CAPACITY OF VERTICALLY LOADED PILES IN LOW DENSITY SANDS

By

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ABSTRACT

The reduction of pile capacity associated with volume contraction of the soil close to the pile surface has been reported in carbonate deposits offshore North West Australia and in residual deposits of Southern Africa.

Knowledge of the load carried by the shaft and the pile tip is critical for the determination of the load settlement behavior of piles in structurally unstable and highly variable sand deposits. While the Static and Dynamic formulas and Pile load tests are used for the determination of pile carrying capacity, they are limited in terms of site coverage, cost and adequacy of load – settlement data. Since the mode of shearing around a pile shaft is very similar to that observed in the direct shear tests, it is thus cost effective to develop analytical methods based on controlled laboratory model tests in order to predict load settlement behavior and bearing capacity of piles.

A simple shear apparatus was developed to investigate whether or not significant contractile strains are induced in low density residual sands subject to simple shear strain and to study the effect of such contractile strain of a soil close to the pile shaft on pile load settlement behavior.

The design and development of the simple shear apparatus was based on a new simple shear stress equation. Series of constant normal stresses, constant normal stiffness and constant volume tests were conducted on samples of Berea Sands compacted to low density in the new apparatus, supported by moisture induced collapse settlement and matric suction tests. The tests revealed significant volume contraction of Berea Sands due to imposed simple shear strain.

The tests data were fed into a new Winkler - type load transfer model and were used to determine the load – transfer curves of vertically loaded piles. The curves revealed that both the load-settlement behavior and pile capacity in low density sands are dependent on the volume contraction of the soil in the plastic zone close to the pile surface, horizontal stress normal to the pile shaft and stiffness of the soil outside the plastic zone.

PREFACE

I, Felix Ndubisi Okonta, hereby declare that unless indicated, this thesis is my own work, and that it has not been submitted in whole or in part, for a degree at another university or institution.

Sign..................................Date...................

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CONTENTS

CHAPTER ONE

INTRODUCTION

1.1 GENERAL INTRODUCTION

Soils compacted at a broad range of moisture contents and dry densities and naturally occurring weakly bonded soils can exhibit particle rearrangement and varying degree of volume change as a result of changes in any one or combination of the following; degree of saturation, mean stress, shear stress or pore pressure. This mechanical phenomenon is broadly referred to as collapse (Barden 1972; Maswoswe 1985).

Excessive settlement of structures in many parts of the world are caused by collapse of soil structure. Collapse of soil structure are triggered by different factors. Jennings and Knight (1975), Barden (1972), Maswoswe (1985), and Vaughan et al. (1988) identified four major conditions that are necessary for collapse to occur in a soil. These conditions can be summarized as;

(a) An open, unstable partially saturated soil. Soil grains may be held together by bonding.

(b) A sufficiently large soil suction that stabilizes the soil in the partly saturated condition.

(c) A high enough total stress so that the soil is metastable.

(d) The addition of water to the soil which reduces the soil suction thereby causing shear failures at the interaggregate contacts.

Bulk volume compression of soils associated with conditions (a) (b) (c) and (d) has been referred to as moisture induced soil collapse, hydroconsolidation and hydrocompaction (Barden and Sides 1970; Jennings and Knight 1975 and Mitchel 1976). It is a common occurrence and can be easily interpreted and is well documented in soil mechanics literature.

Bulk volume compression of unsaturated soils due to applied shear stress can be observed in some soils due to conditions (a), (b) and (c) only. This is refereed to in this thesis as shear induced volume compression. While shear induced volume compression is commonly associated with normally consolidated soils, a similar phenomenon has also been recorded in low density weakly bonded saturated and unsaturated soils and some compacted tropical sands encountered in road construction (Maccarini 1980, Townsend 1985 and Lawton et al, 1989). Thus given certain initial and boundary conditions, shear stress induced bulk volume compression of saturated and unsaturated soils can occur.

