the problem made it necessary to model only half of the width.

The analysis proceeded in two stages. The first was a heat transfer problem was analysed to solve for the temperatures (suctions) in the soil profile. The boundary conditions used were a temperature of 1000° along the uncovered soil surface and 1° along the ground water table and

under the slab. These temperatures represent the soil suction in kPa.

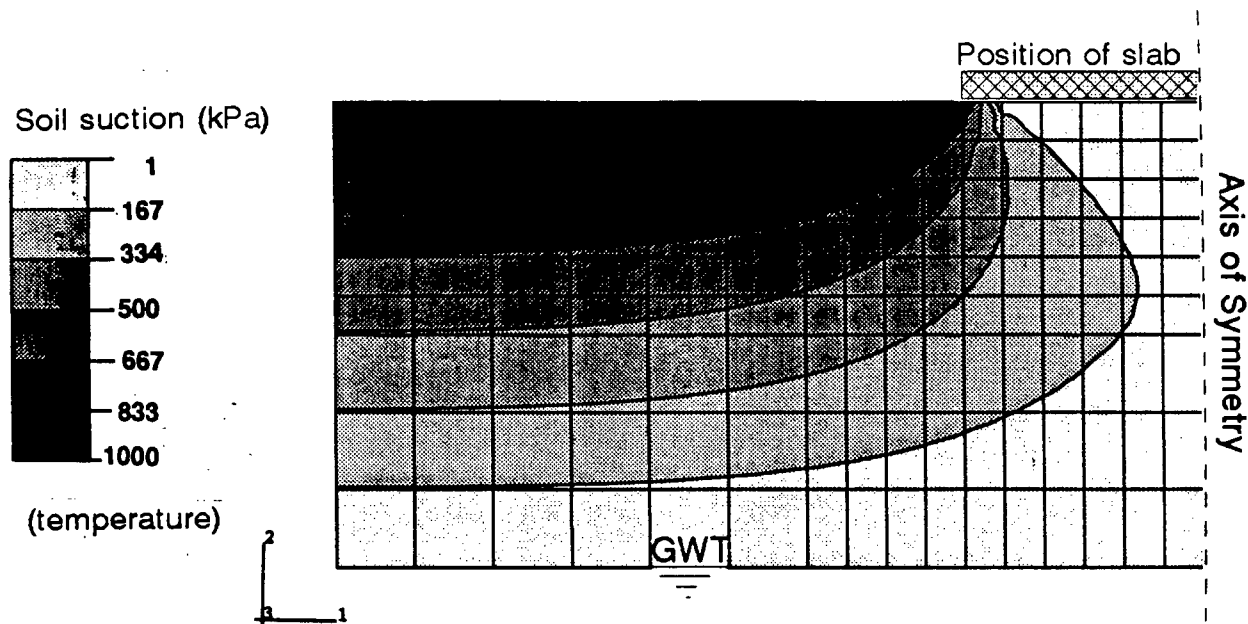


Figure 7.5 Soil suction (temperature) contours in a heaving clay profile partially covered by a slab

Figure 7.5 is a contour plot of the soil suctions (temperatures) that develop in the soil profile. The plot shows an 11 m wide portion of the clay profile. The slab covers 3 m of the profile on the right hand edge (only half of the slab is modelled because of symmetry). The left edge of the profile shows the suction contours that would exist through a profile in natural conditions where vegetation grows at the surface. The right edge shows the suction contours under the centre of the slab. The build up of moisture under the centre of the slab relative to its edges can be seen in the plot.

The second stage of the analysis solved for stress conditions and displacements. An initial step was used to apply geostatic loading conditions to the soil. The results of the temperature analysis were then used as input to the stress analysis procedure in order initiate heave as

a result of suction changes.

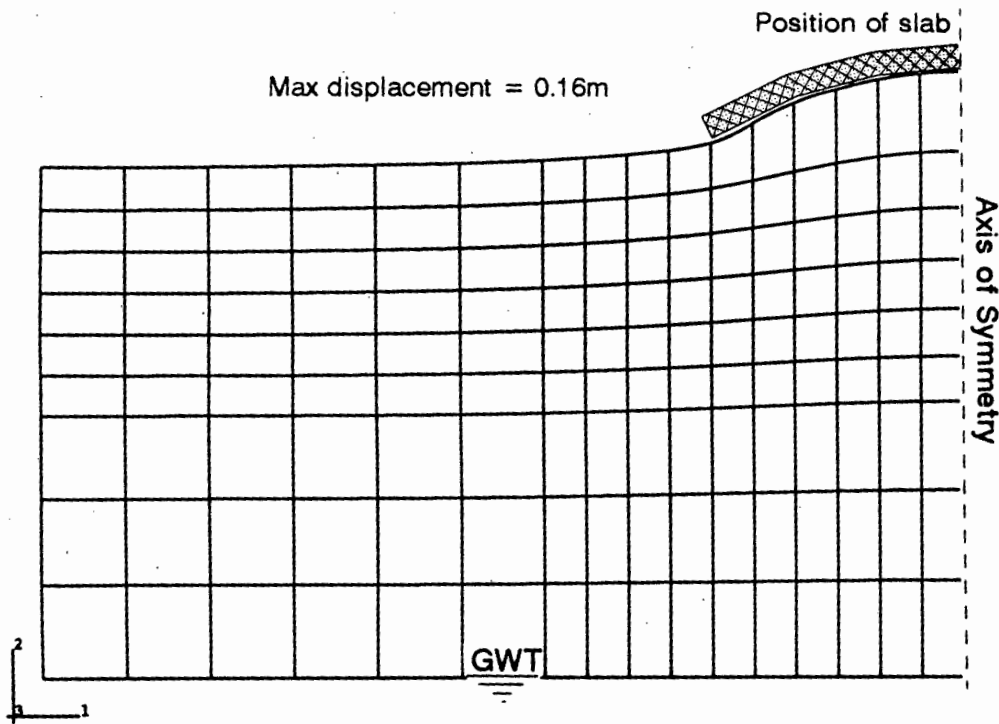


Figure 7.6 Displaced shape mesh for the model after heave has occurred

Figure 7.6 shows the domed shape of the soil profile after heave is complete. It is difficult to evaluate the results of this analysis through comparison with field observations because of a lack of suitably detailed field observations. Williams, Pidgeon and Day⁴ show an empirical method for calculating the mound shape of the dome, using the formula

$$y = y_m \left(\frac{x}{e_m} \right)^m \quad (7.14)$$

- where
- y is the vertical displacement of the mound profile;
 - y_m is the maximum heave displacement at the centre of the dome;
 - x is a horizontal distance from centre part of slab towards the edge;
 - e_m is an edge moisture variation distance;

element method are that the mound shape can be analysed for the case where the slab is loaded, and the interaction between the clay dome and the slab can also be modelled if desired. Once a reasonable degree of confidence in the finite element results is established from comparison with field observations the method can be used to establish better estimates for the empirical factors m and e_m in terms of the slab dimensions and depth to the water table. This would give more accurate results for the empirical method and provide a cheaper and simpler method for calculating the dome profile.

This chapter has described the numerical implementation of the constitutive model that was proposed for Rosebank clay, and the results of some numerical analyses that were performed to test it. This represents the fulfilment of the aims of the research project. It now remains to evaluate the results of the research work and to present final conclusions.

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CHAPTER 8

DISCUSSION OF RESEARCH WORK AND CONCLUSIONS

The work performed in this research project began with a literature review of the observed behaviour of expansive clay. An experimental programme was instituted to determine the response of Rosebank clay to cycles of loading and unloading because of a lack of information available in the literature. The results of the literature survey and the experimental programme were combined into a constitutive model for the volumetric behaviour of the clay under mechanical loading and changes in soil moisture conditions. The model was then implemented numerically into the finite element method, and some test problems were analysed to check the operation of the model and demonstrate its usefulness to a designer.

This Chapter evaluates the research work. The various different stages of the work are discussed to assess the value of results from each. Areas where further investigation is required are identified. The Chapter closes with an assessment of the research project as a whole, and presents final conclusions based on the work performed.

8.1 Results of Literature Survey

The emphasis of the work reported in the literature has been on the volumetric behaviour of expansive clay under load while it is wetted. This led to the establishment of the parameters *heave pressure* and *percent heave* which describe the heave strain that a clay will undergo when wetted under a particular applied load. It became apparent from the literature that the volume of a heaving clay is dependent on the path of stress and moisture changes imposed upon it. This meant that different test procedures would yield different

values for the *heave pressure* and *percent heave*, and therefore it was important to identify a consistent test procedure for measuring them. The "different pressures method" is the best method described in this thesis because it evaluates only volumetric changes while the soil moisture content is changing and does not measure the effects of subsequent mechanical consolidation on the heave.

The various factors which influence the magnitude of the heave strain could be identified once a consistent testing method for the measuring the heave parameters had been established. These factors were the unstressed void ratio (or specific volume) and the soil moisture conditions.

A state surface relating the heave strain to the state variables of applied stress and soil suction was developed on the basis of results from the literature survey. This state function was ultimately used as the constitutive equation to express heave behaviour.

The literature surveyed suggested that heave is a chemical process resulting from the mineralogy of heaving clays. The process can be thought of as a chemical reaction between the clay minerals and free water. This was shown to change the mechanical properties of the clay. One property that was identified in the literature as being significantly effected was the shear resistance, which decreased significantly after wetting of the clay. Consequently it is important to consider volumetric behaviour due to mechanical consolidation separately from volume changes during heaving, and to consider the effects of wetting on subsequent mechanical behaviour.

8.2 Results of the Experimental Programme

The experimental programme was designed to investigate the effects of cyclic loading on the behaviour of Rosebank clay. The effects of mechanical consolidation on subsequent heave strain, and of heave on subsequent mechanical consolidation were investigated.

Changes in the moisture content of the soil, which induce heave, were found to effect the subsequent mechanical consolidation behaviour of the clay significantly. The elastic logarithmic bulk modulus of the clay κ increases after wetting has occurred, but the elastic-plastic logarithmic bulk modulus λ is not effected. The apparent yield stress of the clay also changed during wetting, and subsequent values were more closely related to the applied stress at which wetting occurred than the value of the yield stress before wetting.

Plastic strain due to mechanical consolidation was found to influence the magnitude of subsequent heave movements, changing the values of the *heave pressure* and *percent heave*. The yield stress of the material after wetting also seemed to be influenced by the plastic strain that occurred before wetting, but insufficient test results were available to quantify the nature of the effect.

The test programme therefore gave important insights into the material behaviour. However, relatively few tests were performed and more experimental work is required to investigate the behaviour of heaving clay more fully. The following aspects merit further investigation:

- 1) Work needs to be carried out on different types of heaving clay in order to establish which aspects of the observed behaviour are specific to Rosebank clay and which can be generalised for all heaving clays.
- 2) The effect of plastic strain before heave on the mechanical consolidation

behaviour of the clay after heave needs further investigation in order to describe and quantify the nature of the effect.

- 3) The shear behaviour of Rosebank clay needs to be investigated in order to establish the effect of heave on subsequent mechanical shear strength.
- 4) Further suction controlled experiments are required to determine the response of a clay to a greater range of loading and wetting test paths. In particular, the effect of cyclic changes in soil suction needs to be investigated, as the proposed constitutive model is only valid for monotonically decreasing soil suctions.

8.3 Discussion of the Constitutive Model Developed for Rosebank Clay

A constitutive model for Rosebank clay was proposed which treated the volume changes due to heave separately from volume changes due to mechanical consolidation, but incorporated the effects each process upon the other. This led to a constitutive model which can describe the effects of different stress and wetting paths on the sample volume. This is significant because it explains the different values of the heave parameters obtained from different test methods, and provides an understanding of the overall volumetric response that can be expected from heaving clay.

The model is based on a number of assumptions. Firstly, lack of experimental data meant that some aspects of the behaviour had to be postulated in terms of "likely" relationships. The effect of changes in soil suction on the subsequent yield stress and elastic logarithmic bulk modulus was known only for the extreme cases of zero suction change and full wetting. Expressions relating these parameters to soil suctions between these limits had to be assumed.

Secondly, assumptions were made to simplify the equations describing the material behaviour. For consolidation after heave it was assumed that the clay response is initially elastic and governed by the saturated elastic logarithmic bulk modulus until it reaches a yield point. While this leads to a convenient formulation of the mechanical consolidation expressions the numerical test problems showed that the agreement between numerical and experimental results is poorer than for other parts of the analysis.

The model only simulates the volumetric behaviour of the clay and makes no attempt to describe its shear resistance and shear failure. The clay exhibits a frictional resistance to shear deformations which requires a constitutive formulation such as the critical state model in order to describe this properly. The volumetric behaviour of the material defined in the model above represents the volumetric part of a critical state model for heaving clays. It would therefore be possible to extend this constitutive model to a full critical state model by developing a formulation for the shear resistance of the clay in terms of the applied pressure and the soil suction.

8.4 Discussion of the Numerical Implementation of the Constitutive Model

The numerically implemented model provides a computer based tool for analysing problems related to the volumetric behaviour of expansive clays. It allows for the analysis of both heave and mechanical consolidation aspects of the behaviour, which is not possible using the constitutive models available with most finite element packages. The model is formulated in terms of total rather than effective stresses. This makes the volume changes easier to conceptualise because the complex χ function used in an effective stress formulation is not required in a total stress formulation.

A disadvantage of the numerical implementation is that it is formulated with the temperature substituted for the soil suction. The solution is therefore not capable of measuring the time rate of changes in volume because a temperature solution and partially saturated flow solution will have a different time responses. This means that the model will only describe changes in volume for final steady state conditions and will not accurately predict the time related changes in volume as the solution converges to the steady state.

8.5 Conclusions

The aim of the research project described in Chapter 2 was to develop a constitutive model for heaving clays. The motivation for this was firstly to provide a numerical analysis tool for designers through the finite element method. Secondly, a detailed means of describing the change of heaving clay volume with changes in soil moisture was required. And thirdly, a better understanding of the volumetric behaviour of heaving clay under different loading and wetting paths was required.

The volumetric response of heaving clay under different test paths has been explained by separating the behaviour into two different aspects, namely heave and mechanical consolidation. This successfully explains the behaviour of the clay when subject to different loading and wetting paths.

The heave strain that develops in a heaving clay with changes in soil moisture was described using a heave state surface which expressed the heave strain in terms of the applied stress and soil suction. This description was sufficient to form the constitutive equation describing the heave behaviour of the clay.

A constitutive model for a heaving clay from Rosebank in the Cape Province

was developed. The model describes the experimentally observed behaviour closely. However, it is not possible to generalise the model for other clays until further experimental work has been done. The model does not describe the shear resistance and failure of the clay, which limits the range of problems for which the model can be used.

The following general conclusions can be made based on the work done in the research project:

- 1) Heave can be defined as the volumetric expansion shown by a heaving clay when free water is added to it. It can be quantified by the parameters *heave pressure* and *percent heave*. The *heave pressure* is the value of the applied stress which will constrain the volume increase to zero when wetted. The *percent heave* is the volumetric strain under a nominal applied load (usually 1 kPa).
- 2) The *heave pressure* and *percent heave* are functions of the moisture conditions and dry density of the soil.
- 3) Linear relationships exist between the heave strain in a heaving clay and the logarithm of the applied stress at wetting, and between heave strain and the logarithm of the soil suction. These two relationships can be combined to form a state function relating the three variables to each other. This function serves as the constitutive equation for heave behaviour.
- 4) The consolidation of heaving clay under changes in load is a mechanical process and should be considered separately from the heave that occurs during wetting, which is a chemical process. There is experimental evidence that supports the proposal of separation of the two aspects. Treating the material behaviour in this way explains the differences in the volume of the clay that results from different stress and wetting paths.
- 5) Separate consideration of these two aspects of the clay response does not imply that they are independent. Plastic deformation that occurs

before wetting increases the *heave pressure* and *percent heave* of the clay and therefore increases the heave strain that a sample will display at a specific applied load. Wetting of the clay causes an increase in the elastic logarithmic bulk modulus κ and a change in the yield stress p_c that will be observed during subsequent mechanical consolidation.

- 6) A constitutive model for the volumetric response of Rosebank clay to changes in loading and wetting has been developed and numerically implemented within the finite element method. This is capable of modelling the volumetric behaviour under changes in loading and wetting reasonably accurately, but makes no attempt to simulate the shear response of the clay. However, the volumetric model proposed can be developed into a full critical state model for heaving clay which would model both the volumetric and shear behaviour.

This research work has contributed to the understanding of the volumetric behaviour of heaving clay through an original experimental programme which investigated the effect of test stress path on the material volume. This was combined with existing knowledge of the clay behaviour to develop a constitutive model for the clay. This will enable researchers and analysts to make better qualitative and quantitative predictions of heaving clay behaviour.

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APPENDIX I
TEST DATA AND RESULT PLOTS FROM EXPERIMENTAL
PROGRAMME

CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 1
 Sample No.: R1-1
 Machine No: A
 Particle Density: 2.755

Ring Data: Ring # A1 Sample Parameters

Diameter:	76.150		Initial	Final
Height:	19.295	Void Ratio e	0.874	0.929
Mass:	96.000	Saturation Sr	0.422	1.172
Area:	4554.385	Dry Density	1.470	1.428
Volume:	87876.850			
Ht aft tst	19.868			
Vol aft.	90485.146			

Weighings:

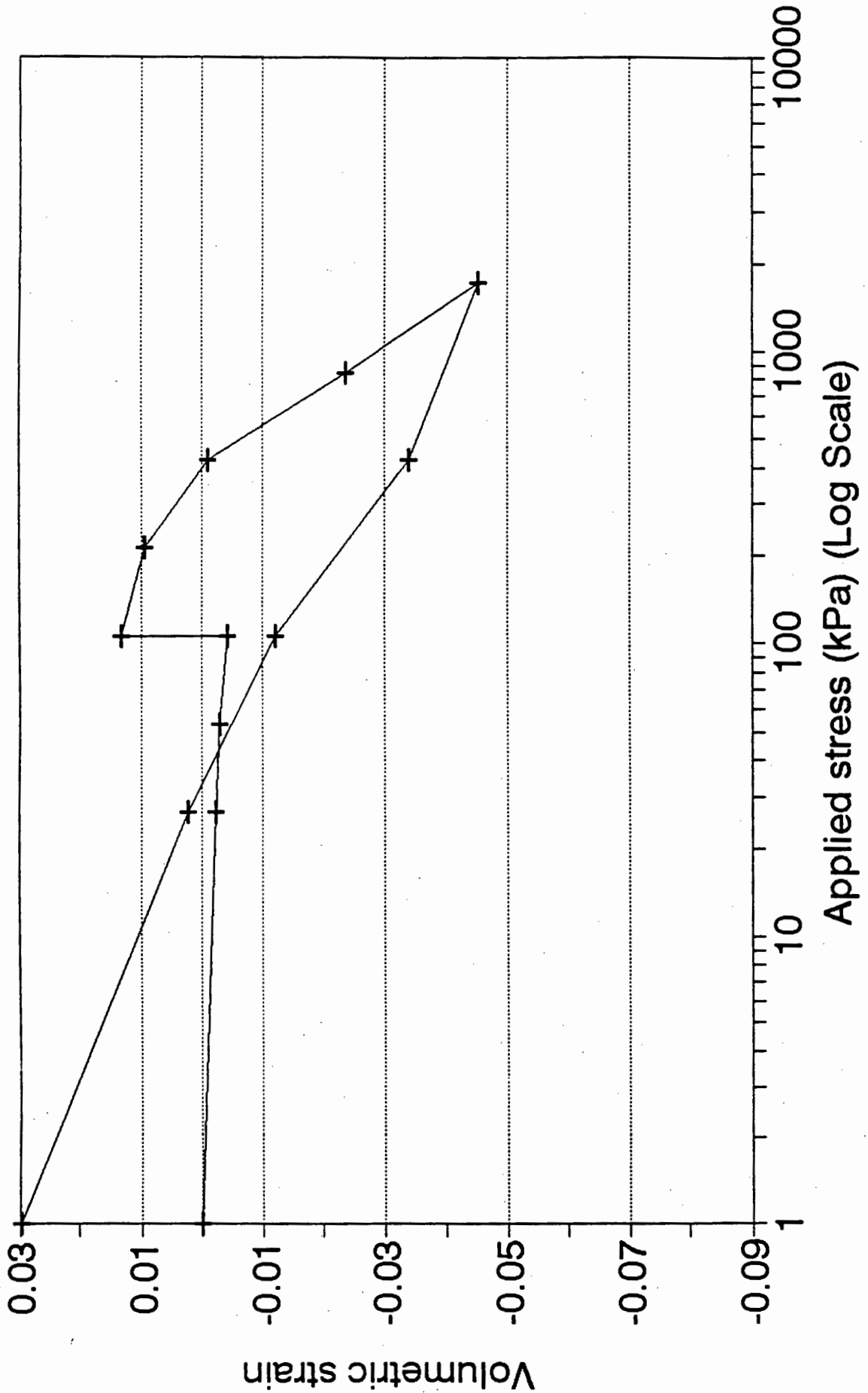
	Before Test	After Test
Wet soil + ring + Tray	243.000	281.000
Ring + Tray	96.500	100.700
Dry Soil + Ring + Tray		229.900
Wet Soil	146.500	180.300
Dry Soil	129.200	129.200
Moisture content	0.134	0.396
Moist Density (kg/m ³)	1667.106	1992.592

Moisture Content from trimmings

Containers	1	2	3	4	5
	12	169	223	115	
Wet Soil+ cont	19.600	13.800	20.100	25.300	
Dry soil + cont	18.200	12.900	18.900	23.700	
Container	2.100	2.000	2.100	2.100	
Water	1.400	0.900	1.200	1.600	0.000
Dry Soil	16.100	10.900	16.800	21.600	0.000
Moisture Content	0.087	0.083	0.071	0.074	NA
Average		7.876 %			

Rosebank Series 1

Sample R1-1 (Machine A)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 1
 Sample No.: R1-2
 Machine No: B
 Particle Density: 2.755

Ring Data: Ring # B1

Diameter: 76.060
 Height: 19.350
 Mass: 94.400
 Area: 4543.625
 Volume: 87919.152
 Ht aft tst 20.195
 Vol aft. 91758.516

Sample Parameters

	Initial	Final
Voids Ratio e	ERR	0.958
Saturation Sr	ERR	1.154
Dry Density	ERR	1.407

Weighings:

	Before Test	After Test
Wet soil + ring + Tray		280.100
Ring + Tray		99.200
Dry Soil + Ring + Tray		228.300
Wet Soil	0.000	180.900
Dry Soil	129.100	129.100
Moisture content	-1.000	0.401
Moist Density (kg/m ³)	0.000	1971.479

Moisture Content from trimmings

Containers	1	2	3	4	5
	131	155	224	153	191
Wet Soil+ cont	20.100	19.800	21.400	20.600	21.800
Dry soil + cont	18.400	18.000	19.200	18.300	19.600
Container	2.100	2.100	2.100	2.100	2.100
Water	1.700	1.800	2.200	2.300	2.200
Dry Soil	16.300	15.900	17.100	16.200	17.500
Moisture Content	0.104	0.113	0.129	0.142	0.126
Average		12.277	%		

CONSOLIDATION TEST: SETTLEMENT READINGS

Rosebank Clay Samples

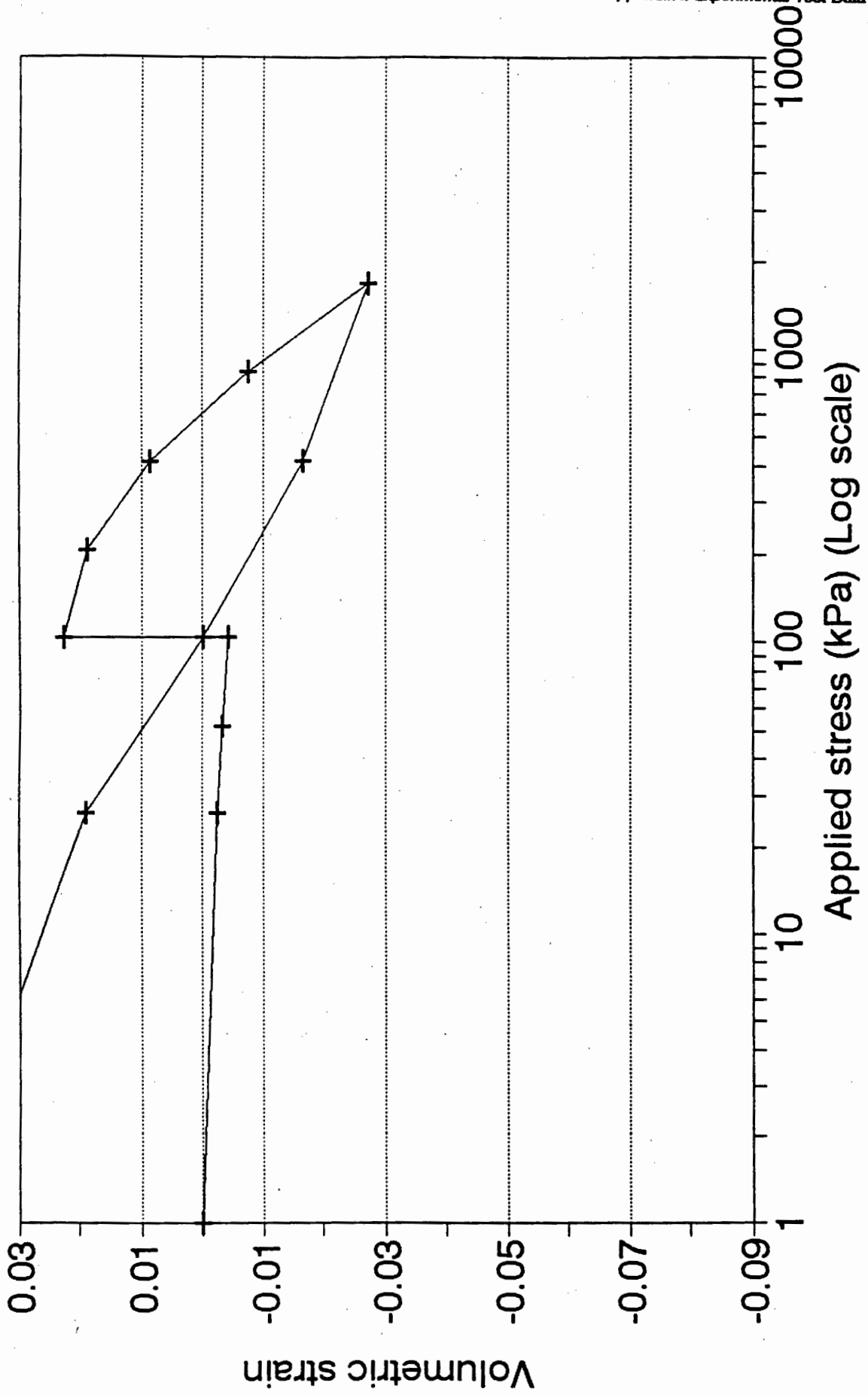
Test Series Code:	1	Loading Stone Diameter										
Sample No:	R1-2	Avg: 75.45										
Machine Number	B	Area (mm ²): 4471.038										
		Area (m ²): 0.004471										
Init. sample ht	19.35	Loading Arm Ratio: 10.55094										
Load Inc.	1	2	3	4	5	6	7	8	9	10	11	12
Load (kg)	1.135	2.27	4.54	water	9.08	18.16	36.32	72.64	18.16	4.54	1.135	0
Load (kPa)	1	26.27531	52.55061	105.1012	105.1012	210.2025	420.4049	840.8098	1681.62	420.4049	105.1012	26.27531
Time (min)	0	0.25	24.98	24.85	24.64	4.82	4.112	2.128	20.17	22.25	0.41	4.173
	0.25	0.5										
	0.5	0.707107										
	1	1										
	2	1.414214										
	5	2.236068										
	10	3.162278										
	30	5.477226										
	60	7.745967										
	120	10.95445										
	240	15.49193										
	480	21.9089										
	1440	37.94733										

DIAL GAUGE READINGS

Nett Compr. over inc. (mm)	-0.25	24.85	24.64	29.82	4.112	2.128	20.17	22.25	25.41	4.173	8.93
	0.5	0.13	0.21	-5.18	0.708	1.984	3.84	-2.08	-3.16	-3.763	-4.757
	0.05	0.013	0.021	-0.518	0.0708	0.1984	0.384	-0.208	-0.316	-0.3763	-0.4757
	0.05	0.063	0.084	-0.434	-0.3632	-0.1648	0.531	0.323	0.007	-0.3693	-0.845
Cumulative compr.	0.05	0.063	0.084	-0.434	-0.3632	-0.1648	0.531	0.323	0.007	-0.3693	-0.845
Cumul. correction	19.3	19.287	19.266	19.784	19.7132	19.5148	18.819	19.027	19.343	19.7193	20.195
Final sample height	0	-0.00258	-0.00326	-0.00434	0.022429	0.01877	0.008517	-0.02744	-0.00036	0.019085	0.043669
Vol. Strain	0	-0.00258	-0.00326	-0.00434	0.022429	0.01877	0.008517	-0.02744	-0.00036	0.019085	0.043669

Rosebank Series 1

Sample R1-2 (Machine B)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 1
 Sample No.: R1-3
 Machine No: C
 Particle Density: 2.755

Ring Data: Ring # C1 Sample Parameters

Diameter:	69.950		Initial	Final
Height:	19.000	Voids Ratio e	0.863	0.893
Mass:	82.600	Saturation Sr	0.237	1.115
Area:	3842.955	Dry Density	1.479	1.456
Volume:	73016.148			
Ht aft tst	19.306			
Vol aft.	74192.093			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	199.100	234.300
Ring + Tray	83.100	87.300
Dry Soil + Ring + Tray		195.300
Wet Soil	116.000	147.000
Dry Soil	108.000	108.000
Moisture content	0.074	0.361
Moist Density (kg/m ³)	1588.690	1981.343

Moisture Content from trimmings

Containers	1	2	3	4	5
	26	128	216	25	
Wet Soil+ cont	16.700	21.100	19.400	18.900	
Dry soil + cont	15.700	20.000	18.400	17.800	
Container	2.100	2.100	2.000	2.100	
Water	1.000	1.100	1.000	1.100	0.000
Dry Soil	13.600	17.900	16.400	15.700	0.000
Moisture Content	0.074	0.061	0.061	0.070	NA
Average		6.651	%		

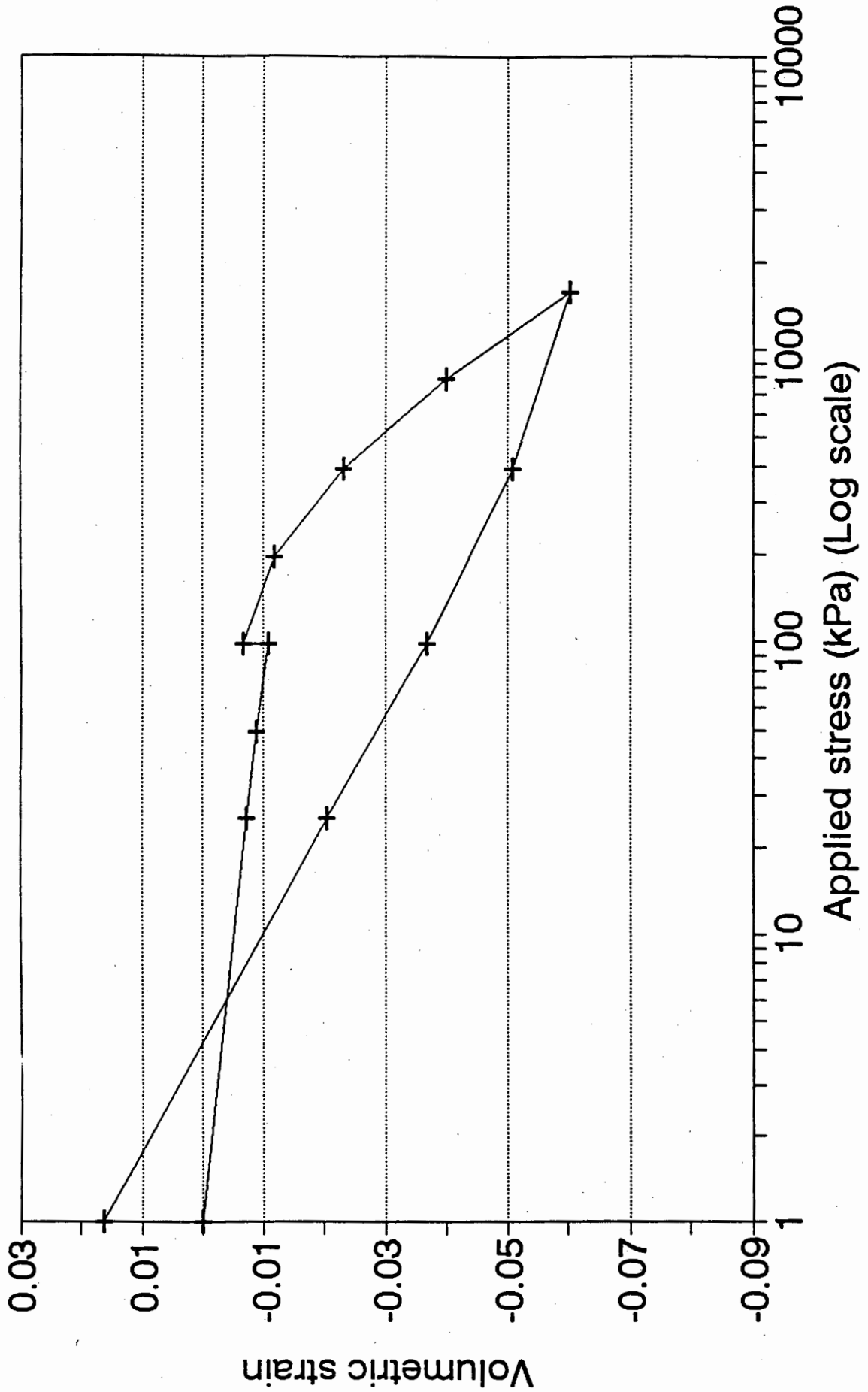
CONSOLIDATION TEST: SETTLEMENT READINGS

Rosebank Clay Samples

Test Series Code:	1	2	3	4	5	6	7	8	9	10	11	12
Sample No:	R1-3											
Machine Number	C											
Init. sample ht	19											
Load Inc.	1	2	3	4	5	6	7	8	9	10	11	12
Load (kg)	0.885	1.75	3.5	water								
Load (kPa)	1	25.00101	49.43702	98.87404	98.87	197.7481	395.4962	790.9923	1581.985	395.4962	98.87404	25.00101
Time (min)												
SQRT (time)	0	0	0	0	0	0	0	0	0	0	0	0
	0.25	0.5										
	0.5	0.707107										
	1	1	1	1	1	1	1	1	1	1	1	1
	2	1.414214										
	5	2.236068										
	10	3.162278										
	30	5.477226										
	60	7.745967										
	120	10.95445										
	240	15.49193										
	480	21.9089										
	1440	37.94733										
Nett Compr. over inc.												
(mm)												
Cumulative compr.												
Cumul. correction												
Nett Cum. Compr.												
Final sample height												
Vol. Strain												

Rosebank Series 1

Sample R1-3 (Machine C)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 1
 Sample No.: R1-4
 Machine No: D
 Particle Density: 2.755

Ring Data: Ring # D1

Sample Parameters

		Initial	Final
Diameter:	70.087		
Height:	19.125	Voids Ratio e	0.938 0.923
Mass:	81.800	Saturation Sr	0.294 1.136
Area:	3858.023	Dry Density	1.422 1.433
Volume:	73784.692		
Ht aft tst	18.976		
Vol aft.	73207.917		

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	197.700	231.400
Ring + Tray	82.300	86.600
Dry Soil + Ring + Tray		191.500
Wet Soil	115.400	144.800
Dry Soil	104.900	104.900
Moisture content	0.100	0.380
Moist Density (kg/m ³)	1564.010	1977.928

Moisture Content from trimmings

Containers	1	2	3	4	5
	249	186	70	4	
Wet Soil+ cont	17.700	22.300	21.600	25.100	
Dry soil + cont	16.400	20.700	19.800	23.200	
Container	2.100	2.200	2.100	2.100	
Water	1.300	1.600	1.800	1.900	0.000
Dry Soil	14.300	18.500	17.700	21.100	0.000
Moisture Content	0.091	0.086	0.102	0.090	NA

Average 9.228 %

CONSOLIDATION TEST: SETTLEMENT READINGS

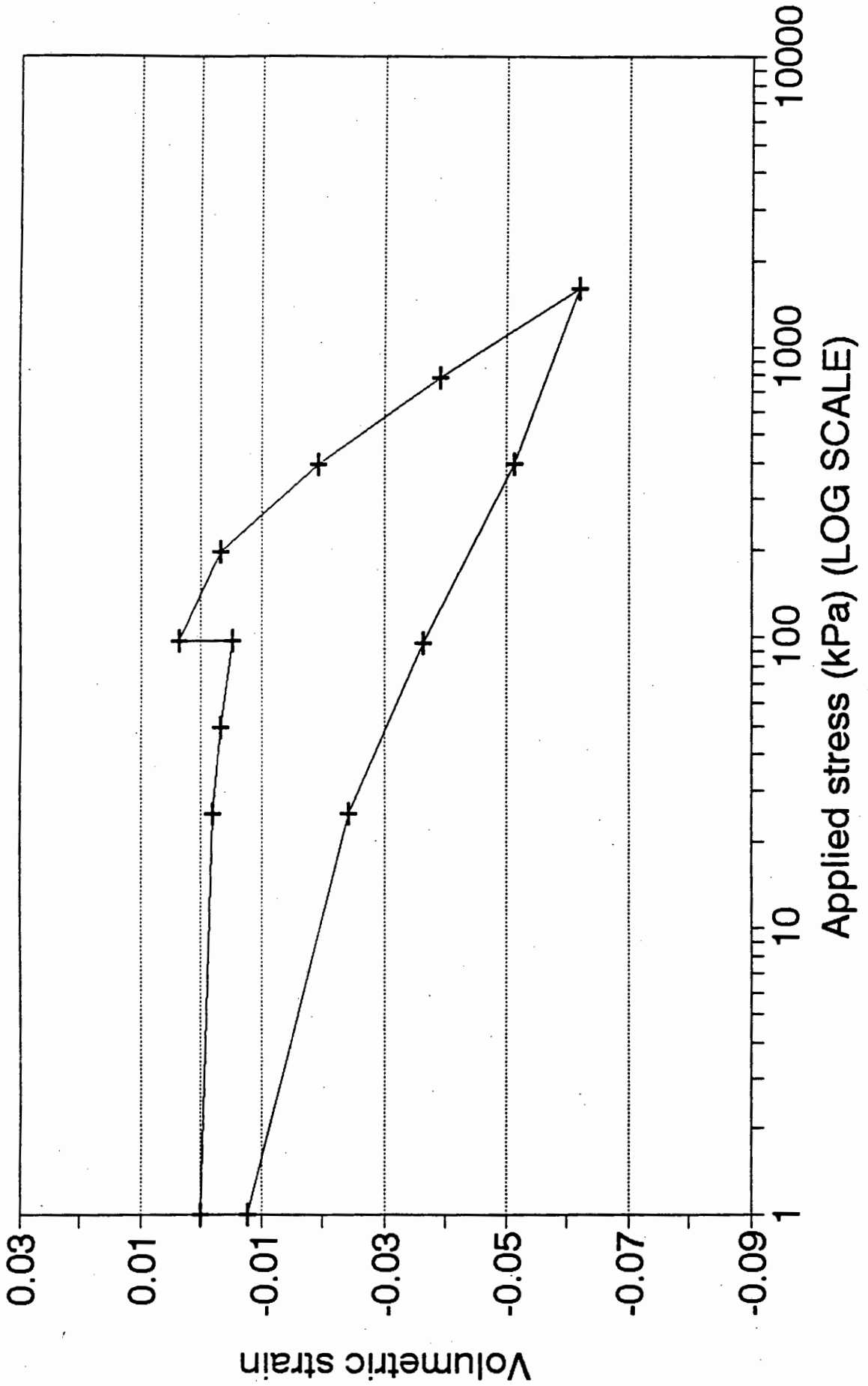
Rosebank Clay Samples

Test Series Code:	1	Loading Stone Diameter											
Sample No:	R1-4	1	2	3	4	5	6	7	8	9	10	11	12
Machine Number	D	68.4	68.6	68.2	68.4	68.2	68.4	68.4	68.2	68.4	68.2	68.4	68.4
Init. sample ht	19.125	Avg: 68.36667											
Load Inc.	1	6	7	8	9	10	11	12	0	1	2	3	4
Load (kg)	0.885	1.75	3.5	water	7	14	28	56	14	3.4	0.885	0	1
Load (kPa)	1	25.16086	49.75312	99.50624	99.506	199.0125	398.025	796.0499	1592.1	398.025	96.66321	25.16086	1
Time (min)	0	1.62	2.02	2.27	2.65	2.25	5.285	9.109	13.47	11.43	8.58	6.265	6.265
	0.25	0.5											
	0.5	0.707107											
	1	1											
	2	1.414214											
	5	2.236068											
	10	3.162278											
	30	5.477226											
	60	7.745967											
	120	10.95445											
	240	15.49193											
	480	21.9089											
	1440	37.94733	2.02	2.27	2.65	0.931	2.25	5.285	9.109	13.47	11.43	8.58	6.265
Nett Compr. over inc.	(mm)	0.4	0.25	0.38	-1.719	1.319	3.035	3.824	4.361	-2.04	-2.85	-2.315	-3.15
		0.04	0.025	0.038	-0.1719	0.1319	0.3035	0.3824	0.4361	-0.204	-0.285	-0.2315	-0.315
		0.04	0.065	0.103	-0.0689	0.063	0.3665	0.7489	1.185	0.981	0.696	0.4645	0.1495
Cumulative compr.													
Cumul. correction													
Nett Cum. Compr.		0.04	0.065	0.103	-0.0689	0.063	0.3665	0.7489	1.185	0.981	0.696	0.4645	0.1495
Final sample height		19.085	19.06	19.022	19.1939	19.062	18.7585	18.3761	17.94	18.144	18.429	18.6605	18.9755
Vol. Strain	0	-0.00209	-0.0034	-0.00539	0.003603	-0.00329	-0.01916	-0.03916	-0.06196	-0.05129	-0.03639	-0.02429	-0.00782

Appendix I: Experimental Test Data

Rosebank Series 1

Sample R1-4 (Machine D)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 2
 Sample No.: R2-1
 Machine No: A
 Particle Density: 2.755

Ring Data: Ring # A2

Sample Parameters

Diameter:	76.206		Initial	Final
Height:	18.983	Voids Ratio e	0.880	0.967
Mass:	93.500	Saturation Sr	0.480	1.035
Area:	4561.086	Dry Density	1.466	1.400
Volume:	86581.718			
Ht aft tst	19.868			
Vol aft.	90618.279			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	418.800	272.100
Ring + Tray	272.450	99.100
Dry Soil + Ring + Tray		226.000
Wet Soil	146.350	173.000
Dry Soil	126.900	126.900
Moisture content	0.153	0.363
Moist Density (kg/m ³)	1690.311	1909.107

Moisture Content from trimmings

Containers	1	2	3	4	5
	7	35	244	249	
Wet Soil+ cont	24.700	26.900	20.400	24.400	
Dry soil + cont	22.200	24.000	18.400	21.800	
Container	2.100	2.100	2.100	2.100	
Water	2.500	2.900	2.000	2.600	0.000
Dry Soil	20.100	21.900	16.300	19.700	0.000
Moisture Content	0.124	0.132	0.123	0.132	NA
Average		12.787 %			

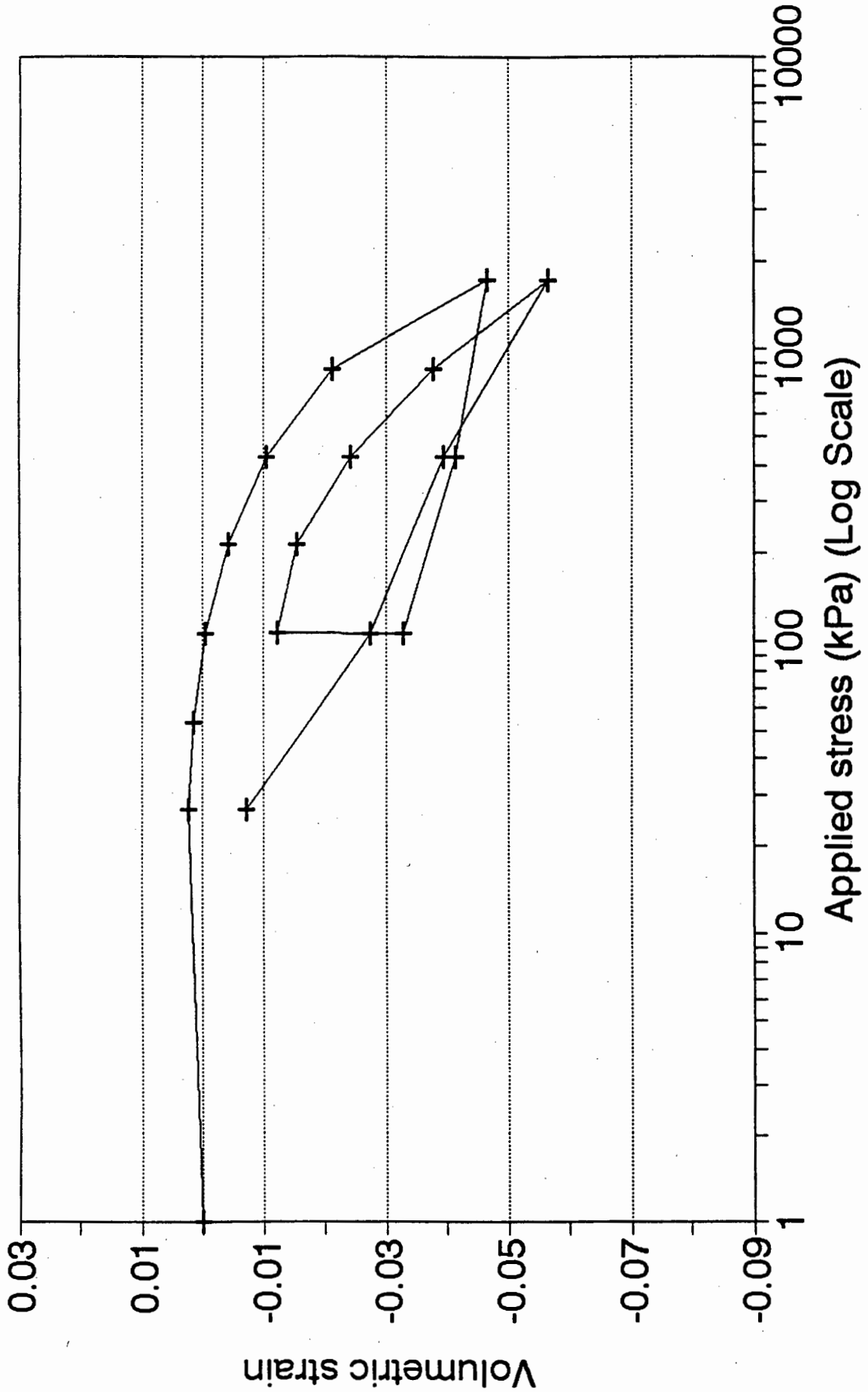
CONSOLIDATION TEST: SETTLEMENT READINGS

Rosebank Clay Samples

Test Series Code:	2	Loading Stone Diameter											
Sample No:	R2-1	Avg:											
Machine Number	A	Area (mm ²)											
		Loading Arm Ratio											
Init. sample ht	18.9827	1	2	3	4	5	6	7	8	9	10	11	12
Load Inc.	1	1.135	2.27	4.54	9.08	18.16	36.32	72.64	18.16	4.54	water	9.08	18.16
Load (kg)	1	26.76877	53.53755	107.0751	214.1502	428.3004	856.6007	1713.201	428.3004	107.0751	107.9525	214.1502	428.3004
Load (kPa)	1	26.76877	53.53755	107.0751	214.1502	428.3004	856.6007	1713.201	428.3004	107.0751	107.9525	214.1502	428.3004
Time (min)	0	24.98	24.55	24.71	0.08	0.795	2.01	4.05	8.81	7.876	6.22	2.34	2.92
	0.5	0.707107	24.65	24.62	24.9	0.5	3.5	4.96	8.02	6.55	6.16	2.42	3.5
	1	1	24.79	24.62	24.91	0.52	3.55	6.1	8.02	6.52	6.05	2.69	3.94
	2	1.414214	24.8	24.63	24.92	0.54	3.61	7.18	8.02	6.49	5.91	2.71	4.1
	5	2.236068	24.825	24.635	24.93	0.57	3.68	7.3	7.99	6.445	5.69	2.75	4.19
	10	3.162278	24.825	24.64	24.95	0.6	3.74	7.38	7.96	6.42	4.92	2.77	4.25
	30	5.477226	24.8	24.65	24.97		3.87	7.5	7.93	6.36	4.85	2.81	4.37
	60	7.745967	24.76	24.66	24.99			7.55	7.92	6.33	4.78	2.84	4.435
	120	10.95445	24.75					7.65	7.9	6.3	3.06	2.86	4.49
	240	15.49193						7.67	7.89	6.27	2.64	2.88	4.53
	480	21.9089											
1440	37.94733	24.55	24.7	25.08	0.795	2.01	4.05	8.81	7.876	6.22	2.34	2.92	4.6
Nett Compr. over inc.	(mm)	-0.43	0.15	0.37	0.715	1.215	2.04	4.76	-0.934	-1.656	-3.88	0.58	1.68
		-0.043	0.015	0.037	0.0715	0.1215	0.204	0.476	-0.0934	-0.1656	-0.388	0.058	0.168
Cumulative compr.		-0.043	-0.028	0.009	0.0805	0.202	0.406	0.882	0.7886	0.623	0.235	0.293	0.461
Cumul. correction													
Nett Cum. Compr.		-0.043	-0.028	0.009	0.0805	0.202	0.406	0.882	0.7886	0.623	0.235	0.293	0.461
Final sample height		19.0257	19.0107	18.9737	18.9022	18.7807	18.5767	18.1007	18.1941	18.3597	18.7477	18.6897	18.5217
Vol. Strain	0	0.002265	0.001475	-0.00047	-0.00424	-0.01064	-0.02139	-0.04646	-0.04154	-0.03282	-0.01238	-0.01544	-0.02429

Rosebank Series 2

Sample R2-1 (Machine A)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 2
 Sample No.: R2-2
 Machine No: B
 Particle Density: 2.755

Ring Data: Ring # B2

Sample Parameters

Diameter:	76.237		Initial	Final
Height:	19.075	Voids Ratio e	0.886	0.852
Mass:	95.000	Saturation Sr	0.502	1.301
Area:	4564.761	Dry Density	1.461	1.487
Volume:	87072.819			
Ht aft tst	18.736			
Vol aft.	85523.083			

Weighings:

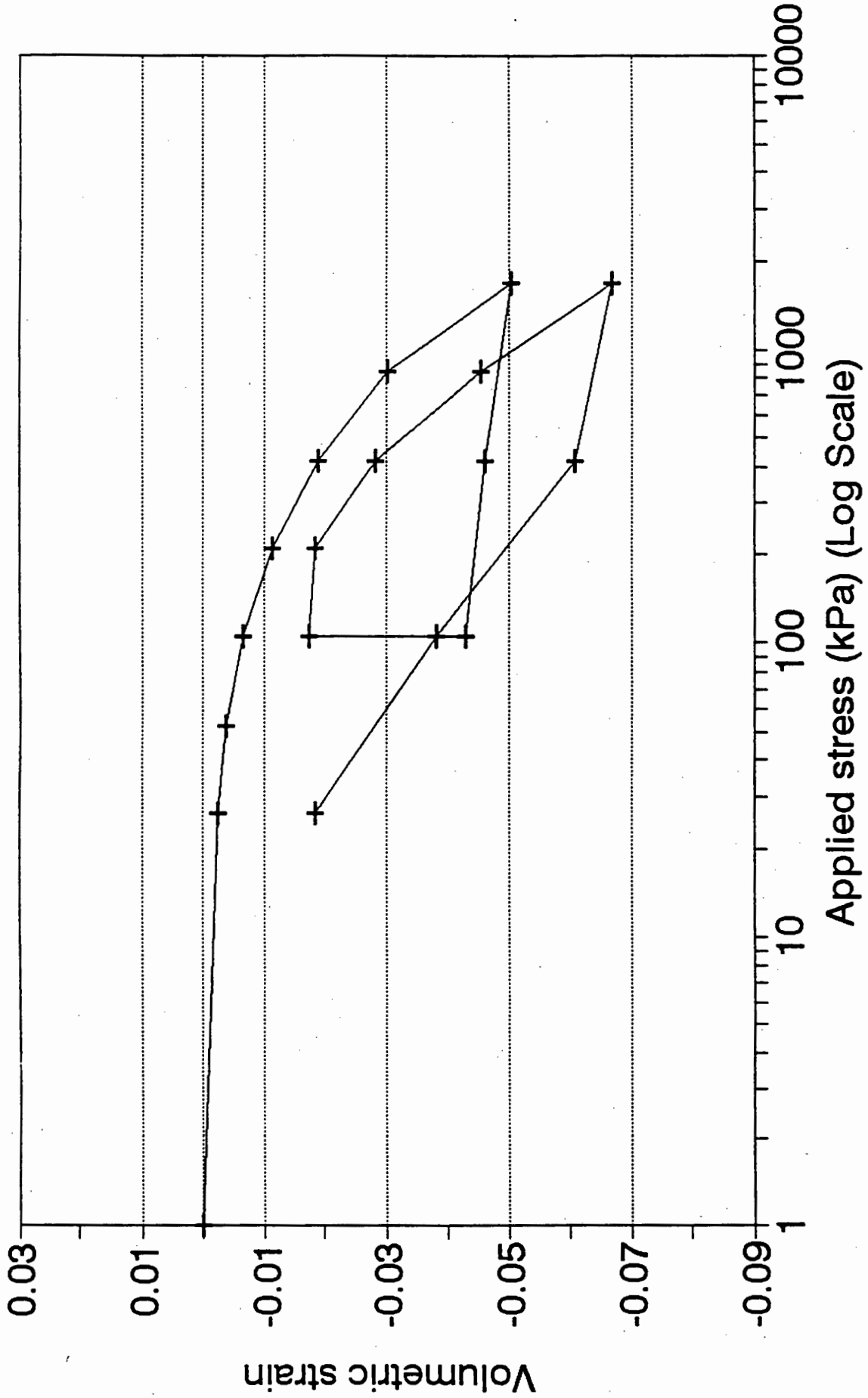
	Before Test	After Test
Wet soil + ring + Tray	421.700	279.200
Ring + Tray	273.950	100.800
Dry Soil + Ring + Tray		228.000
Wet Soil	147.750	178.400
Dry Soil	127.200	127.200
Moisture content	0.162	0.403
Moist Density (kg/m ³)	1696.856	2085.987

Moisture Content from trimmings

Containers	1	2	3	4	5
	A13	A16	A12	C1	A11
Wet Soil+ cont	19.000	21.400	20.600	18.200	21.800
Dry soil + cont	17.100	19.100	18.400	16.300	19.400
Container	2.100	2.100	2.100	2.100	2.100
Water	1.900	2.300	2.200	1.900	2.400
Dry Soil	15.000	17.000	16.300	14.200	17.300
Moisture Content	0.127	0.135	0.135	0.134	0.139
Average		13.389 %			