Shear induced volume compression in normally consolidated soils is gradual from the on set of shear stress application, while compacted and weakly bonded soils show high initial stiffness due to suction, bonding or their combination followed by a "collapse" type volume compression. (Maccarini 1987 and Coop 1990).

The effect of shear compression on excessive settlement of structures has been documented in literature. Typical cases include the reduction in pile capacity associated with volume contraction of the soil alongside the pile interface during load testing of test piles from platforms in the Bass Strait carbonate deposits (Augemeer 1973), excessive settlement of piles of the North Rankin Platform, off shore Northwest Australia, (King and Lodge 1988), and most recently, and the reason for the present research, excessive settlement of test piles during the foundation design of a large aluminium smelter complex installed in residual deposit of Southern Mozambique (McKnight 1999).

Correct evaluation of the settlement and bearing capacity of compressible long slender piles installed in deep structurally unstable and highly variable sand deposits, where significant part of the applied load is transferred to the soil by the pile shaft, require the determination of the load carried by the shaft and the tip. Design of such foundation structures require the evaluation of the mechanism of load transfer to the soil surrounding an axially loaded pile, which is necessary for the determination of the load - settlement behaviour and capacity of piles in such deposits.

The proportion of pile load transferred by the shaft and the tip of a vertically loaded pile is dependent on the initial state of soils close to the pile surface. Results of series of pile material – soil interface tests by Yoshimi and Kishida (1991), analysis of data from instrumented pile load test (Luker 1988; Lehane et al. 1993) and numerical analysis of pile settlement behaviour (Boulon and Foray 1986; Naggar and Novak 1992) revealed that failure of piles in granular soils can occur at the pile soil interface or at the narrow band of soil close to the pile surface depending on the magnitude of the interface friction angle and the soil internal friction angle. The thickness of the shear band is dependent on the roughness of the pile surface and the relative density of the surrounding soil. For surfaces typical of concrete and steel materials in low density soils, local stress measurements (Yoshimi and Kishida 1991 and Lehane et al, 1993) revealed that failure occur in the narrow band of soil close to the pile and this narrow band is generally referred to as the pile interface or the plastic zone.

The mechanical behaviour of this narrow band of soil is affected by both the surface characteristics of the pile material and the soil properties. Shear and volumetric strains developed at peak and ultimate conditions are concentrated in this narrow band of soil, which is also referred to as the shear zone (Evgin and Fakharian 1996). Thus the soil surrounding a typical pile is divided into two regions; the inner, non linear stress field termed the shear or plastic zone and the outer elastic stress field.

Normally consolidated soils generally exhibit reduction in volume due to applied shear stress, and where this occurs alongside a pile in the shear zone, there is a complementary expansion in the soil outside the shear zone resulting in the movement of the outer soil toward the pile. For normally consolidated deposits, both the reduction in volume of the soil in the shear zone and the stiffness of the outer soil are small in magnitude, thus the associated reduction in horizontal stress normal to the pile shaft may not be very significant.

For weakly bonded residual soils and soils compacted to low density where mechanical are influenced by bonding and suction, the reduction in volume of the soil in the shear zone may be significant since these structured soils usually possess void ratio and in situ stiffness that are significantly higher than those of normally consolidated soils. Thus for the structured soils, the resultant expansion of the outer soil may be associated with significant reduction in horizontal stress because of the high in situ stiffness.

The main aim of this study in to investigate whether soils compacted to low density will exhibit significant compression of bulk volume due to increase in applied shear stress and to assess the effect of such compression on the load settlement behaviour and capacity of a vertically loaded pile installed in such deposit.