Rosebank Series 2

Sample R2-2 (Machine B)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 2
 Sample No.: R2-3
 Machine No: C
 Particle Density: 2.755

Ring Data: Ring # C2 Sample Parameters

Diameter:	70.020		Initial	Final
Height:	19.050	Voids Ratio e	0.909	0.891
Mass:	85.300	Saturation Sr	0.431	1.207
Area:	3850.650	Dry Density	1.443	1.457
Volume:	73354.891			
Ht aft tst	18.867			
Vol aft.	72648.296			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	384.800	237.900
Ring + Tray	263.900	90.750
Dry Soil + Ring + Tray		196.600
Wet Soil	120.900	147.150
Dry Soil	105.850	105.850
Moisture content	0.142	0.390
Moist Density (kg/m ³)	1648.152	2025.512

Moisture Content from trimmings

Containers	1	2	3	4	5
	83	236	175	74	
Wet Soil+ cont	17.500	16.300	20.900	23.300	
Dry soil + cont	16.000	15.000	19.200	21.300	
Container	2.100	2.100	2.000	2.200	
Water	1.500	1.300	1.700	2.000	0.000
Dry Soil	13.900	12.900	17.200	19.100	0.000
Moisture Content	0.108	0.101	0.099	0.105	NA
Average		10.306	%		

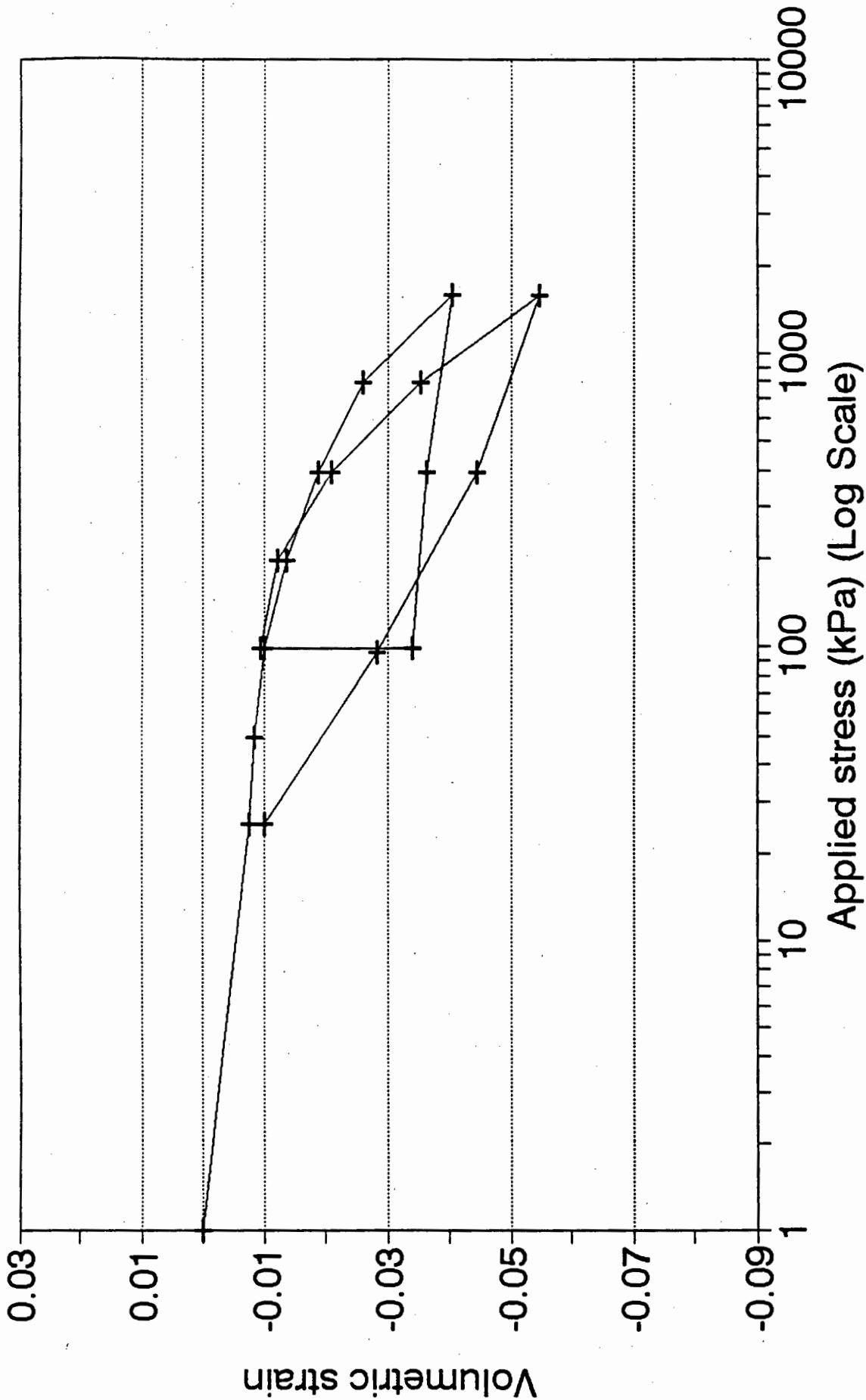
CONSOLIDATION TEST: SETTLEMENT READINGS

Rosebank Clay Samples

Test Series Code:	2	1	2	3	4	5	6	7	8	9	10	11	12
Sample No:	R2-3	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
Machine Number	C	68.35	3669.162	0.003669	10.56604	197.7481	395.4962	790.9923	1581.985	395.4962	98.87404	98.87404	197.7481
Init. sample ht	19.05	19.96	18.56	15.81	16.55	16.98	16.98	16.98	16.98	16.98	16.98	16.98	16.98
Load Inc.	1	2	3	4	5	6	7	8	9	10	11	12	14
Load (kg)	0.885	1.75	3.5	7	14	28	56	14	3.5	water	7	7	14
Load (kPa)	1	25.00101	49.43702	98.87404	197.7481	395.4962	790.9923	1581.985	395.4962	98.87404	98.87404	197.7481	395.4962
Time (min)	0	0	0	0	0	0	0	0	0	0	0	0	0
	0.5	0.707107	22.04	21.89	21.57	20.91	19.96	18.56	15.81	16.55	16.98	21.7	21.17
	1	1	21.98	21.7	21.13	20.19	18.93	16.72	16.4	16.78	16.98	21.39	20.05
	2	1.414214	21.96	21.69	21.11	20.16	18.9	16.4	16.44	16.79	17.04	21.32	19.95
	5	2.236068	21.96	21.68	21.09	20.16	18.87	16.32	16.46	16.8	17.19	21.315	19.89
	10	3.162278	21.95	21.67	21.06	20.16	18.83	16.24	16.48	16.83	17.55	21.31	19.81
	30	5.477226	21.95	21.65	21.05	20.16	18.79	16.18	16.49	16.84	17.93	21.3	19.73
	60	7.745967	21.965	21.64	21.05	20.16	18.7	16.1	16.51	16.87	18.9	21.26	19.67
	120	10.95445	21.998	21.63	21.05	20.16	18.7	16.03	16.52	16.89	20.1	21.255	19.63
	240	15.49193	22.016	21.92	21.05	20.16	18.7	15.95	16.53	16.925	21.15	21.22	19.59
	480	21.9089					15.938	16.54	16.54	16.95	21.55	21.21	19.55
1440	37.94733	22.04	21.89	21.57	20.91	19.96	18.56	15.81	16.55	16.98	21.7	21.17	19.52
Nett Compr. over inc.	(mm)	1.465	0.15	0.32	0.66	0.95	1.4	2.75	-0.74	-0.43	-4.72	0.53	1.65
Cumulative compr.	(mm)	0.1465	0.015	0.032	0.066	0.095	0.14	0.275	-0.074	-0.043	-0.472	0.053	0.165
Cumul. correction		0.1465	0.1615	0.1935	0.2595	0.3545	0.4945	0.7695	0.6955	0.6525	0.1805	0.2335	0.3985
Nett Cum. Compr.		0.1465	0.1615	0.1935	0.2595	0.3545	0.4945	0.7695	0.6955	0.6525	0.1805	0.2335	0.3985
Final sample height		18.9035	18.8885	18.8565	18.7905	18.6955	18.5555	18.2805	18.3545	18.3975	18.8695	18.8165	18.6515
Vol. Strain	0	-0.00769	-0.00848	-0.01016	-0.01362	-0.01861	-0.02596	-0.04039	-0.03651	-0.03425	-0.00948	-0.01226	-0.02092

Rosebank Series 2

Sample R2-3 (Machine C)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 2
 Sample No.: R2-4
 Machine No: D
 Particle Density: 2.755

Ring Data: Ring # D2 Sample Parameters

Diameter:	70.123		Initial	Final
Height:	19.030	Void Ratio e	0.869	0.858
Mass:	82.500	Saturation Sr	0.414	1.245
Area:	3862.024	Dry Density	1.474	1.483
Volume:	73494.313			
Ht aft tst	18.921			
Vol aft.	73071.421			

Weighings:

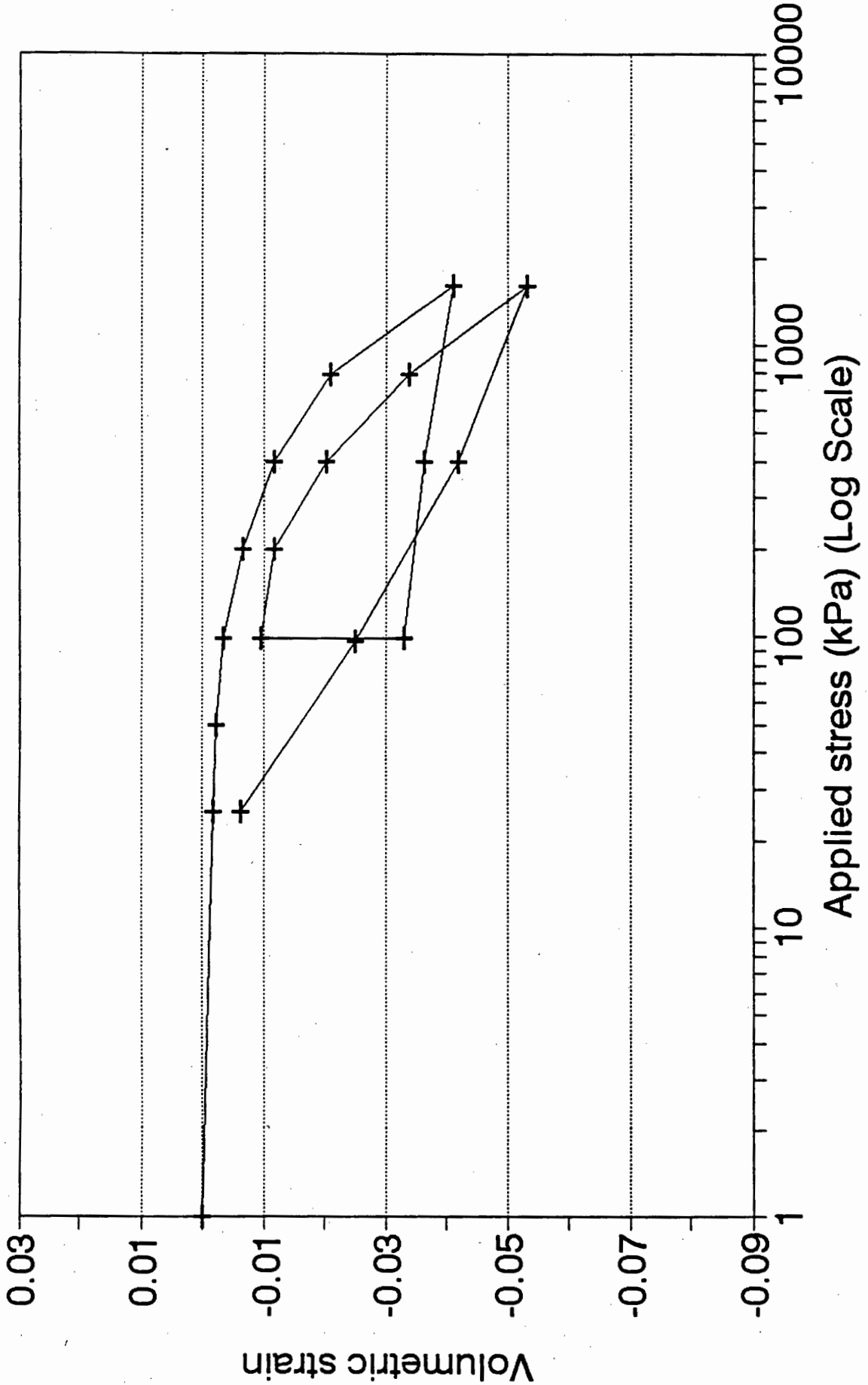
	Before Test	After Test
Wet soil + ring + Tray	383.700	238.300
Ring + Tray	261.200	87.950
Dry Soil + Ring + Tray		196.300
Wet Soil	122.500	150.350
Dry Soil	108.350	108.350
Moisture content	0.131	0.388
Moist Density (kg/m ³)	1666.796	2057.576

Moisture Content from trimmings

Containers	1	2	3	4	5
	23	72	207	213	
Wet Soil+ cont	19.300	24.800	16.800	23.000	
Dry soil + cont	17.700	22.400	15.600	21.200	
Container	2.100	2.100	2.100	2.100	
Water	1.600	2.400	1.200	1.800	0.000
Dry Soil	15.600	20.300	13.500	19.100	0.000
Moisture Content	0.103	0.118	0.089	0.094	NA
Average		10.098 %			

Rosebank Series 2

Sample R2-4 (Machine D)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 3
 Sample No.: R3-1
 Machine No: A
 Particle Density: 2.755

Ring Data: Ring # A1 Sample Parameters

Diameter:	76.150		Initial	Final
Height:	19.295	Voids Ratio e	0.992	1.005
Mass:	96.000	Saturation Sr	0.427	1.077
Area:	4554.385	Dry Density	1.383	1.374
Volume:	87876.850			
Ht aft tst	19.421			
Vol aft.	88449.336			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	415.000	270.170
Ring + Tray	274.800	100.880
Dry Soil + Ring + Tray		222.410
Wet Soil	140.200	169.290
Dry Soil	121.530	121.530
Moisture content	0.154	0.393
Moist Density (kg/m ³)	1595.414	1913.977

Moisture Content from trimmings

Containers	1	2	3	4	5
	183	153	175	59	169
Wet Soil+ cont	24.000	22.700	22.900	21.800	21.600
Dry soil + cont	21.700	20.700	21.100	19.700	19.900
Container	2.100	2.100	2.100	2.100	2.100
Water	2.300	2.000	1.800	2.100	1.700
Dry Soil	19.600	18.600	19.000	17.600	17.800
Moisture Content	0.117	0.108	0.095	0.119	0.096
Average		10.689	%		

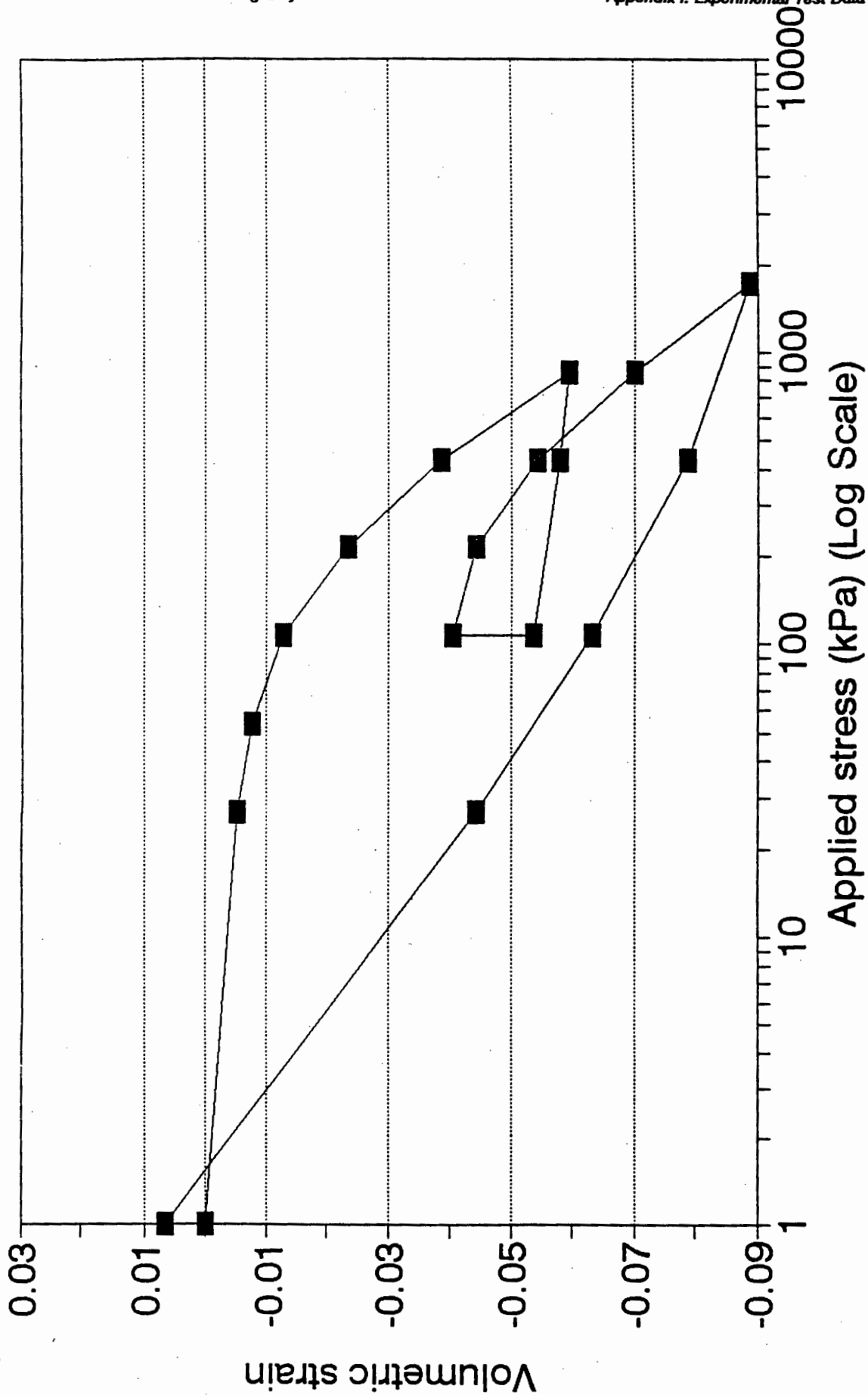
CONSOLIDATION TEST: SETTLEMENT READINGS

Rosebank Clay Samples

Test Series Code:	3	Loading Stone Diameter											
Sample No.:	R3-1	Avg:											
Machine Number	A	Area (mm ²)											
		Area (m ²)											
Init. sample ht	19.295	Loading Arm Ratio											
Load Inc.	1	2	3	4	5	6	7	8	9	10	11	12	
Load (kg)	1.135	2.27	4.54	9.08	18.16	36.32	18.16	4.54	water	9.08	18.16	36.32	
Load (kPa)	1	26.76877	53.53755	107.0751	214.1502	428.3004	856.6007	107.0751	107.0751	214.1502	428.3004	856.6007	
Time (min)	SQRT (time)	DIAL GAUGE READINGS											
0	0	7.4	8.445	8.9	9.89	11.96	14.89	18.59	17.75	15.23	15.95	17.877	
0.5	0.707107	7.98	8.63	9.5	11.45	14.21	18.05	18.07	17.69	15.23	17.2	19.52	
1	1	7.99	8.64	9.55	11.5	14.3	18.18	18.05	17.61	9.08	17.3	19.74	
2	1.414214	8.01	8.66	9.58	11.56	14.39	18.3	18.01	17.49	214.1502	17.39	19.93	
5	2.236068	8.03	8.68		11.63	14.48	18.44	17.97	17.09	428.3004	17.49	20.19	
10	3.162278		8.7	9.66	11.68	14.54	18.52	17.94	16.69	214.1502	17.49	20.37	
30	5.477226		8.74			14.63	18.65	17.89	16.06	428.3004	17.49	20.635	
60	7.745967		8.76			14.69	18.701	17.82	15.78	856.6007	17.49	20.74	
120	10.95445		8.8			14.76	18.77	17.83	15.51		17.49	20.8	
240	15.49193		8.82			14.78	18.82	17.8	15.36		17.49	20.85	
480	21.9089		8.86	9.87		14.82	18.87	17.8	15.36		17.49	20.85	
1440	37.94733		8.445	9.89	11.96	14.89	18.9	17.75	15.23	15.95	17.88	20.94	
Nett Compr. over inc. (mm)		1.045	0.455	0.99	2.07	2.93	4.01	-0.84	-2.52	0.72	1.93	3.063	
		0.1045	0.0455	0.099	0.207	0.293	0.401	-0.084	-0.252	0.072	0.193	0.3063	
Cumulative compr.		0.1045	0.15	0.249	0.456	0.749	1.15	1.035	0.783	0.855	1.048	1.3543	
Cumul. correction		0.036	0.06195	0.084	0.1175	0.1445	0.175	0.143	0.143	0.159	0.173	0.193	
Nett Cum. Compr.		0.0685	0.08805	0.165	0.3385	0.6045	0.975	0.892	0.64	0.696	0.875	1.1613	
Final sample height		19.2265	19.20695	19.13	18.9565	18.6905	18.32	18.403	18.655	18.599	18.42	18.1337	
Vol. Strain	0	-0.00355	-0.00456	-0.00855	-0.01754	-0.03133	-0.05053	-0.04623	-0.03317	-0.03607	-0.04535	-0.06019	

Rosebank Series 3

Sample R3-1 (Machine A)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 3
 Sample No.: R3-2
 Machine No: B
 Particle Density: 2.755

Ring Data: Ring # A1 Sample Parameters

Diameter:	76.060		Initial	Final
Height:	19.350	Voids Ratio e	0.940	1.005
Mass:	94.400	Saturation Sr	0.492	1.229
Area:	4543.625	Dry Density	1.420	1.374
Volume:	87919.152			
Ht aft tst	19.992			
Vol aft.	90836.160			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	419.000	280.070
Ring + Tray	273.200	99.293
Dry Soil + Ring + Tray		224.120
Wet Soil	145.800	180.777
Dry Soil	124.827	124.827
Moisture content	0.168	0.448
Moist Density (kg/m ³)	1658.342	1990.144

Moisture Content from trimmings

Containers	1	2	3	4	5
	F3-LL	A12	C1	C11	E1
Wet Soil+ cont	23.700	21.900	21.700	20.770	21.000
Dry soil + cont	21.600	20.100	19.600	18.900	18.900
Container	2.100	2.100	2.100	2.100	2.100
Water	2.100	1.800	2.100	1.870	2.100
Dry Soil	19.500	18.000	17.500	16.800	16.800
Moisture Content	0.108	0.100	0.120	0.111	0.125
Average		11.280	%		

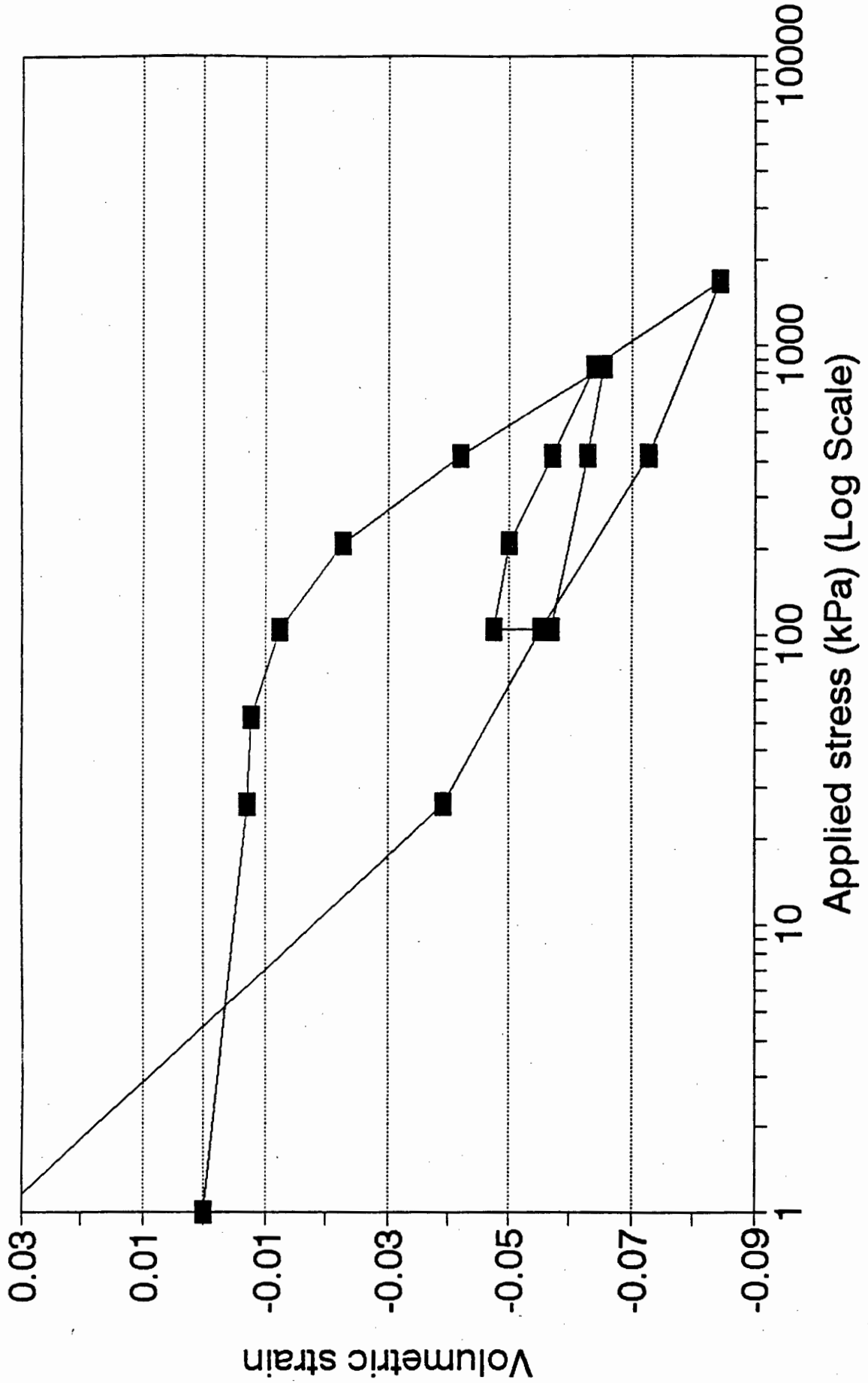
CONSOLIDATION TEST: SETTLEMENT READINGS

Rosebank Clay Samples

Test Series Code:	3	Loading Stone Diameter											
Sample No:	R3-2	Avg:											
Machine Number	B	Area (mm ²)											
Init. sample ht	19.35	Loading Arm Ratio											
Load Inc.	1	2	3	4	5	6	7	8	9	10	11	12	
Load (kg)	1.135	2.27	4.54	9.08	18.16	36.32	18.16	4.54	water	9.08	18.16	36.32	
Load (kPa)	1	26.27531	52.55061	105.1012	210.2025	420.4049	840.8098	420.4049	105.1012	210.2025	420.4049	840.8098	
Time (min)	0	0	17.47	16.11	17.3	16.43	14.43	10.71	6.63	7.775	9.11	7.75	
SQRT (time)	0	0	0.707107	15.95	14.9	14.8	14.65	7.2	7.18	7.8	8.42	6.8	
0.5	0.707107	16.6	15.58	15.69	15.71	15.71	15.71	6.14	6.4	7.83	8.42	6.8	
1	1.414214	16.56	16.56	15.672	15.665	15.665	15.665	6.56	6.58	8.81	9.11	6.72	
2	2.236068	16.525	16.51	15.672	15.665	15.665	15.665	6.6	6.6	9.08	9.11	6.62	
5	3.162278	16.51	16.11	15.672	15.665	15.665	15.665	6.61	6.61	9.32	9.11	6.56	
10	5.477226	16.11	16.11	15.672	15.665	15.665	15.665	6.625	6.625	9.46	9.11	6.51	
30	7.745967	16.11	16.11	15.672	15.665	15.665	15.665	6.63	6.63	9.59	9.11	6.43	
60	10.95445	16.11	16.11	15.672	15.665	15.665	15.665	6.63	6.63	9.59	9.11	6.43	
120	15.49193	16.11	16.11	15.672	15.665	15.665	15.665	6.63	6.63	9.59	9.11	6.43	
240	21.9089	16.11	16.11	15.672	15.665	15.665	15.665	6.63	6.63	9.59	9.11	6.43	
480	37.94733	16.11	16.11	15.672	15.665	15.665	15.665	6.63	6.63	9.59	9.11	6.43	
1440	37.94733	16.11	16.11	15.672	15.665	15.665	15.665	6.63	6.63	9.59	9.11	6.43	
Nett Compr. over inc. (mm)	1.36	0.16	0.87	0.87	2	3.72	4.57	4.57	-0.49	-1.145	1.36	1.32	
Cumulative compr.	0.136	0.016	0.087	0.087	0.2	0.372	0.457	0.457	-0.049	-0.1145	0.136	0.132	
Cumul. correction	0.136	0.152	0.239	0.239	0.439	0.811	1.268	1.268	1.219	1.1045	1.107	1.239	
Nett Cum. Compr.	0.027	0.0395	0.06	0.06	0.0825	0.10575	0.1325	0.1325	0.1295	0.1031	0.1295	0.1484	
Final sample height	0.109	0.1125	0.179	0.179	0.3565	0.70525	1.1355	1.1355	1.0895	0.8199	0.9775	1.0906	
Vol. Strain	19.241	19.2375	19.171	18.9935	18.64475	18.2145	18.2605	18.2605	18.2605	18.5301	18.496	18.2594	
	0	-0.00563	-0.00581	-0.00925	-0.01842	-0.03645	-0.05868	-0.05868	-0.0563	-0.04237	-0.04413	-0.05052	

Rosebank Series 3

Sample R3-2 (Machine B)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 3
 Sample No.: R3-3
 Machine No: C
 Particle Density: 2.755

Ring Data: Ring # C3 Sample Parameters

Diameter:	70.050		Initial	Final
Height:	19.176	Voids Ratio e	0.908	0.990
Mass:	82.800	Saturation Sr	0.378	1.114
Area:	3853.951	Dry Density	1.444	1.384
Volume:	73903.360			
Ht aft tst	19.999			
Vol aft.	77075.161			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	381.500	237.110
Ring + Tray	261.500	87.700
Dry Soil + Ring + Tray		194.410
Wet Soil	120.000	149.410
Dry Soil	106.710	106.710
Moisture content	0.125	0.400
Moist Density (kg/m ³)	1623.742	1938.497

Moisture Content from trimmings

Containers	1	2	3	4	5
	178	85	206	223	
Wet Soil+ cont	19.500	21.900	24.900	21.500	
Dry soil + cont	18.300	20.300	23.300	19.900	
Container	2.100	2.100	2.200	2.100	
Water	1.200	1.600	1.600	1.600	0.000
Dry Soil	16.200	18.200	21.100	17.800	0.000
Moisture Content	0.074	0.088	0.076	0.090	NA
Average		8.193 %			

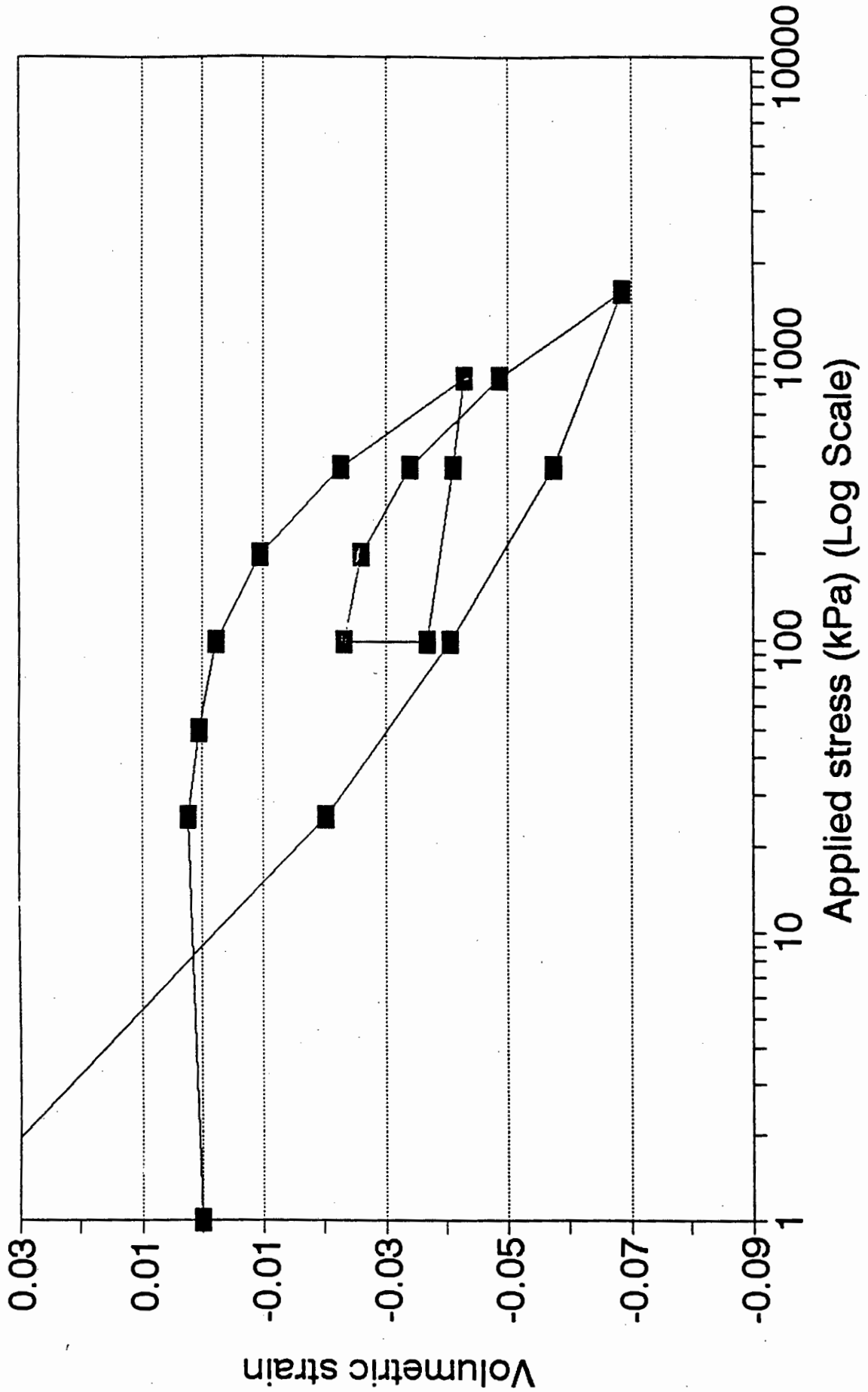
CONSOLIDATION TEST: SETTLEMENT READINGS

Rosebank Clay Samples

Test Series Code:	3	Loading Stone Diameter						1	2	3	4	5	6
Sample No:	R3-3	Avg:						68.5	68.2	68.8	68.2	68.2	68.2
Machine Number	C	Area (mm ²)						3669.162					
		Area (m ²)						0.003669					
Init. sample ht	19.176	Loading Arm Ratio						10.56604					
Load Inc.	1	2	3	4	5	6	7	8	9	10	11	12	
Load (kg)	0.885	1.75	3.5	7	14	28	14	3.5	water	7	14	28	
Load (kPa)	1	25.00101	49.43702	98.87404	197.7481	395.4962	790.9923	395.4962	98.87404	197.7481	395.4962	790.9923	
Time (min)	SQRT (time)	DIAL GAUGE READINGS											
0	0	10.98	11.4	11.1	10.55	9.12	6.61	2.72	3.05	3.88	6.51	5.99	4.408
0.5	0.707107	9.96	11.3	10.85	9.58	7.21	3.5	2.96	3.56	3.885	4.9	4.9	2.84
1	1	9.95	11.295	10.83	9.54	7.12	3.38	2.97	3.59	3.89	4.85	4.85	2.75
2	1.414214	9.93	11.28	10.76	9.49	7.05	3.27	2.99	3.62	3.98	4.78	4.78	2.51
5	2.236068	9.91	11.26	10.76	9.44	6.98	3.14	2.995	3.65	4.38	4.7	4.7	2.28
10	3.162278	9.9	11.248	10.76	9.39	6.92	3.06	3	3.7	4.72	2.1	2.1	1.89
30	5.477226	9.93	11.22	10.76	9.39	6.84	2.96	3.02	3.76	5.48	1.89	1.89	1.8
60	7.745967	9.97	11.2	10.76	9.39	6.79	2.89	3.03	3.81	5.87	1.8	1.8	1.7
120	10.95445		11.18	10.76	9.39	6.72	2.84	3.038	3.81	6.21	1.7	1.7	1.66
240	15.49193		11.158	10.76	9.39	6.7	2.79	3.045	3.845	6.38			
480	21.9089		11.12	10.57	9.12	6.66	2.74	3.05	3.88	6.51	5.99	4.408	1.62
1440	37.94733	11.4	11.1	10.55	9.12	6.61	2.72	3.05	3.88	6.51	5.99	4.408	1.62
Nett Compr. over inc.	(mm)	-0.42	0.3	0.55	1.43	2.51	3.89	-0.33	-0.83	-2.63	0.52	1.582	2.788
		-0.042	0.03	0.055	0.143	0.251	0.389	-0.033	-0.083	-0.263	0.052	0.1582	0.2788
Cumulative compr.		-0.042	-0.012	0.043	0.186	0.437	0.826	0.793	0.71	0.447	0.499	0.6572	0.936
Cumul. correction		0.019	0.03	0.0485	0.0665	0.0985	0.1095	0.1	0.0855	0.0855	0.0991	0.112	0.1276
Nett Cum. Compr.		-0.061	-0.042	-0.0055	0.1195	0.3485	0.7165	0.693	0.6245	0.3615	0.3999	0.5452	0.8084
Final sample height		19.237	19.218	19.1815	19.0565	18.8275	18.4595	18.483	18.5515	18.8145	18.7761	18.6308	18.3676
Vol. Strain	0	0.003181	0.00219	0.000287	-0.00623	-0.01817	-0.03736	-0.03614	-0.03257	-0.01885	-0.02085	-0.02843	-0.04216

Rosebank Series 3

Sample R3-3 (Machine C)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 3
 Sample No.: R3-4
 Machine No: D
 Particle Density: 2.755

Ring Data: Ring # D1 Sample Parameters

Diameter:	70.087		Initial	Final
Height:	19.125	Voids Ratio e	0.771	0.849
Mass:	81.800	Saturation Sr	0.517	1.278
Area:	3858.023	Dry Density	1.556	1.490
Volume:	73784.692			
Ht aft tst	19.970			
Vol aft.	77043.950			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	391.900	246.730
Ring + Tray	260.500	86.720
Dry Soil + Ring + Tray		201.530
Wet Soil	131.400	160.010
Dry Soil	114.810	114.810
Moisture content	0.144	0.394
Moist Density (kg/m ³)	1780.857	2076.867

Moisture Content from trimmings

Containers	1	2	3	4	5
	226	180	181		
Wet Soil+ cont	14.600	16.200	20.400		
Dry soil + cont	13.500	15.100	19.000		
Container	2.100	2.100	2.100		
Water	1.100	1.100	1.400	0.000	0.000
Dry Soil	11.400	13.000	16.900	0.000	0.000
Moisture Content	0.096	0.085	0.083	NA	NA
Average		8.798 %			

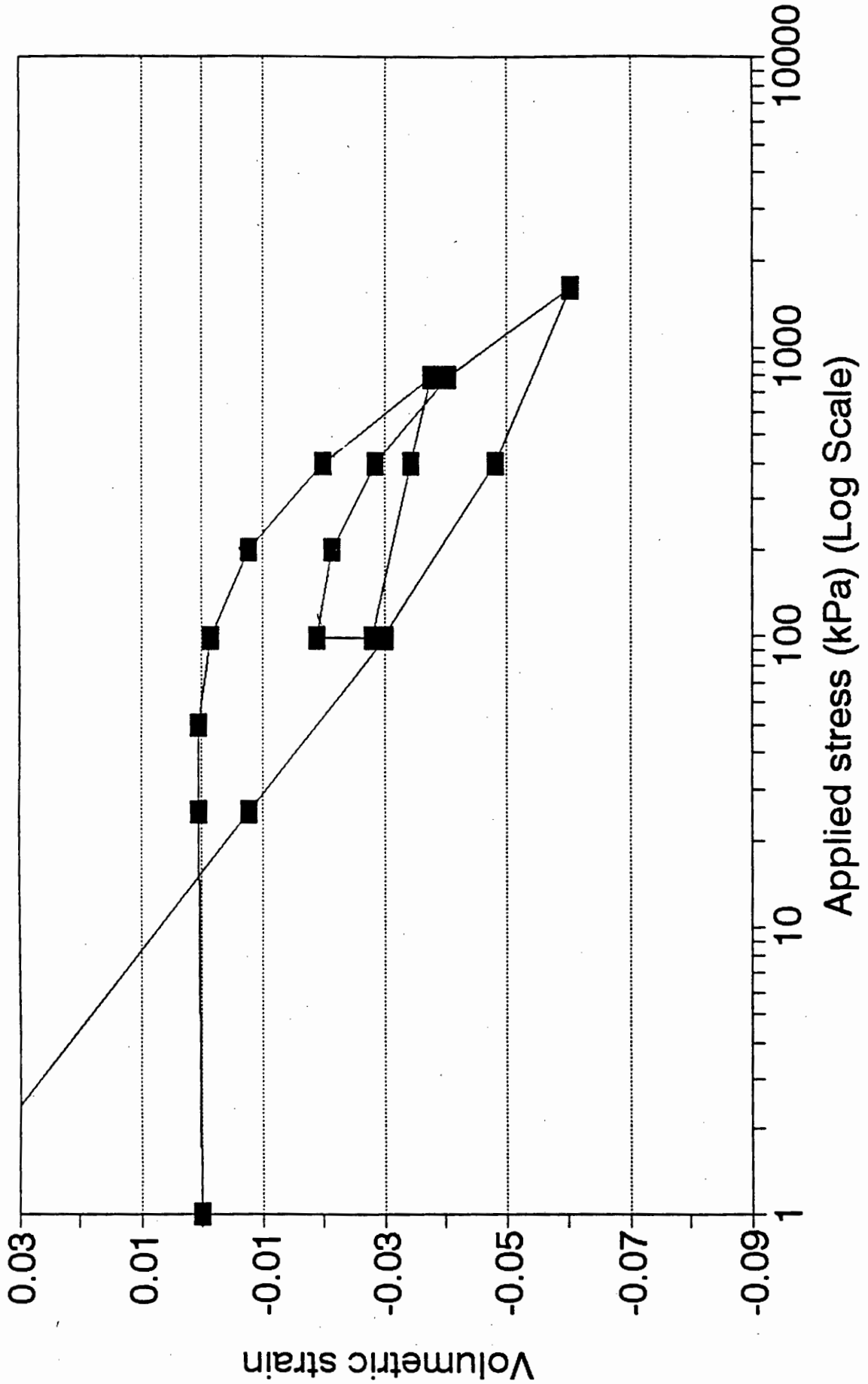
CONSOLIDATION TEST: SETTLEMENT READINGS

Rosebank Clay Samples

Test Series Code:	3	Loading Stone Diameter										
Sample No:	R3-4	Avg: 68.36667										
Machine Number	D	Area (mm ²): 3670.952										
		Area (m ²): 0.003671										
Init. sample ht	19.125	Loading Arm Ratio: 10.63878										
Load Inc.	1	2	3	4	5	6	7	8	9	10	11	12
Load (kg)	0.885	1.75	3.5	7	14	28	14	3.5	water	7	14	28
Load (kPa)	1	25.16086	49.75312	99.50624	199.0125	398.025	796.0499	398.025	99.50624	199.0125	398.025	796.0499
Time (min)	0	DIAL GUJAGE READINGS										
SQRT (time)	0	0	23.138	23.02	23.05	23.45	-0.37	1.95	4.71	1.78	2.24	3.63
	0.5	0.707107	22.7	23.08		24.16	1.25	4.3	4.15			4.6
	1	1	22.74		24.21	1.33						
	2	1.414214	22.75		24.26	1.41						
	5	2.236068	22.77		24.32							
	10	3.162278			24.37							
	30	5.477226				1.58						
	60	7.745967		23.13		1.675		5.01	4.8	2.69		5.581
	120	10.95445		23.14		1.735		5.155	4.77	2.38		5.69
	240	15.49193		23.14		1.81		5.21	4.75	2.1		5.75
	480	21.9089		23.12		1.84		5.26	4.73	1.93		5.79
	1440	37.94733		23.02		1.88		5.26	4.72			
Nett Compr. over inc.	(mm)			23.05		1.95		5.33	4.71	1.78	3.63	5.87
				-0.118		2.32		3.38	-0.62	-1.71	1.39	2.24
				-0.0118		0.4		0.338	-0.062	-0.171	0.139	0.224
				-0.0118		0.04		0.338	-0.062	-0.171	0.139	0.224
				-0.0118		0.0312		0.7192	0.6572	0.3642	0.5492	0.7732
Cumulative compr.				0.013		0.04015		0.102	0.085	0.068	0.09	0.1159
Cumul. correction				-0.0248		-0.00895		0.6172	0.5722	0.2962	0.4592	0.6573
Nett Cum. Compr.				19.1498		19.13395		18.5078	18.5528	18.8288	18.7979	18.6658
Final sample height				0		0.000468		-0.03227	-0.02992	-0.01549	-0.02401	-0.03437
Vol. Strain				0		0.000468		-0.03227	-0.02992	-0.01549	-0.02401	-0.03437

Rosebank Series 3

Sample R3-4 (Machine D)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 4
 Sample No.: R4-1
 Machine No: A
 Particle Density: 2.755

Ring Data: Ring # A2

Sample Parameters

Diameter:	76.206		Initial	Final
Height:	18.983	Voids Ratio e	0.923	0.819
Mass:	93.530	Saturation Sr	0.502	0.988
Area:	4561.086	Dry Density	1.433	1.515
Volume:	86581.718			
Ht aft tst	17.958			
Vol aft.	81906.605			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	417.400	258.770
Ring + Tray	272.470	98.268
Dry Soil + Ring + Tray		222.330
Wet Soil	144.930	160.502
Dry Soil	124.062	124.062
Moisture content	0.168	0.294
Moist Density (kg/m ³)	1673.910	1959.573

Moisture Content from trimmings

Containers	1	2	3	4	5
	124	79	77	165	192
Wet Soil+ cont	26.135	28.360	21.934	25.479	30.716
Dry soil + cont	23.716	25.938	19.700	23.218	27.820
Container	2.087	2.112	2.077	2.120	2.104
Water	2.419	2.422	2.234	2.261	2.896
Dry Soil	21.629	23.826	17.623	21.098	25.716
Moisture Content	0.112	0.102	0.127	0.107	0.113
Average		11.201	%		

CONSOLIDATION TEST: SETTLEMENT READINGS

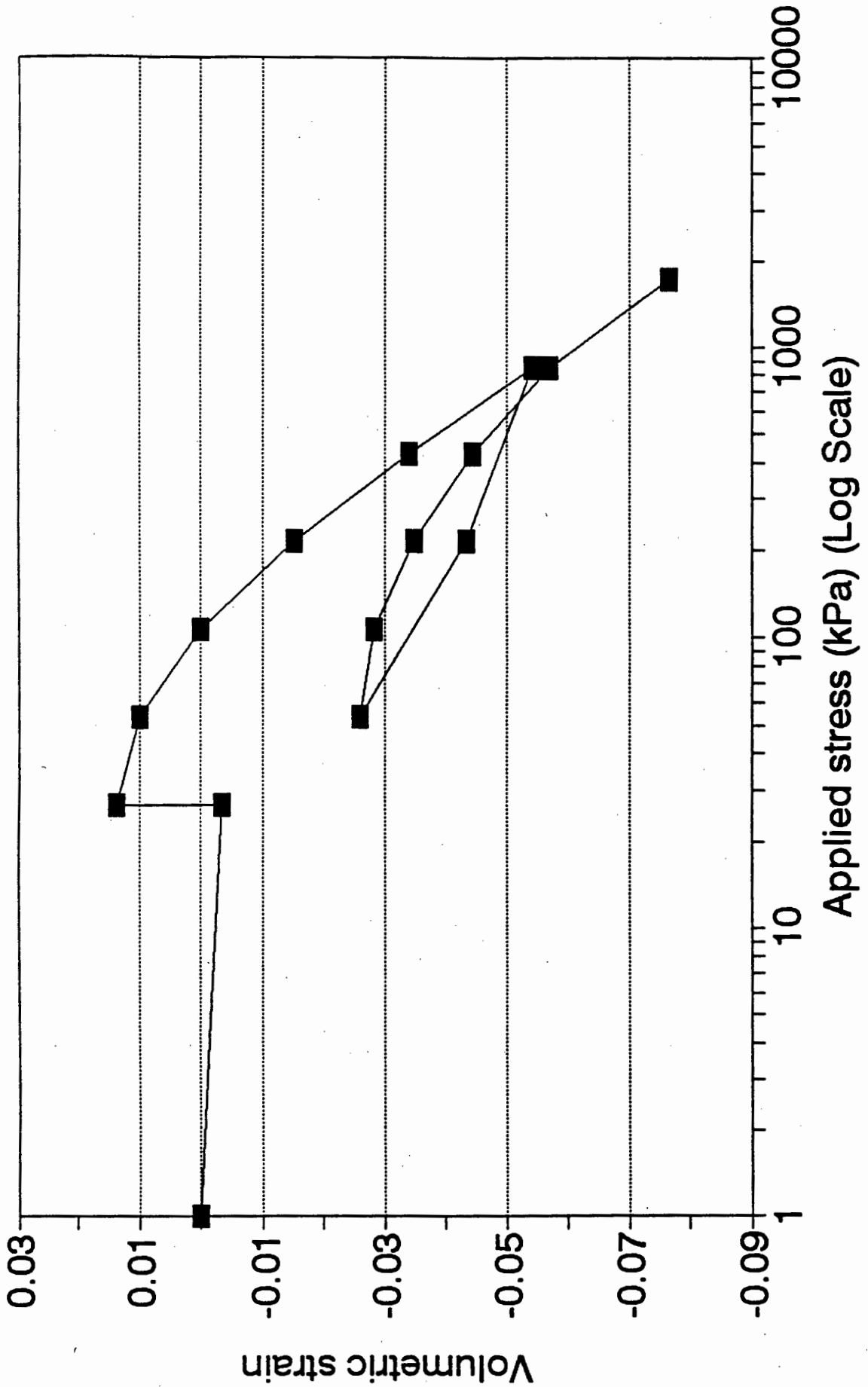
Rosebank Clay Samples

Test Series Code:	4	Loading Stone Diameter					
Sample No:	R4-1	Avg: 75.46667					
Machine Number	A	Area (mm ²): 4473.014					
		Area (m ²): 0.004473					
		Loading Arm Ratio: 10.75385					
Init. sample ht	18.9827	1	2	3	4	5	6
Load Inc.	1.135	water	2.27	4.54	9.08	18.16	36.32
Load (kg)	26.76877	26.76877	53.53755	107.0751	214.1502	428.3004	856.6007
Load (kPa)	1	1	1	1	1	1	1
Time (min)	0	0	3.54	4.215	4.215	4.215	4.215
	0.5	0.707107	4.215	4.215	4.215	4.215	4.215
	1	1	1	1	1	1	1
	2	1.414214	4.16	4.16	4.16	4.16	4.16
	5	2.236068	4.03	4.03	4.03	4.03	4.03
	10	3.162278	3.87	3.87	3.87	3.87	3.87
	30	5.477226	3.13	3.13	3.13	3.13	3.13
	60	7.745967	2.67	2.67	2.67	2.67	2.67
	120	10.95445	1.985	1.985	1.985	1.985	1.985
	240	15.49193	1.49	1.49	1.49	1.49	1.49
	480	21.9089	1.09	1.09	1.09	1.09	1.09
	1440	37.94733	4.215	4.215	4.215	4.215	4.215
Nett Compr. over inc.	(mm)	0.675	-3.255	0.71	1.87	2.9	3.58
		0.0675	-0.3255	0.071	0.187	0.29	0.358
		0.0675	-0.258	-0.187	0	0.29	0.648
Cumulative compr.		0	0	0	0	0	0
Cumul. correction		0.0675	-0.258	-0.187	0	0.29	0.648
Nett Cum. Compr.		18.9152	19.2407	19.1697	18.9827	18.6927	18.3347
Final sample height		0	-0.00356	0.013591	0	-0.01528	-0.03414
Vol. Strain							

	1	2	3	4	5	6	7	8	9	10	11	12
	75.4	75.3	75.5	75.5	75.5	75.5	75.5	75.5	75.5	75.6	75.5	75.5
	214.1502	214.1502	214.1502	214.1502	214.1502	214.1502	214.1502	214.1502	214.1502	214.1502	214.1502	214.1502
	53.53755	53.53755	53.53755	53.53755	53.53755	53.53755	53.53755	53.53755	53.53755	53.53755	53.53755	53.53755
	107.0751	107.0751	107.0751	107.0751	107.0751	107.0751	107.0751	107.0751	107.0751	107.0751	107.0751	107.0751
	856.6007	856.6007	856.6007	856.6007	856.6007	856.6007	856.6007	856.6007	856.6007	856.6007	856.6007	856.6007
	6.43	6.43	6.43	6.43	6.43	6.43	6.43	6.43	6.43	6.43	6.43	6.43
	8.21	8.21	8.21	8.21	8.21	8.21	8.21	8.21	8.21	8.21	8.21	8.21
	8.42	8.42	8.42	8.42	8.42	8.42	8.42	8.42	8.42	8.42	8.42	8.42
	8.61	8.61	8.61	8.61	8.61	8.61	8.61	8.61	8.61	8.61	8.61	8.61
	8.93	8.93	8.93	8.93	8.93	8.93	8.93	8.93	8.93	8.93	8.93	8.93
	9.19	9.19	9.19	9.19	9.19	9.19	9.19	9.19	9.19	9.19	9.19	9.19
	9.64	9.64	9.64	9.64	9.64	9.64	9.64	9.64	9.64	9.64	9.64	9.64
	9.72	9.72	9.72	9.72	9.72	9.72	9.72	9.72	9.72	9.72	9.72	9.72
	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81
	9.88	9.88	9.88	9.88	9.88	9.88	9.88	9.88	9.88	9.88	9.88	9.88
	10.01	10.01	10.01	10.01	10.01	10.01	10.01	10.01	10.01	10.01	10.01	10.01
	3.58	3.58	3.58	3.58	3.58	3.58	3.58	3.58	3.58	3.58	3.58	3.58
	0.358	0.358	0.358	0.358	0.358	0.358	0.358	0.358	0.358	0.358	0.358	0.358
	0.648	0.648	0.648	0.648	0.648	0.648	0.648	0.648	0.648	0.648	0.648	0.648
	11.8	11.8	11.8	11.8	11.8	11.8	11.8	11.8	11.8	11.8	11.8	11.8
	-2.05	-2.05	-2.05	-2.05	-2.05	-2.05	-2.05	-2.05	-2.05	-2.05	-2.05	-2.05
	0.382	0.382	0.382	0.382	0.382	0.382	0.382	0.382	0.382	0.382	0.382	0.382
	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03
	0	0	0	0	0	0	0	0	0	0	0	0
	0.648	0.648	0.648	0.648	0.648	0.648	0.648	0.648	0.648	0.648	0.648	0.648
	18.1577	18.1577	18.1577	18.1577	18.1577	18.1577	18.1577	18.1577	18.1577	18.1577	18.1577	18.1577
	-0.04346	-0.04346	-0.04346	-0.04346	-0.04346	-0.04346	-0.04346	-0.04346	-0.04346	-0.04346	-0.04346	-0.04346
	0.825	0.825	0.825	0.825	0.825	0.825	0.825	0.825	0.825	0.825	0.825	0.825
	0.496	0.496	0.496	0.496	0.496	0.496	0.496	0.496	0.496	0.496	0.496	0.496
	18.4867	18.4867	18.4867	18.4867	18.4867	18.4867	18.4867	18.4867	18.4867	18.4867	18.4867	18.4867
	-0.02613	-0.02613	-0.02613	-0.02613	-0.02613	-0.02613	-0.02613	-0.02613	-0.02613	-0.02613	-0.02613	-0.02613
	0.537	0.537	0.537	0.537	0.537	0.537	0.537	0.537	0.537	0.537	0.537	0.537
	18.3187	18.3187	18.3187	18.3187	18.3187	18.3187	18.3187	18.3187	18.3187	18.3187	18.3187	18.3187
	-0.03498	-0.03498	-0.03498	-0.03498	-0.03498	-0.03498	-0.03498	-0.03498	-0.03498	-0.03498	-0.03498	-0.03498