Proposals for design methods for the shaft capacity of piles in terms of effective stresses acting on the soil close to the pile emphasize the need for a closer look at the stress changes due to pile installation and subsequent loading. Pile installation in granular soils can rearrange the fabric, change the soil density and crush the grain, the extent however depending on whether the pile is driven or bored pile. For driven piles in sands fabric changes along the pile shaft results in a reduction of less than 5 degrees in friction angle measured with particle alignment to the major principal axis and the toe resistance is affected by a reduction of less than 2 degrees in friction angle. The vibration of driving results in permeable soils next the pile that is as dense as or denser than the undisturbed insitu soils. For bored piles installed in sands however, set up effects i.e. changes in density and fabric are negligible and any pore pressure changes that might develop during installation would be quickly dissipated in the permeable sand deposits.

It was observed that while soil strength parameters are estimated from triaxial compression test the mode of shearing around a pile is very similar to that observed in simple shear test (Scott 1981; Parry and Swain 1977; Luker 1988). The strain path undergone by the soil close to the pile is similar to that undergone by the same soil in a simple shear device, thus this leads logically to the study of stress changes in a simple shear device in order to deduce the stress state of the soil in the shear zone close to the pile surface.

A major objective of this research is thus the development of a simple shear apparatus, since the shear box, the other conventional direct shear type of apparatus, is associated with significant degree of non-uniformity of internal stress and strain conditions and difficulty in estimating contractile strain and soil modulus.

1.2 THE AIM AND OBJECTIVES

The aim of this research is to investigate whether or not contractile strains are induced in low density unsaturated residual sands subject to simple shear strain and then to assess the effect of such contractile strains on the shear and radial stresses around a pile shaft, pile load settlement behaviour and capacity of vertically loaded piles in residual sands.

The major objectives of this study can be summed up as follows;

- The development of a simple shear apparatus for the laboratory simulation of the stress conditions in the shear zone adjacent to the pile shaft. These are the constant normal stress condition, the normal stiffness condition and the constant volume condition.

- To evaluate the magnitude of contractile strains induced in Berea Red Sands samples that were compacted to low density and subjected to simple shear strain.

- To assess the effect of such contractile strains on changes in the stress state around a pile shaft installed in Berea Red Sand formations.

- The determination of load settlement behaviour and capacity of vertically loaded piles in Berea Red Sand, using laboratory test results from a new simple shear apparatus.

1.3 SCOPE OF WORK

The scope of work can be broken down into three major interrelated sections as follows;

- Section A; The evaluation of the magnitude of moisture induced collapse settlement and shear induced compression of Berea Red Sands compacted to low dry density in the conventional direct shear apparatus. The oedometer and the conventional direct shear box devices were used for the laboratory determination of direct shear induced compression and the moisture induced collapse respectively.

- Section B; The formulation of stress equation for a specimen subject to simple shear strain and the development and construction of a simple shear apparatus. Constant volume, constant stiffness and constant normal stress tests were conducted on compacted samples of Berea Red Sands in the new simple shear apparatus.

- Evaluation of the load settlement behaviour and capacity of vertically loaded piles for constant stiffness and constant normal stress load transfer conditions in Berea Red Sand formations.

1.4 OUTLINE OF THESIS

The mechanical behaviour of naturally occurring low density residual sands is dependent on the complex interrelationships between mineralogy, suction, densities and inter particle bonding.

Frameworks or models proposed to characterize the behaviour in terms of the above variables were reviewed in chapter two. The mechanical behaviour of compacted soils is dependent on the interrelationships between mineralogy, suction, density and moulding water content. Similarities and contrasts with the behaviour of undisturbed tropical sands were presented here. Also presented in chapter two was a detailed review of direct shear devices and simple shear devices, +case histories of pile failures, methods for determining and predicting the load - settlement response of vertical loaded piles as well as the research plan.

The method of sample preparation and basic tests that facilitated the physical description of Berea Red Sands (BRS), the soil studied in this research, was presented in chapter three.

Investigation of the effect of compaction moisture content and dry density on moisture induced collapse settlement behaviour were presented in chapter four and direct shear induced compression behaviour of compacted Berea Red Sands were presented on chapter five.