Rosebank Series 4

Sample R4-1 (Machine A)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 4
 Sample No.: R4-2
 Machine No: B
 Particle Density: 2.755

Ring Data: Ring # B2 Sample Parameters

Diameter:	76.237		Initial	Final
Height:	19.075	Voids Ratio e	0.881	0.781
Mass:	95.000	Saturation Sr	0.449	1.044
Area:	4564.761	Dry Density	1.465	1.547
Volume:	87072.819			
Ht aft tst	18.065			
Vol aft.	82461.497			

Weighings:

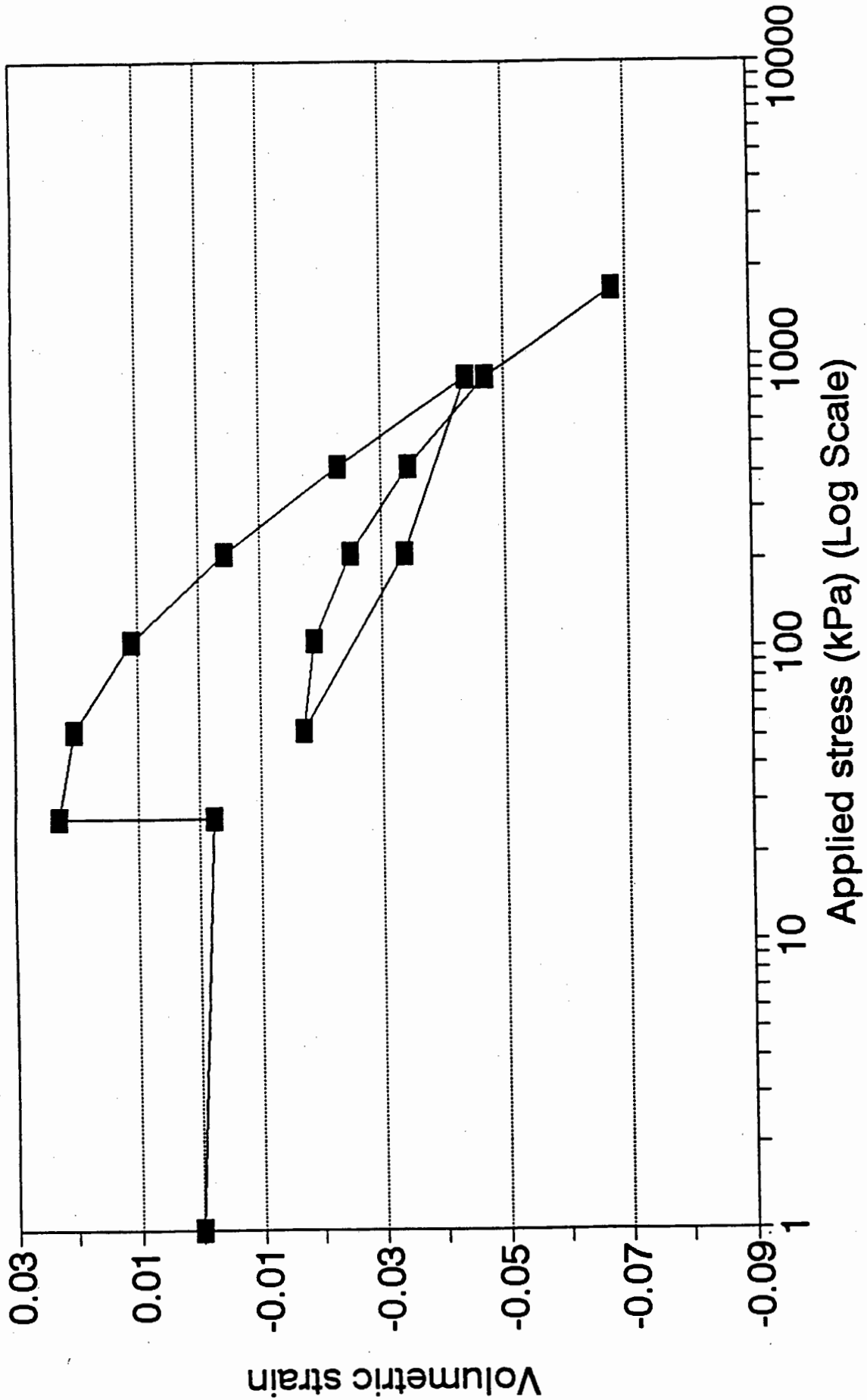
	Before Test	After Test
Wet soil + ring + Tray	419.800	265.060
Ring + Tray	273.940	99.739
Dry Soil + Ring + Tray		227.290
Wet Soil	145.860	165.321
Dry Soil	127.551	127.551
Moisture content	0.144	0.296
Moist Density (kg/m ³)	1675.150	2004.827

Moisture Content from trimmings

Containers	1	2	3	4	5
	62	26	110	35	246
Wet Soil+ cont	27.007	25.634	22.956	24.874	22.294
Dry soil + cont	24.250	23.491	21.274	22.596	20.547
Container	2.069	2.120	2.100	2.057	2.124
Water	2.757	2.143	1.682	2.278	1.747
Dry Soil	22.181	21.371	19.174	20.539	18.423
Moisture Content	0.124	0.100	0.088	0.111	0.095
Average		10.361	%		

Rosebank Series 4

Sample R4-2 (Machine B)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 4
 Sample No.: R4-3
 Machine No: C
 Particle Density: 2.755

Ring Data: Ring # C2

Sample Parameters

Diameter:	70.020		Initial	Final
Height:	19.050	Voids Ratio e	0.920	0.824
Mass:	85.320	Saturation Sr	0.556	1.034
Area:	3850.650	Dry Density	1.435	1.511
Volume:	73354.891			
Ht aft tst	18.098			
Vol aft.	69687.146			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	273.000	227.920
Ring + Tray	148.180	90.101
Dry Soil + Ring + Tray		195.380
Wet Soil	124.820	137.819
Dry Soil	105.279	105.279
Moisture content	0.186	0.309
Moist Density (kg/m ³)	1701.591	1977.682

Moisture Content from trimmings

Containers	1	2	3	4	5
	170	245	64	150	157
Wet Soil+ cont	27.989	29.723	24.573	24.513	24.090
Dry soil + cont	24.927	25.742	21.758	21.673	21.440
Container	2.137	2.112	2.089	2.094	2.117
Water	3.062	3.981	2.815	2.840	2.650
Dry Soil	22.790	23.630	19.669	19.579	19.323
Moisture Content	0.134	0.168	0.143	0.145	0.137
Average		14.563	%		

CONSOLIDATION TEST: SETTLEMENT READINGS

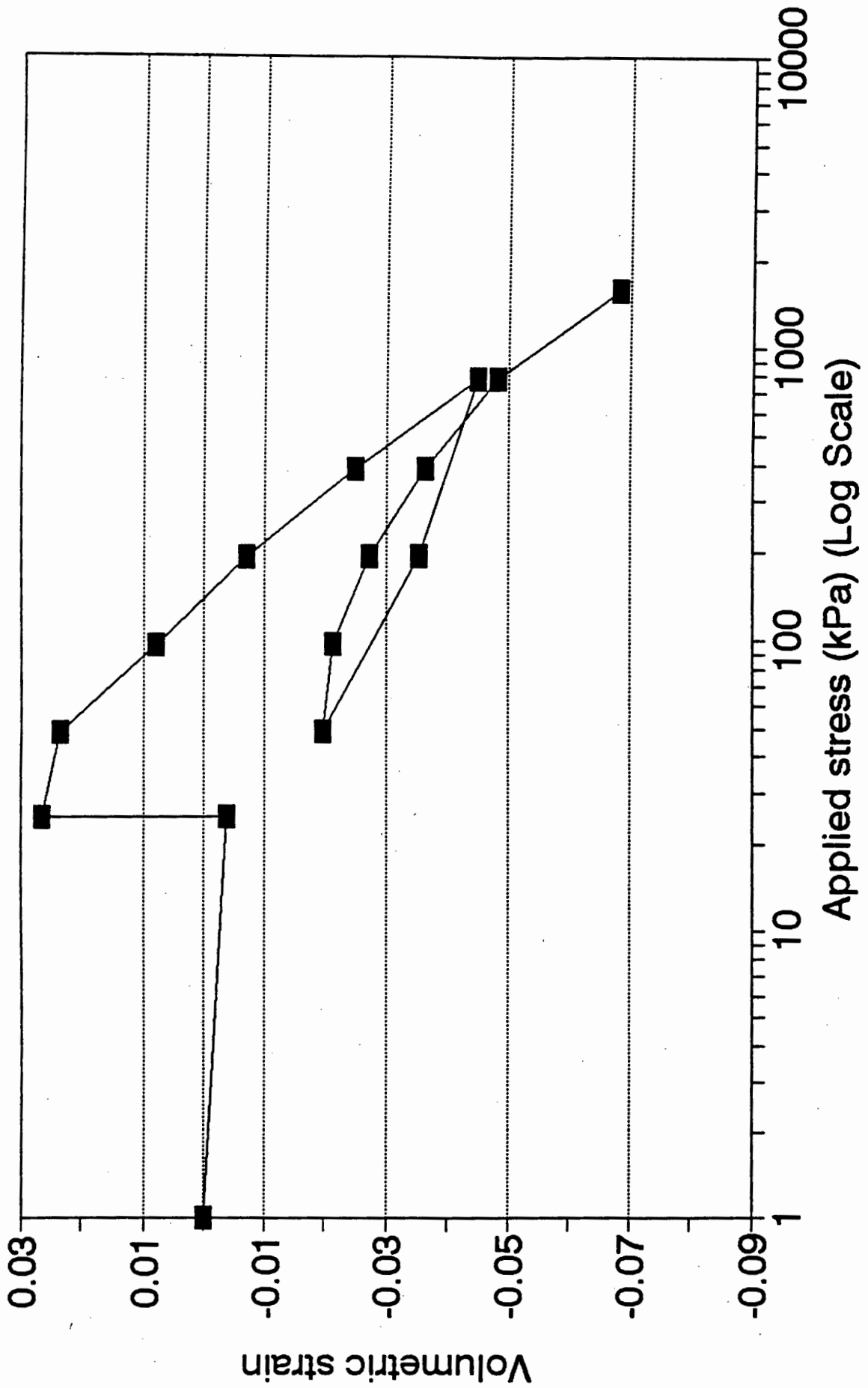
Rosebank Clay Samples

Test Series Code:	4	Loading Stone Diameter	68.35	1	2	3	4	5	6	7	8	9	10	11	12
Sample No:	R4-3	Avg:	3669.162	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
Machine Number	C	Area (mm ²)	0.003669	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
Init. sample ht	19.05	Area (m ²)	10.56604	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
Load inc.	1	Loading Arm Ratio	20.803	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
Load (kg)	0.885	2	1.75	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
Load (kPa)	1	3	3.5	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
Time (min)	0	4	10.56604	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
SQRT (time)	0	5	13.38	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
	0.5	6	10.52	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
	1	7	8.9	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
	2	8	8.72	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
	5	9	8.48	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
	10	10	8.18	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
	30	11	7.93	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
	60	12	7.57	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
	120	13	7.41	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
	240	14	7.28	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
	480	15	7.2	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
1440	37.94733	16	7.08	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
Nett Compr. over inc.	0.727	17	3.44	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
(mm)	0.0727	18	0.344	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
Cumulative compr.	0.0727	19	0.4785	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
Cumul. correction	0	20	0	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
Nett Cum. Compr.	0.0727	21	0.4785	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
Final sample height	18.9773	22	18.5715	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
Vol. Strain	0	23	-0.02512	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		24	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		25	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		26	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		27	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		28	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		29	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		30	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		31	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		32	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		33	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		34	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		35	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		36	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		37	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		38	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		39	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		40	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		41	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		42	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		43	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		44	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		45	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		46	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		47	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		48	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		49	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		50	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		51	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		52	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		53	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		54	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		55	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		56	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		57	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		58	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		59	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		60	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		61	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		62	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		63	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		64	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		65	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		66	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		67	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		68	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		69	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		70	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		71	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		72	-0.00706	68.5	68.2	68.8	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2	68.2
		73	-0.00706	68.5	68.2										

560	53								
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560	53								
10.56604	Load Inc.								
15	Load (kg)	14	15						
0	Load (kPa)	56	0						
1	Time (min)	790.9923	1581.985						
0	SQRT (time)	4.98	3.01	-0.78					
0.5		4.33	2.09						
1		4.16	1.85						
2		3.97	1.54						
5		3.625	0.98						
10		3.33	0.44						
30		2.97	-0.26						
60		2.91							
120		2.85							
240		2.8							
480		2.7	-0.78						
1440		2.28	3.79						
37.94733	Nett Compr. over inc. (mm)	0.228	0.379	-0.348					
	Cumulative compr.	0.9215	1.3005	0.9525					
	Cumul. correction	0	0	0					
	Nett Cum. Compr.	0.9215	1.3005	0.9525					
	Final sample height	18.1285	17.7495	18.0975					
	Vol. Strain	-0.04837	-0.06827	-0.05					

Rosebank Series 4

Sample R4-3 (Machine C)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 4
 Sample No.: R4-4
 Machine No: D
 Particle Density: 2.755

Ring Data: Ring # D3 Sample Parameters

Diameter:	70.170		Initial	Final
Height:	19.190	Voids Ratio e	0.882	0.869
Mass:	82.760	Saturation Sr	0.503	1.080
Area:	3867.166	Dry Density	1.464	1.474
Volume:	74210.919			
Ht aft tst	19.057			
Vol aft.	73696.586			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	270.670	233.190
Ring + Tray	144.530	87.520
Dry Soil + Ring + Tray		196.180
Wet Soil	126.140	145.670
Dry Soil	108.660	108.660
Moisture content %	16.087	34.060
Moist Density (kg/m ³)	1699.750	1976.618

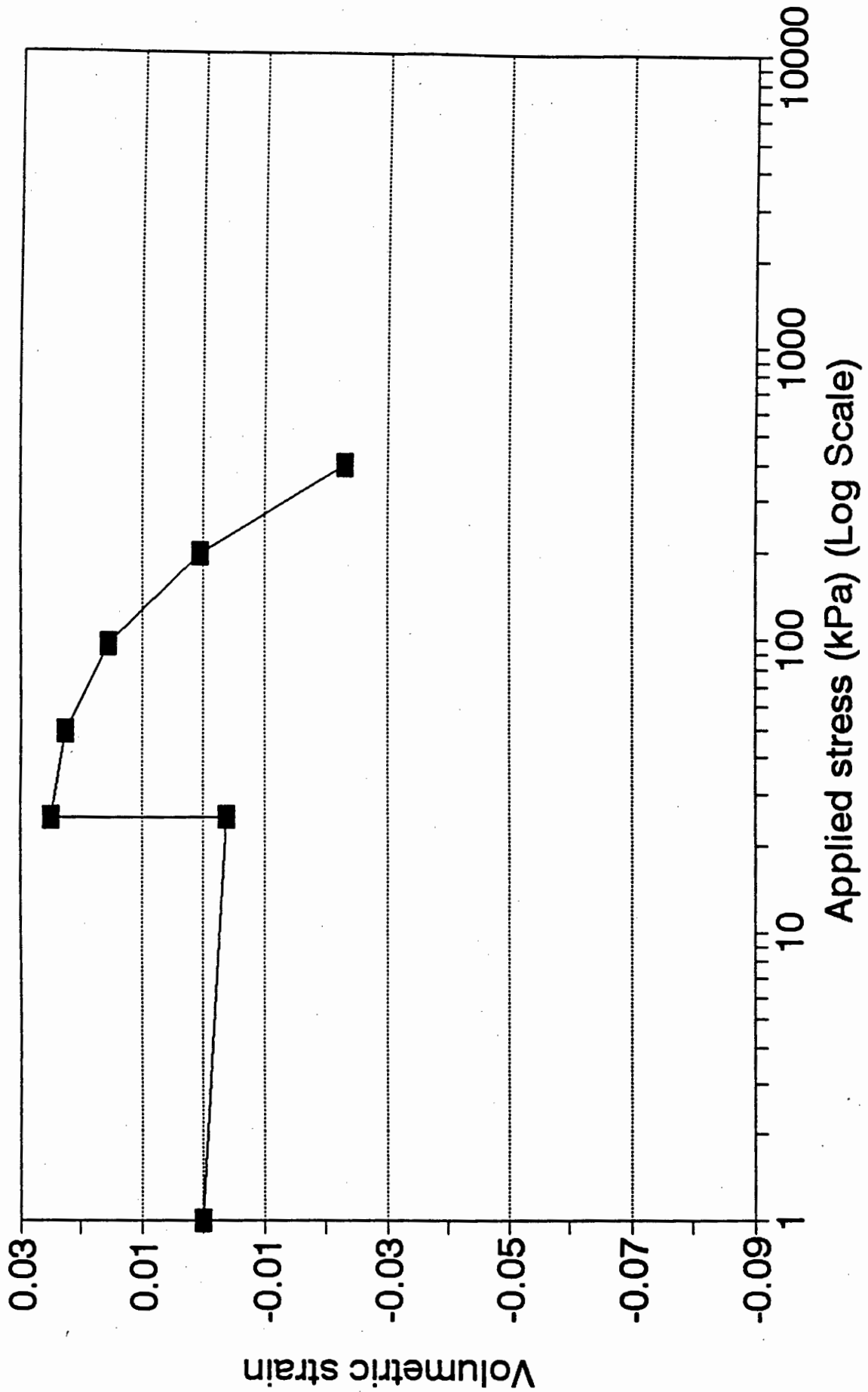
Moisture Content from trimmings

Containers	1	2	3	4	5
	190	247	5	244	59
Wet Soil+ cont	18.502	15.692	17.162	18.290	19.760
Dry soil + cont	17.412	14.516	15.860	16.858	18.436
Container	2.110	2.150	2.100	2.070	2.090
Water	1.090	1.176	1.302	1.432	1.324
Dry Soil	15.302	12.366	13.760	14.788	16.346
Moisture Content	0.0712	0.0951	0.0946	0.0968	0.0810

Average 8.78 %

Rosebank Series 4

Sample R4-4 (Machine D)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 5
 Sample No.: R5-1
 Machine No: A
 Particle Density: 2.755

Ring Data: Ring # A1

Sample Parameters

		Initial	Final
Diameter:	76.150		
Height:	19.295	Voids Ratio e	0.836 0.765
Mass:	95.980	Saturation Sr	0.383 1.258
Area:	4554.385	Dry Density	1.500 1.561
Volume:	87876.850		
Ht aft tst	18.544		
Vol aft.	84456.507		

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	306.000	278.930
Ring + Tray	158.820	101.051
Dry Soil + Ring + Tray		232.900
Wet Soil	147.180	177.879
Dry Soil	131.849	131.849
Moisture content	0.116	0.349
Moist Density (kg/m ³)	1674.844	2106.161

Moisture Content from trimmings

Containers	1	2	3	4	5
	223	180	43	178	
Wet Soil+ cont	11.726	20.086	20.348	19.224	
Dry soil + cont	11.103	18.952	18.854	18.083	
Container	2.116	2.120	2.064	2.098	
Water	0.623	1.134	1.494	1.141	0.000
Dry Soil	8.987	16.832	16.790	15.985	0.000
Moisture Content	0.069	0.067	0.089	0.071	NA

Average 7.426 %

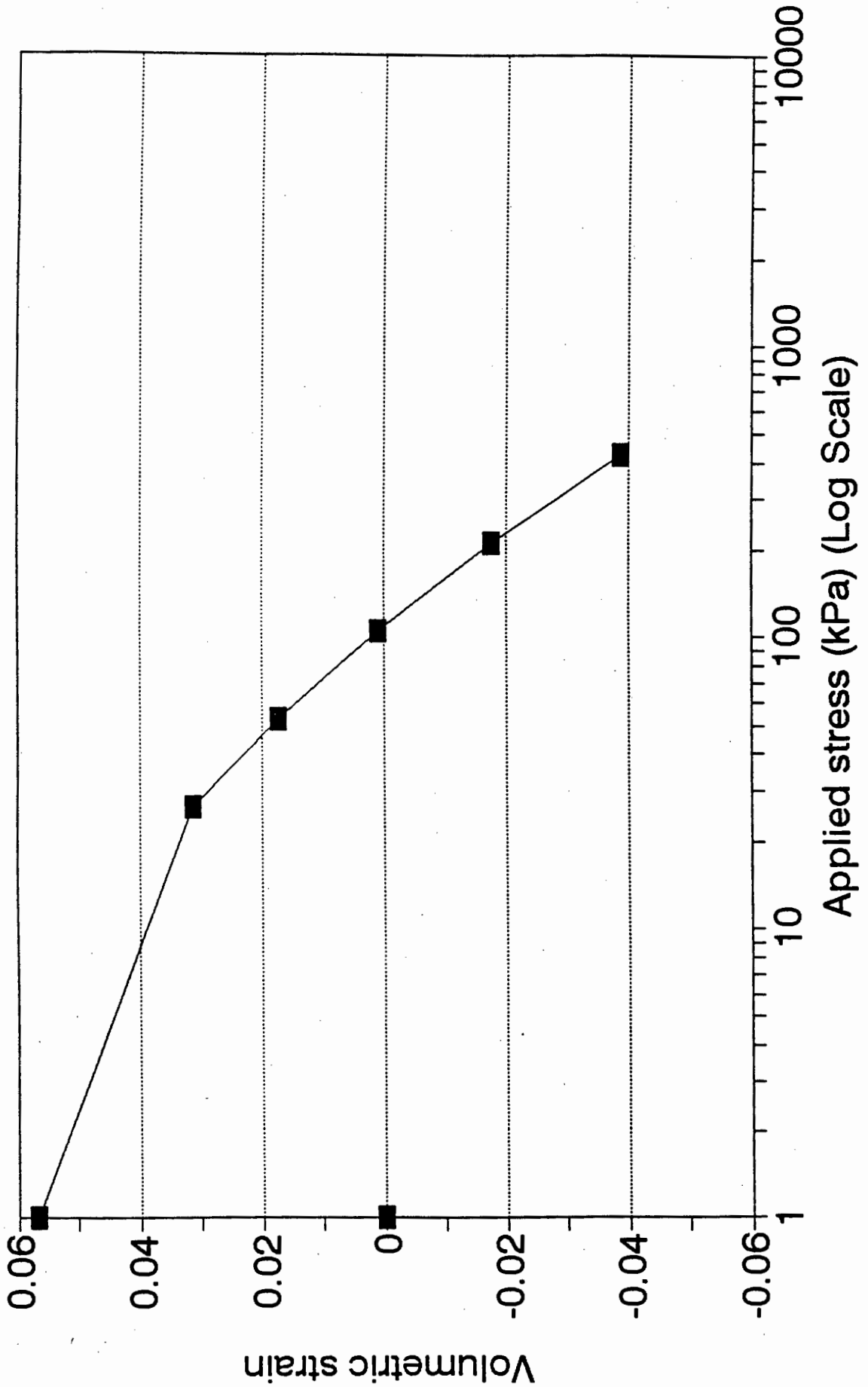
CONSOLIDATION TEST: SETTLEMENT READINGS

Rosebank Clay Samples

Test Series Code:	5	Loading Stone Diameter	1	2	3	4	5	6
Sample No:	R5-1	Avg:	75.4	75.3	75.5	75.5	75.6	75.5
Machine Number	A	Area (mm ²)	4473.014	0.004473				
		Area (m ²)	0.004473	10.75385				
Init. sample ht	19.295	Loading Arm Ratio	10.75385					
Load Inc.	1	2	3	4	5	6	7	0
Load (kg)	water	1.135	2.27	4.54	9.08	18.16		
Load (kPa)	1	26.76877	53.53755	107.0751	214.1502	428.3004		1500
Time (min)	0	10.02	0.91	3.978	6.65	9.83		
	0.5	0.707107	9.6	2.2	5.14	8.33		
	1	1	9.4	2.44	5.32	8.5		
	2	1.414214		2.7	5.5	8.7		
	5	2.236068	8.54	3.12	5.81	9		
	10	3.162278		3.39	6.03	9.26		
	30	5.477226		3.64		9.55		
	60	7.745967		3.741	6.43	9.6		
	120	10.95445			6.485	9.66		
	240	15.49193			6.53	9.71		
	480	21.9089	-0.88			9.75		
	1440	37.94733	-0.91	3.978	6.65	9.83		
Nett Compr. over inc.	(mm)	-10.93	4.888	2.672	3.18	3.57		0
		-1.093	0.4888	0.2672	0.318	0.357		0
Cumulative compr.		-1.093	-0.6042	-0.337	-0.019	0.338		0.751
Cumul. correction								
Nett Cum. Compr.		-1.093	-0.6042	-0.337	-0.019	0.338		0.751
Final sample height		20.388	19.8992	19.632	19.314	18.957		18.544
Vol. Strain	0	0.056647	0.031314	0.017466	0.000985	-0.01752		-0.03892

Rosebank Series 5

Sample R5-1 (Machine A)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 5
 Sample No.: R5-2
 Machine No: B
 Particle Density: 2.755

Ring Data: Ring # B1

Sample Parameters

Diameter:	76.060		Initial	Final
Height:	19.350	Voids Ratio e	0.982	0.925
Mass:	94.400	Saturation Sr	0.441	1.110
Area:	4543.625	Dry Density	1.390	1.431
Volume:	87919.152			
Ht aft tst	18.802			
Vol aft.	85426.974			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	400.100	267.400
Ring + Tray	258.650	99.573
Dry Soil + Ring + Tray		221.810
Wet Soil	141.450	167.827
Dry Soil	122.237	122.237
Moisture content	0.157	0.373
Moist Density (kg/m ³)	1608.864	1964.567

Moisture Content from trimmings

Containers	1	2	3	4	5
	156	153	181	216	121
Wet Soil+ cont	21.113	24.072	22.550	22.128	25.730
Dry soil + cont	19.228	22.320	20.692	20.486	23.239
Container	2.111	2.079	2.113	2.049	2.135
Water	1.885	1.752	1.858	1.642	2.491
Dry Soil	17.117	20.241	18.579	18.437	21.104
Moisture Content	0.110	0.087	0.100	0.089	0.118
Average		10.076 %			

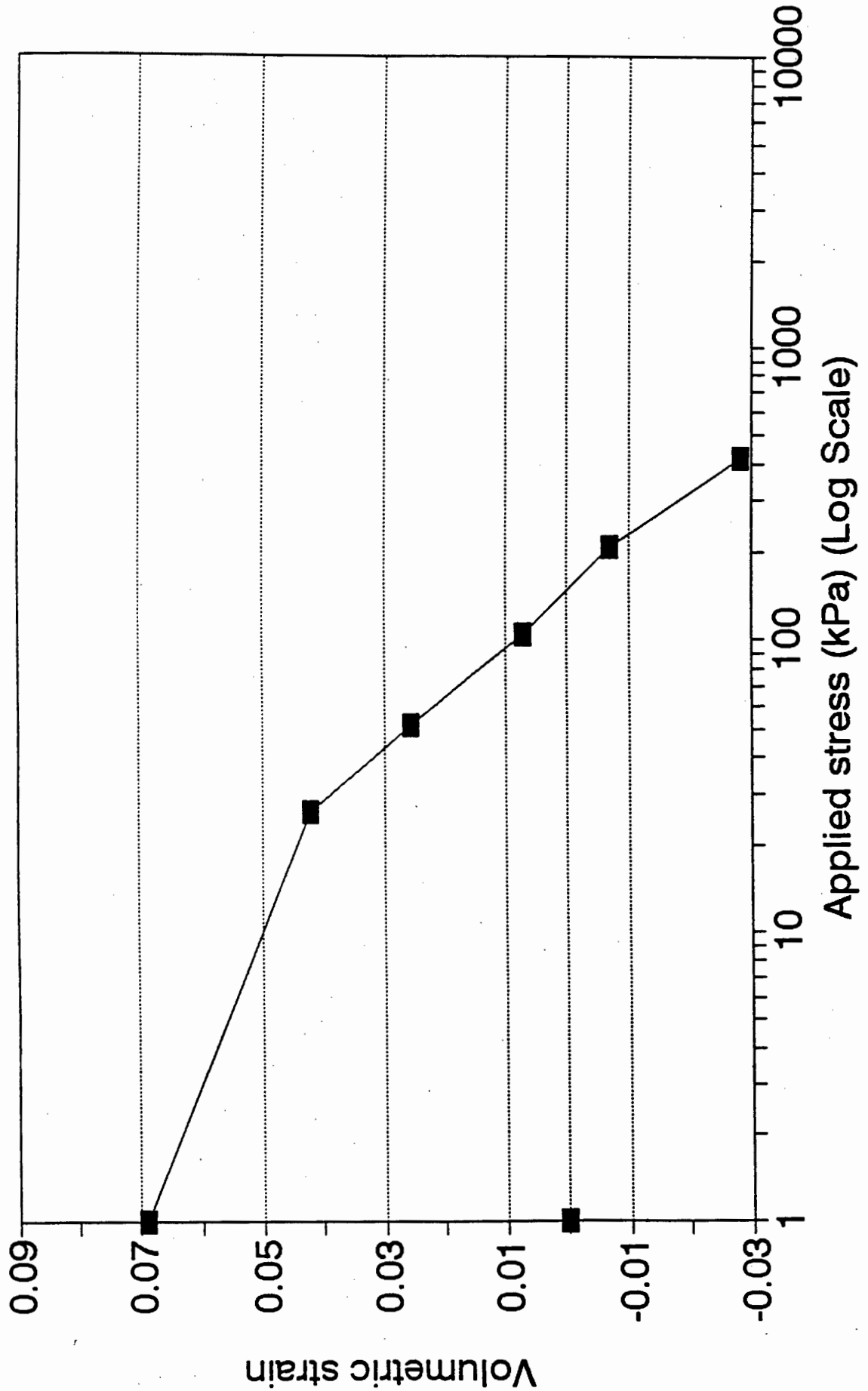
CONSOLIDATION TEST: SETTLEMENT READINGS

Rosebank Clay Samples

Test Series Code:	5	Loading Stone Diameter	75.45	1	2	3	4	5	6
Sample No:	R5-2	Avg:	4471.038	75.6	75.3	75.5	75.3	75.5	75.5
Machine Number	B	Area (mm ²)	0.004471						
		Area (m ²)	10.55094						
Init. sample ht	19.35	Loading Arm Ratio	105.1012						
Load Inc.	1	2	3	4	5	6	7		
Load (kg)	water	1.135	2.27	4.54	9.08	18.16	1.135		
Load (kPa)	1	26.27531	52.55061	105.1012	210.2025	420.4049	1		
Time (min)	SQRT (time)			DIAL GUAGE READINGS					
0	0	12.09	26	20.775	17.622	13.04	10.28		
0.5	0.707107		23.45	19.41	15.75	12.03	8.26		
1	1	13.65	23.04	19.18	15.56	11.72	7.97		
2	1.414214		22.54	18.94	15.32	11.55	7.63		
5	2.236068		21.94	18.58	14.98	11.18	7.08		
10	3.162278		21.56	18.315		10.89	6.85		
30	5.477226		21.15		14.34	10.59	6.42		
60	7.745967		21.06		17.85	14.28	6.362		
120	10.95445				17.79	14.21	6.305		
240	15.49193				17.74	14.16	6.263		
480	21.9089	25.04			14.12	14.12	6.22		
1440	37.94733	25.44	20.775	17.622	14.04	10.28	6.165		
Nett Compr. over inc.	(mm)	-13.35	5.225	3.153	3.582	2.76	4.115	0	
Cumulative compr.		-1.335	0.5225	0.3153	0.3582	0.276	0.4115	0	
Cumul. correction		-1.335	-0.8125	-0.4972	-0.139	0.137	0.5485	0.5485	
Nett Cum. Compr.		-1.335	-0.8125	-0.4972	-0.139	0.137	0.5485	0.5485	
Final sample height		20.685	20.1625	19.8472	19.489	19.213	18.8015	18.8015	
Vol. Strain	0	0.068992	0.04199	0.025695	0.007183	-0.00708	-0.02835	-0.02835	

Rosebank Series 5

Sample R5-2 (Machine B)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 5
 Sample No.: R5-3
 Machine No: C
 Particle Density: 2.755

Ring Data: Ring # C3

Sample Parameters

Diameter:	70.050		Initial	Final
Height:	19.176	Voids Ratio e	0.824	0.785
Mass:	82.770	Saturation Sr	0.319	1.214
Area:	3853.951	Dry Density	1.510	1.543
Volume:	73903.360			
Ht aft tst	18.766			
Vol aft.	72323.240			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	266.760	238.280
Ring + Tray	144.500	88.031
Dry Soil + Ring + Tray		199.650
Wet Soil	122.260	150.249
Dry Soil	111.619	111.619
Moisture content	0.095	0.346
Moist Density (kg/m ³)	1654.323	2077.465

Moisture Content from trimmings

Containers	1	2	3	4	5
	70	120	205		
Wet Soil+ cont	19.106	19.637	18.332		
Dry soil + cont	18.137	18.709	17.491		
Container	2.132	2.101	2.608		
Water	0.969	0.928	0.841	0.000	0.000
Dry Soil	16.005	16.608	14.883	0.000	0.000
Moisture Content	0.061	0.056	0.057	NA	NA
Average		5.764 %			

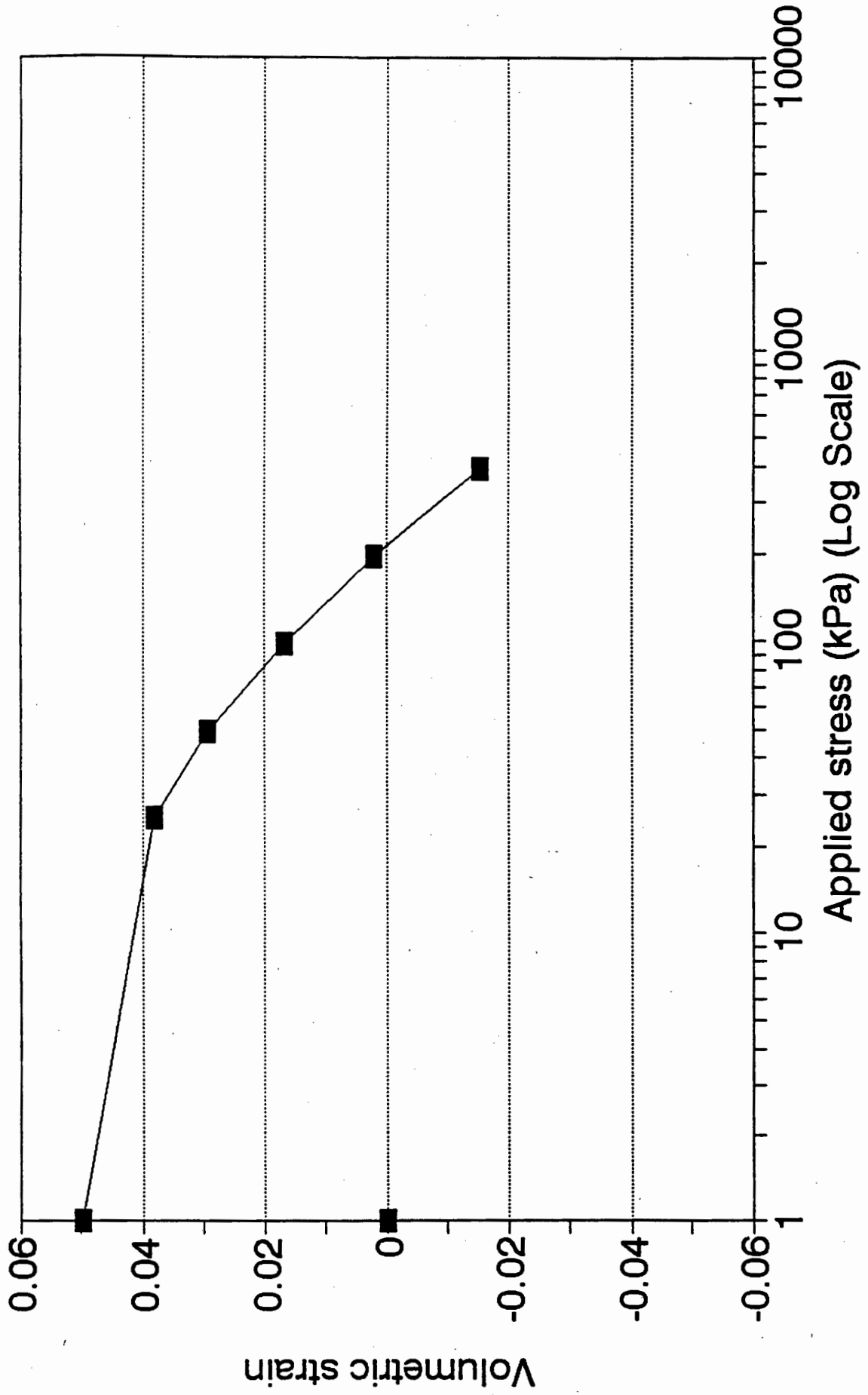
CONSOLIDATION TEST: SETTLEMENT READINGS

Rosebank Clay Samples

Test Series Code:	5	Loading Stone Diameter	1	2	3	4	5	6
Sample No:	R5-3	Avg:	68.35	68.2	68.8	68.2	68.2	68.2
Machine Number	C	Area (mm ²)	3669.162					
		Area (m ²)	0.003669					
Init. sample ht	19.176	Loading Arm Ratio	10.56604					
Load inc.	1	2	3	4	5	6	7	
Load (kg)	water	0.885	1.75	3.5	7	14	0	
Load (kPa)	1	25.00101	49.43702	98.87404	197.7481	395.4962	1	1500
Time (min)	SQRT (time)			DIAL GUAGE READINGS				
0	0	13.5	22.83	20.5	18.63	16.01	12.962	
0.5	0.707107	13.85	21.32	19.63	17.25	14.45	11.05	
1	1	14.02	21.16	19.51	17.1	14.15	10.85	
2	1.414214	14.24	21.04	19.38	16.94	13.98	101.61	
5	2.236068		20.9	19.15	16.7	13.71	10.27	
10	3.162278		20.818	19.05	16.55	13.68	10.025	
30	5.477226		20.71		16.3	13.33	9.68	
60	7.745967		20.67		16.23	13.18	9.61	
120	10.95445				16.73	13.11	9.55	
240	15.49193				16.68	13.07	9.51	
480	21.9089				16.07		9.46	
1440	37.94733				16.01	12.962	9.4	
Nett Compr. over inc. (mm)		22.83	20.5	18.63	16.01	12.962	9.4	
		-9.33	2.33	1.87	2.62	3.048	3.562	0
		-0.933	0.233	0.187	0.262	0.3048	0.3562	0
		-0.933	-0.7	-0.513	-0.251	0.0538	0.41	0.41
Cumulative compr.								
Cumul. correction								
Nett Cum. Compr.		-0.933	-0.7	-0.513	-0.251	0.0538	0.41	0.41
Final sample height		20.109	19.876	19.689	19.427	19.1222	18.766	18.766
Vol. Strain	0	0.048655	0.036504	0.026752	0.013089	-0.00281	-0.02138	-0.02138

Rosebank Series 5

Sample R5-3 (Machine C)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 5
 Sample No.: R5-4
 Machine No: D
 Particle Density: 2.755

Ring Data: Ring # D1 Sample Parameters

Diameter:	70.087		Initial	Final
Height:	19.125	Void Ratio e	0.821	0.806
Mass:	81.810	Saturation Sr	0.440	1.161
Area:	3858.023	Dry Density	1.513	1.525
Volume:	73784.692			
Ht aft tst	18.976			
Vol aft.	73207.917			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	386.900	236.590
Ring + Tray	260.610	86.984
Dry Soil + Ring + Tray		198.640
Wet Soil	126.290	149.606
Dry Soil	111.656	111.656
Moisture content	0.131	0.340
Moist Density (kg/m ³)	1711.602	2043.577

Moisture Content from trimmings

Containers	1	2	3	4	5
	226	143	227	175	147
Wet Soil+ cont	23.643	23.651	22.717	21.447	25.193
Dry soil + cont	21.296	22.118	20.974	20.176	23.296
Container	2.098	2.093	2.119	2.055	2.094
Water	2.347	1.533	1.743	1.271	1.897
Dry Soil	19.198	20.025	18.855	18.121	21.202
Moisture Content	0.122	0.077	0.092	0.070	0.089
Average		9.017	%		

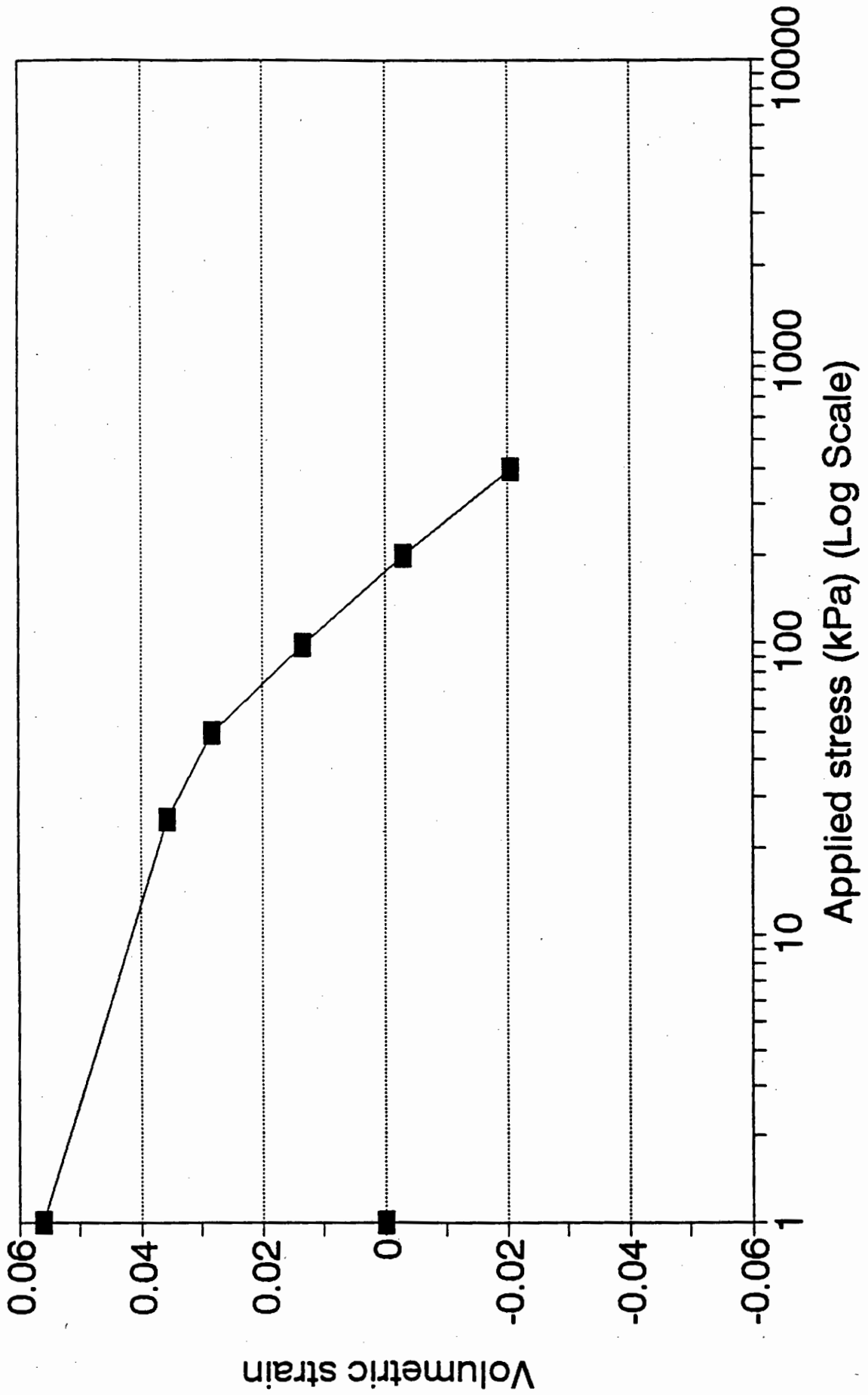
CONSOLIDATION TEST: SETTLEMENT READINGS

Rosebank Clay Samples

Test Series Code:	5	Loading Stone Diameter	1	2	3	4	5	6
Sample No:	R5-4	Avg:	68.4	68.6	68.2	68.4	68.2	68.4
Machine Number	D	Area (mm ²)	3670.952					
		Area (m ²)	0.003671					
Init. sample ht	19.125	Loading Arm Ratio	10.63878					
Load Inc.	1			7				
Load (kg)	water			0				
Load (kPa)	1	25.16086	49.75312	99.50624	199.0125	398.025	1500	
Time (min)	SQRT (time)							
0	0	23.94	13.239	17.12	18.54	21.28	-0.446	
0.5	0.707107	22.28	15.79	17.58	20	23.04	1.33	
1	1	22.02	15.98	17.68	20.18	23.22	1.52	
2	1.414214		16.19		20.38	23.43	1.76	
5	2.236068		16.5	17.915	20.62	23.71	2.115	
10	3.162278		16.7	18.078	20.86	23.925	2.32	
30	5.477226		16.92		21.05	24.162	2.65	
60	7.745967		16.98	18.346	21.16	24.24	2.71	
120	10.95445			18.4	21.22	24.3	2.765	
240	15.49193			18.44	21.26	24.34	2.805	
480	21.9089				21.3		2.85	
1440	37.94733							
Nett Compr. over inc.	(mm)	13.32	17.12	18.54	21.38	24.445	2.9	
		-10.701	3.881	1.42	2.84	3.165	3.346	0
Cumulative compr.		-1.0701	0.3881	0.142	0.284	0.3165	0.3346	0
Cumul. correction		-1.0701	-0.682	-0.54	-0.256	0.0605	0.3951	0.3951
Nett Cum. Compr.		-1.0701	-0.682	-0.54	-0.256	0.0605	0.3951	0.3951
Final sample height		20.1951	19.807	19.665	19.381	19.0645	18.7299	18.7299
Vol. Strain	0	0.055953	0.03566	0.028235	0.013386	-0.00316	-0.02066	-0.02066

Rosebank Series 5

Sample R5-4 (Machine D)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 6
 Sample No.: R6-1
 Machine No: A
 Particle Density: 2.755

Ring Data: Ring # A2 Sample Parameters

Diameter:	76.206		Initial	Final
Height:	18.983	Voids Ratio e	0.865	0.949
Mass:	93.530	Saturation Sr	0.533	1.215
Area:	4561.086	Dry Density	1.477	1.413
Volume:	86581.718			
Ht aft tst	19.839			
Vol aft.	90486.007			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	305.710	275.900
Ring + Tray	156.430	94.466
Dry Soil + Ring + Tray		222.360
Wet Soil	149.280	181.434
Dry Soil	127.894	127.894
Moisture content	0.167	0.419
Moist Density (kg/m ³)	1724.152	2005.106

Moisture Content from trimmings

Containers	1	2	3	4	5
	E1	F3-LL	A12	165	192
Wet Soil+ cont	20.697	23.616	25.044	21.189	35.331
Dry soil + cont	18.194	21.036	22.121	18.792	31.359
Container	2.106	2.092	2.110	2.126	2.108
Water	2.503	2.580	2.923	2.397	3.972
Dry Soil	16.088	18.944	20.011	16.666	29.251
Moisture Content	0.156	0.136	0.146	0.144	0.136

Average 14.349 %

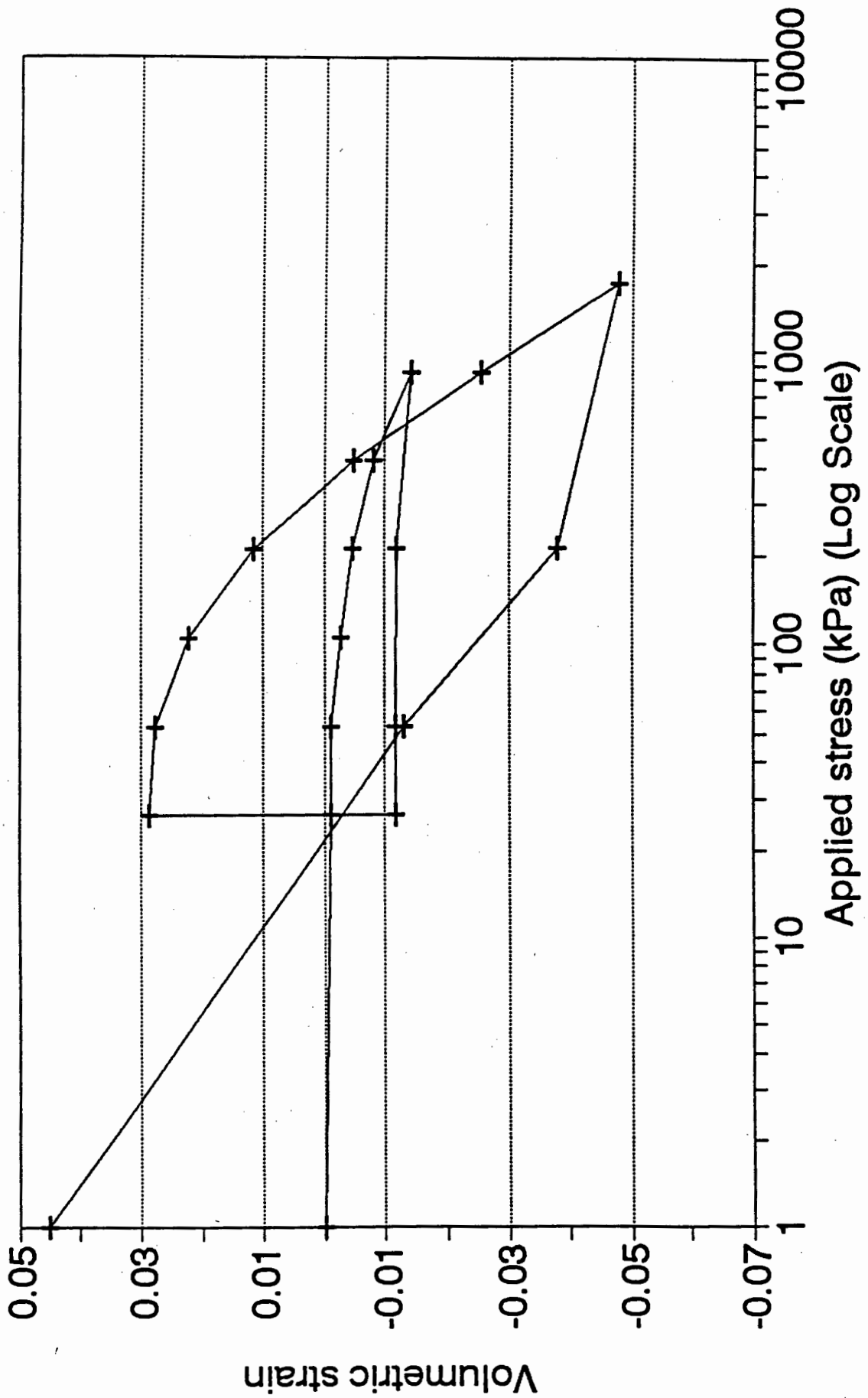
CONSOLIDATION TEST: SETTLEMENT READINGS

Rosebank Clay Samples

Test Series Code:	6	Loading Stone Diameter											
Sample No:	R6-1	Avg:											
Machine Number	A	Area (mm ²)											
		Area (m ²)											
		Loading Arm Ratio											
Init. sample ht	18.9827	DIAL GAUGE READINGS											
Load Inc.	1	2	3	4	5	6	7	8	9	10	11	12	
Load (kg)	1.135	2.27	4.54	9.08	18.16	36.32	9.08	2.27	1.135	water	2.27	4.54	
Load (kPa)	1	26.76877	53.53755	107.0751	214.1502	428.3004	856.6007	214.1502	53.53755	26.76877	26.76877	53.53755	107.0751
Time (min)	0	0	16.84	17.395	17.66	18.86	19.84	21.3	20.825	20.49	20.42	12.86	13.232
SQRT (time)	0.5	0.707107	17.32	17.33	17.34	17.36	17.37	17.38	17.39	17.4	17.42	17.43	17.44
	1	1.414214	17.33	17.34	17.36	17.37	17.38	17.39	17.4	17.42	17.43	17.44	17.45
	2	2.236068	17.33	17.34	17.36	17.37	17.38	17.39	17.4	17.42	17.43	17.44	17.45
	5	2.236068	17.33	17.34	17.36	17.37	17.38	17.39	17.4	17.42	17.43	17.44	17.45
	10	3.162278	17.33	17.34	17.36	17.37	17.38	17.39	17.4	17.42	17.43	17.44	17.45
	30	5.477226	17.33	17.34	17.36	17.37	17.38	17.39	17.4	17.42	17.43	17.44	17.45
	60	7.745967	17.33	17.34	17.36	17.37	17.38	17.39	17.4	17.42	17.43	17.44	17.45
	120	10.95445	17.33	17.34	17.36	17.37	17.38	17.39	17.4	17.42	17.43	17.44	17.45
	240	15.49193	17.33	17.34	17.36	17.37	17.38	17.39	17.4	17.42	17.43	17.44	17.45
	480	21.9089	17.33	17.34	17.36	17.37	17.38	17.39	17.4	17.42	17.43	17.44	17.45
1440	37.94733	17.395	17.66	18.16	18.86	19.84	21.3	20.825	20.49	20.42	12.86	13.232	
Nett Compr. over inc. (mm)	0.555	0.265	0.5	0.7	0.98	1.46	1.46	-0.475	-0.335	0	-7.56	0.372	1.238
Cumulative compr.	0.0555	0.0265	0.05	0.07	0.098	0.146	0.146	-0.0475	-0.0335	0	-0.756	0.0372	0.1238
Cumul. correction	0.0555	0.082	0.132	0.202	0.3	0.446	0.446	0.3985	0.365	0.365	-0.391	-0.3538	-0.23
Nett Cum. Compr.	0.036	0.06195	0.084	0.1175	0.1445	0.175	0.175	0.173	0.143	0.143	0.159	0.173	0.193
Final sample height	0.0195	0.02005	0.048	0.0845	0.1555	0.271	0.271	0.2255	0.222	0.222	-0.55	-0.5268	-0.423
Vol. Strain	18.9632	18.96265	18.9347	18.8982	18.8272	18.7117	18.7117	18.7572	18.7607	18.7607	19.5327	19.5095	19.4057
	0	-0.00103	-0.00253	-0.00445	-0.00819	-0.01428	-0.01428	-0.01188	-0.01169	-0.01169	0.028974	0.027752	0.022283