Analysis of simple shear failure conditions in sands were presented in chapter six. A shear stress equation for stress conditions at the boundaries of a specimen tested in simple shear was formulated based on changes in the stress state at the core of a sand specimen subjected to simple shear strain in a Cambridge type device. The simple shear stress equation also facilitated the establishment of the general specification for sample dimension that guided the development and construction of a new simple shear apparatus.

The development, construction and testing of the simple shear device were presented in chapter seven.

Further evaluation i.e. proving of the apparatus which entails the comparison of the experimental and predicted stress - strain behavior as well as further evaluation of the effect of changes in specimen dimension as well as the effect of membranes on the stress – strain behavior of Berea Red Sands (BRS) subject to simple shear strain were presented in chapter eight.

The effect of moisture content on simple shear stress mobilized by samples that were subjected to constant normal stress, normal stiffness and constant volume were presented in chapter nine as well as results on changes in magnitude of matric suction due to shear induced volume compression of Berea Red Sands.

Details of the application of the t - z method for the determination of load – settlement behaviour of vertically loaded piles that were subjected to normal stiffness condition in Berea Red Sands were presented in chapter ten. A newly formulated equation that predicts the resistance of the soil below the pile tip to the settlement of the pile tip was also presented in chapter ten.

In chapter eleven, the final chapter, the summary, conclusions and proposals for further research were presented.

CHAPTER TWO

LITERATURE REVIEW

2.1 INTRODUCTION

Excessive settlement of some off shore structures in cemented carbonate deposits has been reported by King and Lodge (1988) and Coop and Atkinson (1993) while Jackson (1980) and Chueng et al, (1988) have documented failure of structures associated with excessive settlement of the foundations in residual deposits. Problems associated with the prediction of settlement of structures founded in these soils may be linked to their complex mechanical behavior.

Residual soils have undergone varying degrees of weathering which have either altered or completely erased their stress history. This means that void ratio and stiffness alone cannot completely characterize the mechanical behavior of these soils. Consideration has to be given to weathering state, bond strength, suction and mineralogy (Vaughan et al, 1988).

The shear strength of remoulded residual soils are significantly less than that of the in situ undisturbed deposits and these remoulded soils often require some degree of compaction for strength improvement (Vaughan et al, 1988). The mechanical behavior of compacted soils is influenced by mineralogy, compaction moisture content, suction and relative density. Vaughan et al, (1988) indicated that the mechanical behavior of weakly bonded soils is best considered in relation to the behavior of the same soil in the unbonded destructured state, as it is towards this state that the soil will tend as it is subjected to large strains. Compacting residual soils destroys the fragile weakly bonded structure of the undisturbed soils, however carefully controlled compaction process enables the production of large numbers of samples at selected moisture contents and dry densities.

Brief reviews of soil mechanics and geotechnical literatures, relevant to the present studies, presented in this chapter, are divided into three major sections in relation to the three areas of studies that embody the current work. These are (a) review of methods of characterizing the shear strength and deformation behavior of structurally unstable undisturbed and compacted soils, (b) direct shear devices and distribution of stress and strain in samples tested in these devices as well as existing analytical methods of predicting the behavior of soils in simple shear, (c) stress conditions in sands close to the shaft of a vertically loaded pile, case histories of excessive settlement of piles as well as methods of determining and predicting the load – settlement behavior of vertically loaded piles.

2.2 EFFECTIVE STRESS CONCEPT FOR UNSATURATED SOILS

While effective stress models for saturated soil (Terzaghi 1936) were able to predict changes in shear stress due to externally applied normal stress, accurate prediction of associated volume changes remain unresolved until Skempton (1961) reformulated the effective stress model in terms of the different effects of the effective stress on measured volume and shear strength. The effective stress concept compatible with the mechanics of saturated soils was extended to unsaturated soil by Bishop (1959).