Rosebank Series 6

Sample R6-1 (Machine A)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 6
 Sample No.: R6-2
 Machine No: B
 Particle Density: 2.755

Ring Data: Ring # B2 Sample Parameters

Diameter:	76.237		Initial	Final
Height:	19.075	Voids Ratio e	0.940	1.034
Mass:	95.000	Saturation Sr	0.468	1.160
Area:	4564.761	Dry Density	1.420	1.355
Volume:	87072.819			
Ht aft tst	19.996			
Vol aft.	91276.964			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	402.700	273.340
Ring + Tray	259.310	95.881
Dry Soil + Ring + Tray		219.540
Wet Soil	143.390	177.459
Dry Soil	123.659	123.659
Moisture content	0.160	0.435
Moist Density (kg/m ³)	1646.783	1944.182

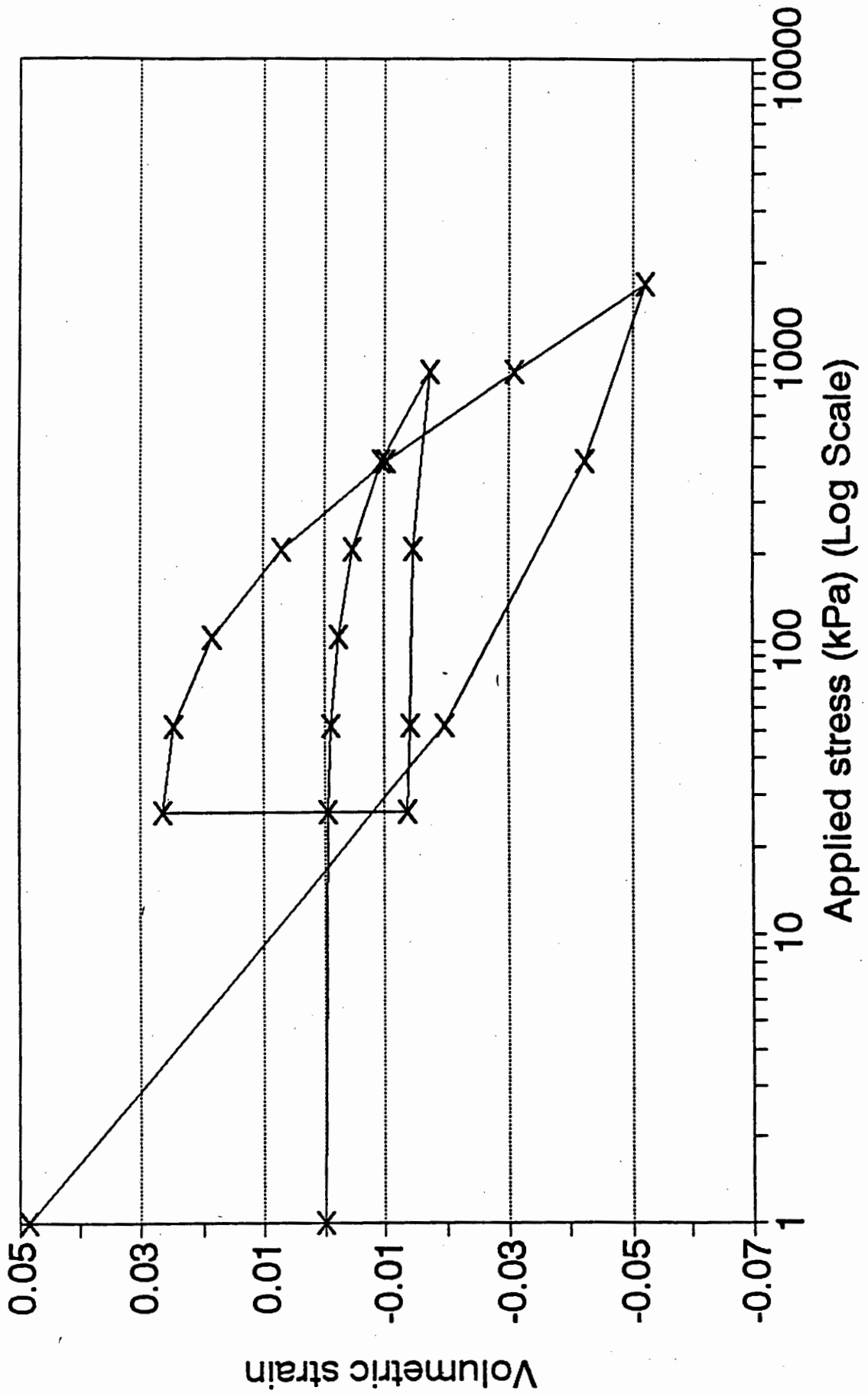
Moisture Content from trimmings

Containers	1	2	3	4	5
	62	65	77	150	246
Wet Soil+ cont	26.753	22.897	26.535	29.301	19.679
Dry soil + cont	24.355	20.435	23.651	26.213	17.550
Container	2.070	2.046	2.075	2.092	2.121
Water	2.398	2.462	2.884	3.088	2.129
Dry Soil	22.285	18.389	21.576	24.121	15.429
Moisture Content	0.108	0.134	0.134	0.128	0.138

Average 12.823 %

Rosebank Series 6

Sample R6-2 (Machine B)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 6
 Sample No.: R6-3
 Machine No: C
 Particle Density: 2.755

Ring Data: Ring # C2

Sample Parameters

Diameter:	70.020		Initial	Final
Height:	19.050	Voids Ratio e	0.788	0.839
Mass:	85.320	Saturation Sr	0.469	1.232
Area:	3850.650	Dry Density	1.540	1.498
Volume:	73354.891			
Ht aft tst	19.589			
Vol aft.	75430.391			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	275.240	241.620
Ring + Tray	147.090	86.212
Dry Soil + Ring + Tray		199.210
Wet Soil	128.150	155.408
Dry Soil	112.998	112.998
Moisture content	0.134	0.375
Moist Density (kg/m ³)	1746.986	2060.284

Moisture Content from trimmings

Containers	1	2	3	4	5
	191	72	183	8	
Wet Soil+ cont	22.190	19.618	21.075	20.944	
Dry soil + cont	19.941	17.599	18.903	18.868	
Container	2.113	2.082	2.151	2.083	
Water	2.249	2.019	2.172	2.076	0.000
Dry Soil	17.828	15.517	16.752	16.785	0.000
Moisture Content	0.126	0.130	0.130	0.124	NA
Average		12.740	%		

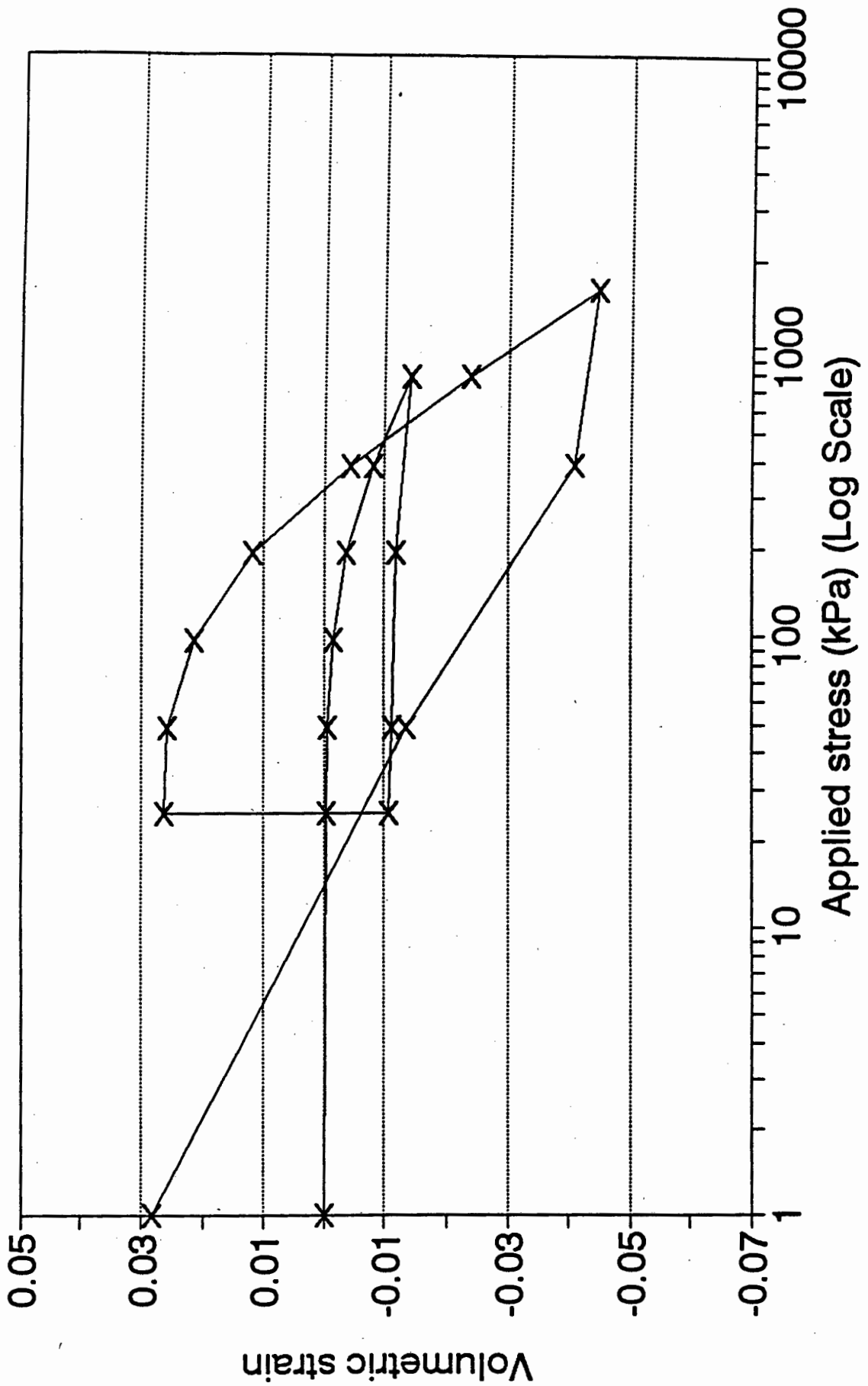
CONSOLIDATION TEST: SETTLEMENT READINGS

Rosebank Clay Samples

Test Series Code:	6	1	2	3	4	5	6	7	8	9	10	11	12
Sample No:	R6-3	19.05	1.75	3.5	7	14	28	59	61	62	65	68	72
Machine Number	C	0.885	49.43702	98.87404	197.7481	395.4962	790.9923	197.7481	49.43702	98.87404	197.7481	25.00101	49.43702
Init. sample ht	19.05	19.05	19.05	19.05	19.05	19.05	19.05	19.05	19.05	19.05	19.05	19.05	19.05
Load Inc.	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Load (kg)	1	1	1	1	1	1	1	1	1	1	1	1	1
Load (kPa)	5	5	5	5	5	5	5	5	5	5	5	5	5
Time (min)	0	0	0	0	0	0	0	0	0	0	0	0	0
SQRT (time)	0.707107	0.707107	0.707107	0.707107	0.707107	0.707107	0.707107	0.707107	0.707107	0.707107	0.707107	0.707107	0.707107
Loading Stone Diameter	68.35	68.35	68.35	68.35	68.35	68.35	68.35	68.35	68.35	68.35	68.35	68.35	68.35
Avg:	3669.162	3669.162	3669.162	3669.162	3669.162	3669.162	3669.162	3669.162	3669.162	3669.162	3669.162	3669.162	3669.162
Area (mm ^ 2)	0.003669	0.003669	0.003669	0.003669	0.003669	0.003669	0.003669	0.003669	0.003669	0.003669	0.003669	0.003669	0.003669
Area (m ^ 2)	10.56604	10.56604	10.56604	10.56604	10.56604	10.56604	10.56604	10.56604	10.56604	10.56604	10.56604	10.56604	10.56604
Loading Arm Ratio	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98
DIAL GAUGE READINGS													
Nett Compr. over inc. (mm)	9.14	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98
Cumulative compr.	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26
Cumul. correction	0.026	0.026	0.026	0.026	0.026	0.026	0.026	0.026	0.026	0.026	0.026	0.026	0.026
Nett Cum. Compr.	0.019	0.019	0.019	0.019	0.019	0.019	0.019	0.019	0.019	0.019	0.019	0.019	0.019
Final sample height	19.043	19.043	19.043	19.043	19.043	19.043	19.043	19.043	19.043	19.043	19.043	19.043	19.043
Vol. Strain	0	-0.00037	-0.00063	-0.0016	-0.00344	-0.00822	-0.01425	-0.01199	-0.01102	-0.01063	0.026409	0.025801	0.021501

Rosebank Series 6

Sample R6-3 (Machine C)



CONSOLIDATION TEST: INITIAL DATA

Test Series Code: 6
 Sample No.: R6-4
 Machine No: D
 Particle Density: 2.755

Ring Data: Ring # D2

Sample Parameters

Diameter:	70.123		Initial	Final
Height:	19.030	Voids Ratio e	0.849	0.907
Mass:	82.500	Saturation Sr	0.440	1.167
Area:	3862.024	Dry Density	1.490	1.445
Volume:	73494.313			
Ht aft tst	19.625			
Vol aft.	75790.286			

Weighings:

	Before Test	After Test
Wet soil + ring + Tray	385.700	235.010
Ring + Tray	261.350	83.427
Dry Soil + Ring + Tray		192.930
Wet Soil	124.350	151.583
Dry Soil	109.503	109.503
Moisture content	0.136	0.384
Moist Density (kg/m ³)	1691.968	2000.032

Moisture Content from trimmings

Containers	1	2	3	4	5
	12	35	157	170	236
Wet Soil+ cont	21.140	26.989	27.528	33.668	32.787
Dry soil + cont	19.248	24.496	24.928	29.976	29.318
Container	2.119	2.060	2.119	2.139	2.142
Water	1.892	2.493	2.600	3.692	3.469
Dry Soil	17.129	22.436	22.809	27.837	27.176
Moisture Content	0.110	0.111	0.114	0.133	0.128
Average		11.917	%		

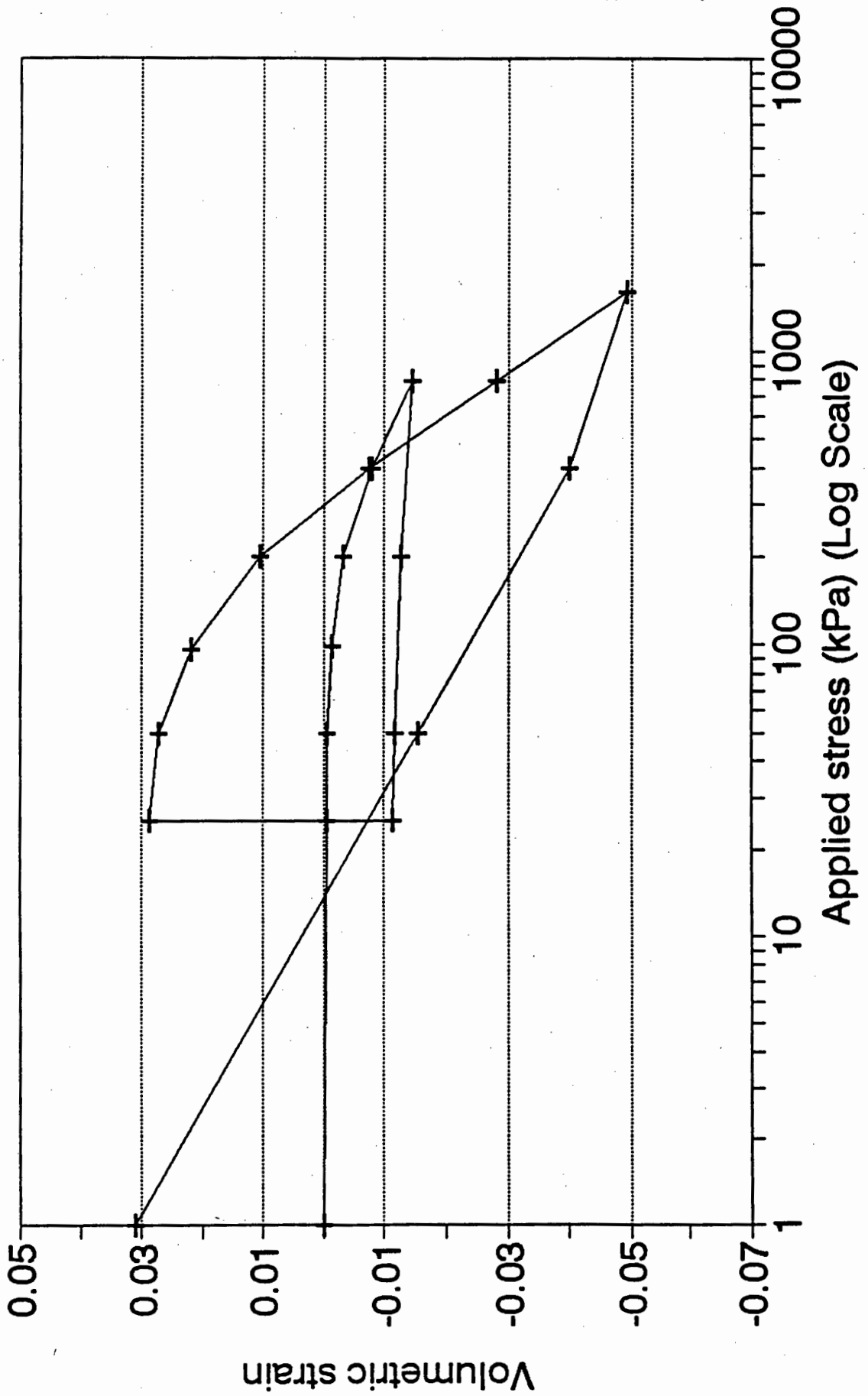
CONSOLIDATION TEST: SETTLEMENT READINGS

Rosebank Clay Samples

Test Series Code:	6	Loading Stone Diameter										
Sample No.:	R6-4	Avg:										
Machine Number	D	Area (mm ²)										
Init. sample ht	19.03	Loading Arm Ratio										
Load Inc.	1	2	3	4	5	6	7	8	9	10	11	12
Load (kg)	0.885	1.75	3.5	7	14	28	7	7	0.885	water	1.75	3.4
Load (kPa)	1	25.16086	49.75312	99.50624	199.0125	398.025	796.0499	199.0125	49.75312	25.16086	25.16086	96.66321
Time (min)	0	0	0.2	0.32	0.64	1.14	2.242	3.74	3.26	2.9	20.3	20.685
SQRT (time)	0.5	0.707107	0.24	0.25	0.28	0.29	0.3	0.3	0.3	0.3	0.3	0.3
0.5	1	1	1	1	1	1	1	1	1	1	1	1
1	2	1.414214	2	2	2	2	2	2	2	2	2	2
2	5	2.236068	5	5	5	5	5	5	5	5	5	5
5	10	3.162278	10	10	10	10	10	10	10	10	10	10
10	30	5.477226	30	30	30	30	30	30	30	30	30	30
30	60	7.745967	60	60	60	60	60	60	60	60	60	60
60	120	10.95445	120	120	120	120	120	120	120	120	120	120
120	240	15.49193	240	240	240	240	240	240	240	240	240	240
240	480	21.9089	480	480	480	480	480	480	480	480	480	480
480	1440	37.94733	1440	1440	1440	1440	1440	1440	1440	1440	1440	1440
1440	DIAL GUAGE READINGS											
Nett Compr. over inc. (mm)	0.2	0.32	0.64	1.14	2.242	3.74	3.26	2.9	2.825	-4.7	20.685	21.96
Cumulative compr.	0.22	0.12	0.32	0.5	1.102	1.498	-0.48	-0.36	-0.075	-7.525	0.385	1.275
Cumul. correction	0.022	0.012	0.032	0.05	0.1102	0.1498	-0.048	-0.036	-0.0075	-0.7525	0.0385	0.1275
Nett Cum. Compr.	0.022	0.034	0.066	0.116	0.2262	0.376	0.328	0.292	0.2845	-0.468	-0.4295	-0.302
Final sample height	0.013	0.02475	0.04015	0.0575	0.0786	0.102	0.085	0.068	0.068	0.0831	0.09	0.1159
Vol. Strain	0.009	0.00925	0.02585	0.0585	0.1476	0.274	0.243	0.224	0.2165	-0.5511	-0.5195	-0.4179
	19.021	19.02075	19.00415	18.9715	18.8824	18.756	18.787	18.806	18.8135	19.5811	19.5495	19.4479
	0	-0.00047	-0.00049	-0.00307	-0.00776	-0.0144	-0.01277	-0.01177	-0.01138	0.02896	0.027299	0.02196

Rosebank Series 6

Sample R6-4 (Machine D)



APPENDIX II
SOURCE CODE FOR USER DEFINED MATERIAL

```

C===== 1
C 2
C      file = claymat.f 3
C 4
C===== 5
c  Module:.....CLAYMAT: An ABAQUS user material for implementing a 6
c      constitutive model for a heaving clay from Rosebank, 7
c      Cape Province. Elastic, plastic and heave strains 8
c      are incorporated into the model. Plastic and heave 9
c      strains are assumed to be volumetric only and thus 10
c      do not influence the shear strains, which are 11
c      assumed to remain elastic. Heave is controlled by 12
c      changes in the temperature, which is used here as 13
c      a substitute variable for the soil suction. 14
c 15
c  Programmer:.....ken wiseman! 16
c 17
c  Date:.....September 1993 18
c 19
c  Reference:.....ABAQUS Version 5.2 User Manual 20
c 21
c  Implementation:...FORTRAN running under IBM AIX operating system 22
c 23
c----- 24
c 25
c      subroutine UMAT (stress, statev, ddsdde, sse, spd, scd, 26
1          rpl, ddsddt, drplde, drpldt, 27
2          stran, dstran, time, dtime, temp, dtemp, 28
3          predef, dpred, cmname, ndi, nshr, ntens, 29
4          nstatv, props, nprops, coords, drot, pnewdt, 30
5          celent) 31
c 32
c      implicit none 33
c 34
c-----ABAQUS Variables 35
character*8 cmname 36
integer      ndi, nshr, ntens, nstatv, nprops, nprec 37
parameter    (nprec = 2) 38
real*8      stress, statev, ddsdde, sse, spd, scd, rpl, ddsddt, 39
1          drplde, drpldt, stran, dstran, time, dtime, temp, 40
2          dtemp, predef, dpred, props, coords, drot, pnewdt, 41
3          celent 42
c 43
c-----Subroutine Variables 44
integer i, j ! loop counters 45
integer msg ! unit number for messages 46
real*8 p ! 1/3*(First stress invariant) 47
real*8 Evt ! First strain invariant at end of increment 48
real*8 Evt0 ! 1st strain invariant at start of increment 49
real*8 kappa ! Value of the logarithmic el. bulk modulus 50
real*8 devstr(3) ! deviatoric part of direct stress tensor 51
real*8 K ! Incremental bulk modulus 52
real*8 G ! Incremental shear modulus 53
real*8 D1, D2 ! Components of DDSDDDE 54
real*8 nztol ! tolerance on values which should be >0 55
real*8 zertol ! tolerance for numbers close to zero 56
c 57
c-----dimension variables 58
dimension stress(ntens), statev(nstatv), ddsdde(ntens,ntens), 59
1          ddsddt(ntens), drplde(ntens), stran(ntens), 60
2          dstran(ntens), time(2), predef(1), dpred(1), 61
3          props(nprops), coords(3), drot(3,3) 62
c 63
c----- 64
c      Description of properties in PROPS() 65
c 66
c      props(1) - lambda: Logarithmic elastic-plastic bulk modulus 67
c      (2) - kappa (dry): Log. el. bulk mod. at init. suction (T0) 68
c      (3) - kappa (sat): Log. el. bulk mod. at full saturation (T=1) 69
c      (4) - Poisson's ratio 70
c      (5) - Pc0: Initial preconsolidation pressure 71
c      (6) - initial suction (ie T0: initial temperature) 72

```

```

c      (7) - H0: percent heave at V0                                73
c      (8) - Ph0: heave pressure at V0: NB USE ABSOLUTE (+) VALUE 74
c      (9) - Mh: gradient of percent heave versus V              75
c      (10) - Mp: gradient of heave strain under Ph0, versus V   76
c      (11) - V0: reference sample specific volume                77
c      (12) - Pfact: Factor relating change in Pc on wetting to V0 78
c                                                                 79
c-----
c      Description of solution dependent variables in STATEV()     80
c                                                                 81
c      statev(1) - preconsolidation pressure pc (should be -ve: compression) 83
c      (2) - strain adjustment Evt0 to relate total to elastic strains 84
c      (+ve: expansion strain. -ve: compression strain).          85
c      (3) - V: vol. due to plastic strains for calc. of max heave. 86
c                                                                 87
c-----
c                                                                 88
c-----define constants                                          89
c      nztol = -0.1d0                                             90
c      zertol = 0.001d0                                           91
c      umsg = 6                                                    92
c                                                                 93
c-----calculate values of first stress and strain invariants    94
c      p = 0.0d0                                                  95
c      Evt = 0.0d0                                                96
c      Evt0 = 0.0d0                                               97
c      do i=1,ndi                                                 98
c          p = p + stress(i)                                       99
c          Evt = Evt + stran(i) + dstran(i)                       100
c          Evt0 = Evt0 + stran(i)                                  101
c      end do                                                     102
c      p = p/3.0d0                                               103
c                                                                 104
c-----define values in the deviatoric stress tensor             105
c      do i = 1,ndi                                               106
c          devstr(i) = stress(i) - p                               107
c      end do                                                     108
c                                                                 109
c-----calculate the current value of the logarithmic el. bulk modulus 110
c      kappa = props(2) + (props(3)-props(2))*(1.d0 -             111
c          log(temp+dtemp)/log(props(6)))                          112
c                                                                 113
c-----Initialise the preconsolidation pressure                  114
c      if (time(2).lt.1.d-20.and.statev(1).gt.-1.) then ! is total time = 0.? 115
c          statev(1) = props(5)                                    116
c          statev(2) = Evt0+(props(1)-kappa)*log(-props(5))+kappa*log(-p) 117
c          statev(3) = props(11)                                  118
c      end if                                                     119
c                                                                 120
c-----give WARNING messages if values are too low or out of    121
c the scope of the constitutive model.                            122
c      if (p.gt.nztol) then                                       123
c          write(umsg,1001) p                                       124
c          format(' ',***** - WARNING: Value of P is very small or ', 125
c              'tensile: ',d15.8)                                    126
c          end if                                                 127
c          if (dtemp.gt.zertol) then                               128
c              write(umsg,1011) dtemp                                129
c              format(' ',***** - WARNING: Value of suction (temperature)', 130
c                  ' is increasing: ',d15.8/                        131
c                  ' ', 'Constitutive model only valid for decreasing', 132
c                  ' suctions (temperatures)'                       133
c          end if                                                 134
c          if ((temp+dtemp).lt.-nztol) then                       135
c              write(umsg,1021) (temp+dtemp)                       136
c              format(' ',***** - WARNING: Value of suction (temperature)', 137
c                  ' is very low or negative: ',d15.8)            138
c          end if                                                 139
c                                                                 140
c-----Check whether "suction" (temperature) is changing:      141
c      If YES: call heave model subroutine                        142
c      if NO: call mechanical consolidation subroutine            143
c                                                                 144

```



```

c i  props  - array of material properties                217
c i  nprops - number of properties in array              218
c o  K      - incremental elastic bulk modulus           219
c                                           220
c----- 221
c                                           222
c      implicit none                                     223
c      integer  nsdv, nprops, umsg                       224
c      real*8   p, Evt, Evt0, T0, dT, sdv(nsdv), props(nprops), K  225
c      real*8   T      ! Temperature at end of increment          226
c      real*8   H      ! % heave for current suction/V0          227
c      real*8   Ph     ! Heave pr for current suction/V0          228
c      real*8   kappa ! logarithmic elastic bulk modulus          229
c      real*8   ptemp ! temporary value of the pressure           230
c      real*8   p0    ! initial value of p                       231
c      real*8   Evh0  ! previous value of heave strain           232
c                                           233
c      umsg = 6                                           234
c      T = T0 + dT                                         235
c      p0 = p                                              236
c                                           237
c----- Calculate the values of Ph and H based on V0 and T0      238
c      H = props(7) - props(9)*(sdv(3)-props(11))          239
c      Ph = props(8)**(H/(props(7)-(props(9)-props(10))*  240
c           (sdv(3)-props(11))))                          241
c                                           242
c----- calculate Evh0                                          243
c      Evh0 = H*(1.d0-log(T0)/log(props(6)))*(1.d0-log(-p0)/log(Ph))  244
c                                           245
c----- calculate return value for p:                            246
c      p = -(Ph**(1.d0-(Evh0+Evt-Evt0)/                    247
c           (H*(1.d0-log(T)/log(props(6))))))              248
c                                           249
c----- calculate return value for K                             250
c      if (p.gt.-0.333333d0) then                          251
c         ptemp = -1.0d0                                   252
c      else                                                253
c         ptemp = p                                       254
c      end if                                             255
c      K = -ptemp*log(Ph)/(H*(1.d0-log(T)/log(props(6))))  256
c                                           257
c----- update Pc                                              258
c      sdv(1) = sdv(1)+((3.d0+props(12))*(sdv(3)-props(11)))**p - sdv(1))**  259
c           (1.d0 - log(T)/log(props(6)))                 260
c      if (sdv(1).gt.p) then                               261
c         sdv(1) = p.                                     262
c      end if                                             263
c                                           264
c----- update Eva                                             265
c      kappa = props(2) + (props(3)-props(2))*(1.d0 -    266
c           log(T)/log(props(6)))                          267
c      sdv(2) = Evt + (props(1)-kappa)*log(-sdv(1)) + kappa*log(-p)  268
c                                           269
c      return                                             270
c      end                                               271
c                                           272
c----- 273
c                                           274
c                                           275
c      subroutine MECC01(p, Evt, sdv, nsdv, lambda, kappa, V0, K)  276
c                                           277
c----- 278
c      MECHANical CONSolidation for umat 1                279
c                                           280
c      Purpose: To calculate the hydrostatic stress p from the constitutive  281
c               equations, update SDV's and calculate the bulk modulus K     282
c               for condition where there is no "suction" change and         283
c               ordinary consolidation will occur.                          284
c                                           285
c      Parameters:                                          286
c o  p      - hydrostatic stress                               287
c i  Evt    - volumetric strain (first strain invariant) at end of incr.     288

```

```

c io sdv - solution dependant variables: IN as values at start of 289
c increment, OUT updated to values at end of increment 290
c i nsdv - number of solution dependant variables 291
c i lambda - elastic-plastic logarithmic bulk modulus 292
c i kappa - current value of elastic logarithmic bulk modulus 293
c i V0 - reference specific volume (Spec. Vol at start of analysis) 294
c o K - incremental elastic bulk modulus 295
c 296
c..... 297
c 298
c implicit none 299
c integer nsdv 300
c real*8 p, Evt, sdv, lambda, kappa, V0, K 301
c 302
c integer plast ! Flags that plastic deformation has occurred 303
c real*8 PPRED1, dPC1 ! Functions 304
c real*8 zertol ! Tolerance to check value is close to 0 305
c real*8 ptemp ! Temporary stress value 306
c real*8 Pc ! Yield stress 307
c 308
c dimension sdv(nsdv) 309
c 310
c-----Set tolerance factors to check convergence of functions 311
c zertol = 0.001d0 312
c 313
c-----Set initial values 314
c Pc = sdv(1) 315
c plast = 0 ! Flag that no plastic deformation has occurred 316
c 317
c-----calculate initial prediction of Hydrostatic pressure p 318
c p = PPRED1(Evt, lambda, kappa, sdv(1), sdv(2)) 319
c 320
c-----Perform iterative loop while yield function > 0 to update Pc (sdv(1)) 321
c do while ((-p+Pc).gt.zertol) !The condition is the yield function 322
c Pc = Pc - dPC1(Pc, Evt, sdv(2), lambda, kappa) 323
c p = PPRED1(Evt, lambda, kappa, Pc, sdv(2)) 324
c plast = 1 ! Flag that plastic deformation has occurred. 325
c end do 326
c 327
c-----calculate value incremental elastic bulk modulus K 328
c if (p.gt.-0.333333d0) then 329
c ptemp = -1.d0 330
c else 331
c ptemp=p 332
c end if 333
c if (plast.eq.1) then 334
c Update the solution dependant variables & calculate K for plast. def 335
c K = -ptemp/lambda 336
c sdv(3) = sdv(3) + (lambda-kappa)*log(sdv(1)/Pc)*V0 337
c sdv(1) = Pc 338
c else 339
c K = -ptemp/kappa 340
c end if 341
c 342
c return 343
c end 344
c 345
c===== 346
c 347
c real*8 function PPRED1(Evt, lambda, kappa, Pc, Eva) 348
c 349
c..... 350
c P PREDictor for umat 1 351
c 352
c Purpose: To calculate the hydrostatic stress p from the constitutive 353
c equations for the case of ordinary consolidation. 354
c 355
c Parameters: 356
c i Evt - volumetric strain (first strain invariant) at end of incr. 357
c i lambda - elastic-plastic logarithmic bulk modulus 358
c i kappa - current value of elastic logarithmic bulk modulus 359
c i Pc - current value of the preconsolidation pressure 360

```

```

c i  Eva    - Plastic adjustment strain          361
c                                             362
c.....                                       363
c                                             364
c      implicit none                          365
c      real*8   Pc, Evt, Eva, lambda, kappa    366
c                                             367
c      PPRED1 = -exp(-(Evt-Eva+(lambda-kappa)*log(-Pc))/kappa) 368
c                                             369
c      return                                  370
c      end                                      371
c                                             372
c=====                                       373
c                                             374
c      real*8 function dPC1(Pc, Evt, Eva, lambda, kappa) 375
c                                             376
c                                             377
c.....                                       378
c      Change in PC for umat 1                 379
c                                             380
c      Purpose: To calculate the increment dPC in the preconsolidation 381
c              pressure Pc for the case where yielding is occurring. 382
c                                             383
c      Parameters:                             384
c i  Pc      - current value of the preconsolidation pressure Pc      385
c i  Evt     - volumetric strain (first strain invariant) at end of incr. 386
c i  Eva     - Plastic adjustment strain                               387
c i  lambda  - elastic-plastic logarithmic bulk modulus               388
c i  kappa   - current value of elastic logarithmic bulk modulus      389
c                                             390
c.....                                       391
c                                             392
c      implicit none                          393
c      real*8   Pc, Evt, Eva, lambda, kappa    394
c      real*8   Con    ! Constant forming part of dPC1 expression. 395
c                                             396
c      Con = exp((Eva-Evt)/kappa)*(-Pc)**(-lambda/kappa) 397
c      dPC1 = Pc*(Con - 1.d0)/((kappa-lambda)/kappa*Con - 1.d0) 398
c                                             399
c      return                                  400
c      end                                      401
c                                             402
c=====                                       403
c=====          the end!!          ===== 404
c=====                                       405

```

APPENDIX III
INPUT DECKS FOR ABAQUS ANALYSES

```

***** 1
** 2
** Heaving of a single element 3
** 4
** by ken! 29 Sept 1993 5
** ROSE1.INP 6
** 7
***** 8
** 9
*HEADING 10
Oedometer test Series R1 on heaving clay. Heave at 100 kPa. UMAT=claymat.f 11
*PREPRINT,MODEL=NO,HISTORY=NO,ECHO=NO 12
** 13
**-----NODES 14
*NODE,NSET=ALLN 15
1,0.,0. 16
2,0.5,0. 17
3,1.,0. 18
4,0.,0.5 19
5,1.,0.5 20
6,0.,1. 21
7,0.5,1. 22
8,1.,1. 23
*NSET,NSET=POREN 24
1,3,6,8 25
*NSET,NSET=BOT 26
1,2,3 27
*NSET,NSET=PORT 28
6,8 29
*NSET,NSET=TOP 30
6,7,8 31
** 32
**-----ELEMENTS 33
*ELEMENT,ELSET=EALL,TYPE=CPE8RT 34
1,1,3,8,6,2,5,7,4 35
** 36
**-----MATERIAL 37
*SOLID SECTION,ELSET=EALL,MATERIAL=CLAY 38
*MATERIAL,NAME=CLAY 39
*USER MATERIAL,CONSTANTS=12 40
0.02736,0.001917,0.01061,0.3,-200.,1000.,0.06,425. 41
1.E-5,1.E-5,1.8,1.E-5 42
*DEPVAR 43
3 44
*CONDUCTIVITY 45
0.002 46
** 47
**-----BOUNDARY 48
*BOUNDARY 49
BOT,1,2 50
ALLN,1 51
** 52
**-----INITIAL CONDITIONS 53
*INITIAL CONDITIONS,TYPE=STRESS 54
EALL,-1.,-1.,-1. 55
*INITIAL CONDITIONS,TYPE=TEMPERATURE 56
POREN,1000. 57
*AMPLITUDE,NAME=TEMP1,TIME=STEP,V=ABS 58
0.,1000.,100.,980.,200.,950.,300.,860. 59
400.,630.,500.,1. 60
*AMPLITUDE,NAME=LOAD5,TIME=STEP,V=ABS 61
0.,1700,90.,1.0 62
***** 63
**-----STEP1-----Do initial loading to 1 kPa 64
*STEP,MONOTONIC=NO 65
Apply 1 kPa Load and establish static equilibrium. 66
*STATIC 67
1.,1. 68
*BOUNDARY 69
POREN,11,,1000. 70
*DLOAD 71
1,P3,1. 72

```

*EL PRINT,FREQ=100,POS=CENTROIDAL	73
S,PRESS,E	74
SDV	75
*NODE PRINT,FREQ=100	76
U,RF,NT11	77
*END STEP	78
**	79
**-----STEP2-----Apply a load of 100 kPa	80
*STEP, INC=50, AMP=RAMP, MONOTONIC=NO	81
Apply 100 kPa load to the soil.	82
*STATIC	83
1.,10.,0.1,1.	84
*BOUNDARY	85
POREN,11,,1000.	86
*DLOAD	87
1,P3,100.	88
*EL PRINT,FREQ=1,POS=CENTROIDAL	89
S,PRESS,E	90
SDV	91
*NODE PRINT,FREQ=1	92
U,RF,NT11	93
*END STEP	94
**	95
**-----STEP3-----Release the rest of the PPRESS	96
*STEP,INC=250,MONOTONIC=NO	97
Reduce the soil suction (temperature) and cause heave	98
*STATIC	99
100.,1000.,0.1,100.	100
*CONTROLS, PARAMETERS=TIME INCREMENTATION	101
8,12,15,30,20	102
*BOUNDARY,AMP=TEMP1	103
POREN,11	104
*EL PRINT,FREQ=1,POS=CENTROIDAL	105
S,PRESS,E	106
SDV	107
*NODE PRINT,FREQ=1	108
U,RF,NT11	109
*END STEP	110
**	111
**-----STEP 4-----Release the pore pressure	112
*STEP,INC=200,AMP=RAMP,MONOTONIC=NO	113
Apply and consolidate 1700 kPa load.	114
*STATIC	115
4.,100.,0.1,5.	116
*DLOAD	117
1,P3,1700.	118
*BOUNDARY	119
POREN,11,,1.	120
*EL PRINT,FREQ=1,POS=CENTROIDAL	121
S,PRESS,E	122
SDV	123
*NODE PRINT,FREQ=1	124
U,RF,NT11	125
*END STEP	126
**	127
**-----STEP 5-----Release the pore pressure	128
*STEP,INC=200,AMP=STEP,MONOTONIC=NO	129
Unload to 1 kPa	130
*STATIC	131
4.,100.,0.1,5.	132
*DLOAD,AMP=LOAD5	133
1,P3	134
*BOUNDARY	135
POREN,11,,1.	136
*EL PRINT,FREQ=1,POS=CENTROIDAL	137
S,PRESS,E	138
SDV	139
*NODE PRINT,FREQ=1	140
U,RF,NT11	141
*END STEP	142
**-----the end!!	143

```

*****
**
**          Swelling of a single element
**
**    by ken!          ROSE2          25 September 1993
**
*****
**
*HEADING
Heave modelling. Preconsolidation then swell at 100 kPa. UMAT=claymat.f
*PREPRINT,MODEL=NO,HISTORY=NO,ECHO=NO
**
-----NODES
*NODE,NSET=ALLN
1,0.,0.
2,0.5,0.
3,1.,0.
4,0.,0.5
5,1.,0.5
6,0.,1.
7,0.5,1.
8,1.,1.
*NSET,NSET=POREN
1,3,6,8
*NSET,NSET=BOT
1,2,3
*NSET,NSET=TOP
6,7,8
**
-----ELEMENTS
*ELEMENT,ELSET=EALL,TYPE=CPE8RT
1,1,3,8,6,2,5,7,4
**
-----MATERIAL
*SOLID SECTION,ELSET=EALL,MATERIAL=CLAY
*MATERIAL,NAME=CLAY
*USER MATERIAL,CONSTANTS=12
0.02736,0.001917,0.01061,0.3,-452.,1000.,0.057,425.
0.312313,0.113775994,1.9,1.E-5
*DEPVAR
3
*CONDUCTIVITY
0.002
**
-----BOUNDARY
*BOUNDARY
BOT,1,2
ALLN,1
**
-----INITIAL CONDITIONS
*INITIAL CONDITIONS,TYPE=STRESS
EALL,-1.,-1.,-1.
*INITIAL CONDITIONS,TYPE=TEMPERATURE
POREN,1000.
*AMPLITUDE,NAME=TEMP1,TIME=STEP,V=ABS
0.,1000.,10.,1.
*AMPLITUDE,NAME=LOAD,TIME=STEP,V=REL
0.,0.,20.,0.02,40.,0.05,60.,0.145
69.,0.24,75.,0.26,80.,0.28,85,0.3
90,0.35,170.,1.
*AMPLITUDE,NAME=LOAD6,TIME=STEP,V=ABS
0.,1700.,90.,1.
*AMPLITUDE,NAME=LOAD3,TIME=STEP,V=ABS
0.,1700.,90.,100.
**
*****
**
**-----STEP 1-----Do initial loading to 1 kPa
*STEP,INC=10
Apply 1 kPa Load and establish static equilibrium.
*STATIC

```

1.,1.	73
*BOUNDARY	74
POREN,11,,1000.	75
*DLOAD	76
1,P3,1.	77
*EL PRINT,FREQ=100,POS=CENTROIDAL	78
S,PRESS,E	79
SDV	80
*NODE PRINT,FREQ=100	81
U,RF,NT11	82
*END STEP	83
**	84
**-----STEP 2-----Do loading to 1700 kPa	85
*STEP,INC=300,AMP=RAMP,MONOTONIC=NO	86
Apply 1700 kPa Load and establish static equilibrium.	87
*STATIC	88
5.,200.,.01,5.	89
*CONTROLS,PARAMETERS=TIME INTEGRATION	90
8,12,15,30,20	91
*BOUNDARY	92
POREN,11,,1000.	93
*DLOAD,AMP=LOAD	94
1,P3,1700.	95
*EL PRINT,FREQ=1,POS=CENTROIDAL	96
S,PRESS,E	97
SDV	98
*NODE PRINT,FREQ=0	99
U,RF,NT11	100
*END STEP	101
**	102
**-----STEP 3-----Unload to 100 kPa	103
*STEP,INC=100,AMP=STEP,MONOTONIC=NO	104
Unload to 100 kPa.	105
*STATIC	106
4.,100.,0.1,10.	107
*CONTROLS,PARAMETERS=TIME INTEGRATION	108
8,12,15,30,20	109
*BOUNDARY	110
POREN,11,,1000.	111
*DLOAD,AMP=LOAD3	112
1,P3	113
*EL PRINT,FREQ=1,POS=CENTROIDAL	114
S,PRESS,E	115
SDV	116
*NODE PRINT,FREQ=0	117
U,RF,NT11	118
*END STEP	119
**	120
**-----STEP 4-----Reduce suction to zero	121
*STEP,INC=100,MONOTONIC=NO	122
Reduce soil suction (temperature) to 1 and trigger heave.	123
*STATIC	124
100.,1000.,0.1,100.	125
*CONTROLS,PARAMETERS=TIME INTEGRATION	126
8,12,15,30,20	127
*BOUNDARY,AMP=TEMP1	128
POREN,11	129
*EL PRINT,FREQ=1,POS=CENTROIDAL	130
S,PRESS,E	131
SDV	132
*NODE PRINT,FREQ=0	133
U,RF,NT11	134
*END STEP	135
**	136
**-----STEP 5-----Apply 1700 kPa Load	137
*STEP,INC=300,AMP=RAMP,MONOTONIC=NO	138
Apply and consolidate 1700 kPa load.	139
*STATIC	140
4.,100.,0.1,10.	141
*CONTROLS,PARAMETERS=TIME INTEGRATION	142
8,12,15,30,20	143
*DLOAD	144

1,P3,1700.	145
*BOUNDARY	146
POREN,11,,1.	147
*EL PRINT,FREQ=1,POS=CENTROIDAL	148
S,PRESS,E	149
SDV	150
*NODE PRINT,FREQ=0	151
U,RF,NT11	152
*END STEP	153
**	154
**-----STEP 6-----Unload to 1 kPa	155
*STEP,INC=300,AMP=STEP,MONOTONIC=NO	156
Unload to 1 kPa	157
*STATIC	158
4.,100.,0.1,10.	159
*CONTROLS,PARAMETERS=TIME INTEGRATION	160
8,12,15,30,20	161
*DLOAD,AMP=LOAD6	162
1,P3	163
*BOUNDARY	164
POREN,11,,1.	165
*EL PRINT,FREQ=1,POS=CENTROIDAL	166
S,PRESS,E	167
SDV	168
*NODE PRINT,FREQ=0	169
U,RF,NT11	170
*END STEP	171
**	172
**-----end!!	173

```

*****
**
**                               Doming under Slab
**
** First stage of problem: Do thermal analysis to solve suction
** temperature contours under slab and write to file "slabtem.fil"
**
** by ken!                               29 September 1993
**
*****
**
*HEADING,UNSYMM
Doming under slab: Pre-solution of suction (temperature) values. (SLABTEM)
***DATA CHECK
*PREPRINT, MODEL=NO,ECHO=NO,HISTORY=NO
*RESTART,WRITE,FREQ=100
**
**-----NODES
*NODE,NSET=BOT
1,0.,-6.
241,6.,-6.
641,11.,-6.
*NGEN,NSET=BOT
1,241,20
241,641,20
*NODE,NSET=TOP
19,0.,0.
259,6.,0.
659,11.,0.
*NGEN,NSET=TOP
19,259,20
259,659,20
*NODE,NSET=MID
7,0.,-3.
247,6.,-3.
647,11.,-3.
*NGEN,NSET=MID
7,247,20
247,647,20
*NFILL,NSET=SOIL
BOT,MID,6,1
MID, TOP, 12, 1
*NSET,NSET=SLAB,GEN
459,659,20
*NSET,NSET=OPEN,GEN
19,419,20
*NSET,NSET=GWT,GEN
1,641,20
**
**-----ELEMENTS
*ELEMENT,ELSET=SOIL,TYPE=DC2D8
1,1,41,43,3,21,42,23,2
*ELGEN,ELSET=SOIL
1,9,2,1,16,40,9
*ELSET,ELSET=TOP,GEN
9,144,9
*ELSET,ELSET=L9,GEN
9,144,9
*ELSET,ELSET=L8,GEN
8,143,9
*ELSET,ELSET=L7,GEN
7,142,9
*ELSET,ELSET=L6,GEN
6,141,9
*ELSET,ELSET=L5,GEN
5,140,9
*ELSET,ELSET=L4,GEN
4,139,9
*ELSET,ELSET=L3,GEN
3,138,9
*ELSET,ELSET=L2,GEN
2,137,9

```

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*ELSET,ELSET=L1,GEN
1,136,9
**
**-----MATERIALS
*SOLID SECTION,ELSET=SOIL,MATERIAL=CLAY
*MATERIAL,NAME=CLAY
*CONDUCTIVITY
0.002
**
**-----INITIAL CONDITIONS
*INITIAL CONDITIONS, TYPE=TEMPERATURE
SOIL,1000.
**
**-----MPCS
*BOUNDARY
SOIL,1
BOT,1,2
*AMPLITUDE,NAME=BC1,TIME=STEP, V=ABS
0.,1000.,100.,1.
**
*****begin history*****
**
*****STEP 1*****
**
*STEP, INC=200, MONOTONIC=NO
Find equilibrium contours after slab is placed.
*HEAT TRANSFER, DELTMX=50., END=SS
4.,400.,0.1,32,0.01
*CONTROLS, PARAMETERS=TIME INCREMENTATION
8,12,15,30,20
*BOUNDARY
OPEN,11,,1000.
*BOUNDARY,AMP=BC1
SLAB,11,,1000.
GWT,11
*NODE FILE, NSET=SOIL
NT
*END STEP
**
**----->end!!!

```

```

*****
**
**          DOMING UNDER SLAB
** Second stage of problem: solving for stresses and displacements
** using input from first step of analysis
**
** by ken!                29 September 1993
**
*****
**
*HEADING, UNSYMM
Final Swelling with earth pressure & Sat contS. TJVDSV Rosebank SLABF6
**DATA CHECK
*PREPRINT, MODEL=NO, ECHO=NO, HISTORY=NO
*RESTART, WRITE, FREQ=100
**
**-----NODES
*NODE, NSET=BOT
1, 0., -6.
241, 6., -6.
641, 11., -6.
*NGEN, NSET=BOT
1, 241, 20
241, 641, 20
*NODE, NSET=TOP
19, 0., 0.
259, 6., 0.
659, 11., 0.
*NGEN, NSET=TOP
19, 259, 20
259, 659, 20
*NODE, NSET=MID
7, 0., -3.
247, 6., -3.
647, 11., -3.
*NGEN, NSET=MID
7, 247, 20
247, 647, 20
*NFILL, NSET=SOIL
BOT, MID, 6, 1
MID, TOP, 12, 1
*NSSET, NSET=SLAB, GEN
459, 659, 40
*NSSET, NSET=OPEN, GEN
19, 419, 40
*NSSET, NSET=GWT, GEN
1, 641, 40
**
**-----ELEMENTS
*ELEMENT, ELSET=SOIL, TYPE=CPE8R
1, 1, 41, 43, 3, 21, 42, 23, 2
*ELGEN, ELSET=SOIL
1, 9, 2, 1, 16, 40, 9
*ELSET, ELSET=TOP, GEN
9, 144, 9
*ELSET, ELSET=L9, GEN
9, 144, 9
*ELSET, ELSET=L8, GEN
8, 143, 9
*ELSET, ELSET=L7, GEN
7, 142, 9
*ELSET, ELSET=L6, GEN
6, 141, 9
*ELSET, ELSET=L5, GEN
5, 140, 9
*ELSET, ELSET=L4, GEN
4, 139, 9
*ELSET, ELSET=L3, GEN
3, 138, 9
*ELSET, ELSET=L2, GEN
2, 137, 9

```

*ELSET, ELSET=L1, GEN	73
1, 136, 9	74
**	75
**-----MATERIALS	76
*SOLID SECTION, ELSET=SOIL, MATERIAL=CLAY	77
*MATERIAL, NAME=CLAY	78
*USER MATERIAL, CONSTANTS=12	79
0.02736, 0.001917, 0.01061, 0.3, -452., 1000., 0.24, 425.	80
0.312313, 0.113775994, 1.9, 1.E-5	81
*DEPVAR	82
3	83
**	84
**-----INITIAL CONDITIONS	85
*INITIAL CONDITIONS, TYPE=STRESS	86
SOIL, -1., -1., -1.	87
*INITIAL CONDITIONS, TYPE=TEMPERATURE	88
SOIL, 1000.	89
**	90
**-----MPCS	91
*BOUNDARY	92
SOIL, 1	93
BOT, 1, 2	94
**	95
*****begin history*****	96
**	97
*****STEP 1*****	98
*STEP, MONOTONIC=NO	99
Apply 1kPa load and check equilibrium.	100
*STATIC	101
1., 1.	102
*DLOAD	103
TOP, P3, 1.	104
*EL PRINT, FREQ=100, POS=CENT	105
S, PRESS, E	106
SDV	107
*NODE PRINT, FREQ=100	108
NT11	109
*END STEP	110
**	111
*****STEP 2*****	112
**	113
*STEP, INC=300, MONOTONIC=NO, AMP=RAMP	114
Find eqm pore pressure contours & add soil loads before slab is placed.	115
*STATIC	116
4., 100., 0.1, 4.	117
*CONTROLS, PARAMETERS=TIME INCREMENTATION	118
12, 18, 23, 45, 30	119
*DLOAD	120
L8, P3, 10.	121
L7, P3, 10.	122
L6, P3, 10.	123
L5, P3, 10.	124
L4, P3, 10.	125
L3, P3, 20.	126
L2, P3, 20.	127
L1, P3, 20.	128
*EL PRINT, FREQ=100, POS=CENT	129
S, PRESS, E	130
SDV	131
*NODE PRINT, FREQ=100	132
NT11	133
*END STEP	134
**	135
*****STEP 3*****	136
**	137
*STEP, INC=200, MONOTONIC=NO	138
Find equilibrium contours after slab is placed.	139
*STATIC	140
4., 104., 0.1, 4.	141
*CONTROLS, PARAMETERS=TIME INCREMENTATION	142
8, 60, 10, 90, 90	143
*TEMPERATURE, /FILE=slabtem, BSTEP=1, BINC=1, EINC=26	144

*NODE PRINT,NSET=TOP,FREQ=100	145
U	146
*END STEP	147
**----->end!!!	148

APPENDIX IV

COURSEWORK COMPLETED IN FULFILMENT OF REQUIREMENTS FOR DEGREE OF MASTER OF SCIENCE IN CIVIL ENGINEERING

The following courses were taken to complete the requirements for the degree of Master of Science in Civil Engineering:

Course Code	Course Name	Credits
CAM 500Z	Applied Mechanics A	3
CAM 501Z	Applied Mechanics B	3
CAM 502Z	An Introduction to Finite Elements	3
CAM 503Z	Finite Element Analysis	4
CAM 504Z	Engineering Software Design and Development	3
CIV 509S	Structural Loading	3
CIV 530Z	Civil Engineering Project	5
CIV 562F	Design of Concrete Structures for Impact Loading	3
CIV 513F	Waste Water Treatment Part 1	5
CIV 587Z	Low Cost Sanitation	5
SEM 523F	Introduction to Business	5
MEC 532Z	Quantitative Methods in Management	5
TOTAL		47

Total credits from coursework:	47
Credits for half thesis:	20
TOTAL credits:	67

Credits required for MSc(Eng) degree: **60**