Bishop's equation is of the form:

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

where σ' = effective stress

σ = total stress

χ = factor related to the degree of saturation

u_w = pore water pressure

u_a = pore air pressure

The use of the effective stress law was criticized by Jennings and Burland (1962). The extreme sensitivity of the parameter χ to such factors as soil structure, the cycle of wetting and drying or of stress change and the occurrence of volumetric strain of opposite sign to those predicted by changes in the apparent effective stress were matters of concern. Bishop's proposal was extended to interpret volume change behavior by Blight (1967).

Further inquiry has led to the common practice of using the stress state variables, applied stress $(\sigma - u_a)$ and suction $(u_a - u_w)$, to describe the stress state in the soil (Schreiner and Gourley 1993).

While this is technically acceptable, it serves to mask the stresses, which are of interest in the natural state as the pore air pressure is an artificial and undesirable part of these stress variables especially with regards to interpretation of soil behavior in situ. The stress variables are often reformulated as vertical or axially applied stress ($\sigma_v - u_a$); radial or horizontal applied stress ($\sigma_r - u_a$) and the variable ($u_a - u_w$) which is numerically equal to suction but of opposite sign.

Not only have these been shown to be the stress variable of importance but they also reflect the stresses that exist in the field in most instances (Schreiner and Gourley 1993). The only situation in the field in which these stress variables may be unsatisfactory is when there are bubbles of air within the soil water in which case the air pressure is significantly different from atmospheric pressure. This could occur during compaction at high moisture content. Burland (1961) has however shown that such bubbles must be transient because the air will in time diffuse through the water at high pressure to the low pressure of the free atmosphere.

Fredlund and Morgenstern (1977) proposed the use of independent stress state variables for an unsaturated soil. The general form of the equation put forward by Fredlund (1979) for the shear strength envelope of unsaturated soils is

$$\sigma' = c' + (\sigma - u_a) \tan\phi' + (u_a - u_w) \tan\phi^b$$

where

σ' is the shear strength of the soil

σ is the applied stress

c' is the effective cohesion

ϕ' is the angle of friction for changes in ($\sigma - u_a$)

ϕ^b is the angle of friction for changes in ($u_a - u_w$)

The use of a linear relationship between σ' and ($u_a - u_w$) was shown to be in error by Escario and Saez (1986). This non-linearity of the strength envelope was later confirmed by Fredlund et al. (1987). The use of two stress variables however places some restriction on the type of predictions that can be made. Most of the suggestions that have been proposed for describing the volume change behavior for unsaturated soils have not been completely successful (Maswoswe 1982).

The component of shear strength due to suction is expressed as χ ($u_a - u_w$) by Bishop (1959) while Fredlund (1979) expressed the same component as ($u_a - u_w$) $\tan\phi^b$, implying that while Bishop's effective stress approach is based on degree of saturation through the factor χ, Fredlund's approach was based on a direct relationship between suction and shear strength. Thus the parameter χ was redefined by Fredlund as tan ϕ^b.

The effective stress concept cannot predict the collapse behavior observed in some soils, however Bishops equation can be used to predict the in situ behavior of soils within accuracy generally adequate for engineering work (Maswoswe 1982, Toll 1991).

Fredlund's stress state variable model cannot be used directly to predict in situ behavior since ϕ^b is not a soil property and also because prediction of in situ behavior is dependent on the correlation of changes in soil strength with variation in in situ suction. Accurate prediction of field behavior by the stress state variable is dependent on the correlation of ϕ^b as function of χ and ϕ'.

A critical state framework for explaining the behavior of unsaturated soils was put forward by Toll (1990). The general equation is of the form

$$q = M_a (p - u_a) + M_w (u_a - u_w)$$

It was also extended to explain the volumetric behavior in terms of specific volume (ν) at critical state in the form

$$\nu = \Gamma - \lambda a \ln (p - u_a) - \lambda_w \ln (u_a - u_w)$$

where

q is the deviatoric stress

p is the mean effective stress

M_a is the total stress ratio

M_w is the suction ratio

Γ and λ are critical state parameters

Toll (1990) emphasized that in unsaturated soils the initial fabric is not destroyed by shearing at large strains. It has however been shown (Toll 1991) that differing fabrics at different degree of saturation cause variation in the M_a and λ_a parameters with respect to degree of saturation. It was observed that the specimens used by Toll (1990) were compacted at different moisture contents and different compactive efforts. The initial void ratios and saturation degrees vary considerably, producing a wide range of variation in the initial soil fabrics of the specimen. Thus several authors have questioned the uniqueness of the relationships presented by Toll (1990) when results from soils with various fabrics are combined.

A unique relationship might be anticipated if the initial fabric were destroyed during shear to produce similar structures at the end of the tests. The database with which Toll (1990) supported his hypothesis raised additional questions (Toll 1991). First there was the concern as to whether the same framework can be obtained for drained tests, and for unsaturated soil states, whether true critical state was achieved at the end of test since the framework was based on data in which the rate of dilation at the end of tests was far from negligible. Secondly was the concern as to whether a soil will become more compressible under an increasing applied load and constant suction with decreasing degree of saturation as the specific volume model tends to imply (Toll 1991). While several concerns have been raised, Toll (1990) model has been successful in providing insight into the study of the constant volume response of soils at different initial states.

2.3 BEHAVIOUR OF STRUCTURALLY UNSTABLE UNDISTURBED SOILS

There are a wide variety of deposits, which have been identified as having unstable or collapsible structure. These include colluvial deposits, residual soils, volcanic tuffs, aeolian sands and aeolian silts (loess). However the most extensive deposits of unstable soils are aeolian sands which are predominantly transported pleistocene sand of aeoline origin in which quartz and feldspar grains were deposited to form the initial loose open structure. Aeolian sands collapse when inundated if the deposit is subjected to external load higher than the overburden stress (Jennings and Knight 1957b).

Aeolian silts or loessial deposits differ principally from aeolian sands because of the presence of a relatively significant amount of calcite grains that soften when inundated at overburden pressure, causing collapse of the loess structure as well as loss of shear strength. Residual deposits develop principally as a result of leaching of soluble and colloidal materials from either rocks or transported materials. The leaching out of the soluble and fine materials results in a high void ratio and unstable structure with eluviation leading to low densities in the upper part of the subsoil (Vaughan et al. 1988).

The structural arrangement common to these naturally occurring deposits has been described as weakly bonded fabrics of predominantly quartzitic grains loosely held together by suction, clay bond or mineral bond or their combinations (Leroueil and Vaughan 1990). The bond strength of the undisturbed deposits is dependent on the degree of weathering and can vary by significant margins within a deposit (Vaughan et al. 1988). The significant variation in bond strength within deposits of residual tropical soils makes the study of the mechanical properties of undisturbed tropical soils costly and difficult.

Vaughan (1988) and Leroueil and Vaughan (1990) have related the mechanical behavior of residual soils to structure. The mechanical behavior of residual soils is similar in many cases, presenting a strong influence of the bonding. Residual soils show yield surfaces associated with their structure. The yield curves of residual soils are centered on the isotropic line, indicating an isotropic structure. The maximum stress is associated with large volumetric compression of the sample. The void ratio of residual soils may vary widely, independent of the source rock, the type of weathering and the stress history. Leroueil and Vaughan (1990) noted that the variations might be due to the variations in the amount of the weathering products, which have been leached from the soil. The strength envelope of residual soils indicates cohesion intercept due structure and bonding rather than to density and dilation (Leroueil and Vaughan 1990).

In a weakly bonded soil, the void ratio has a strong influence on drained strength, which increases with dry density. Vaughan et al. (1988) indicated that the mechanical behavior of weakly bonded soils is best considered in relation to the behavior of the same soil in the unbonded destructured state, as it is towards this state that the soil will tend as it is subjected to large strains. They stated that the void ratio and current stress state might be in one of the three states listed below in order of increasing intrinsic stability.

Meta – stable or structure permitted state; i.e. the natural soil exists at a void ratio which is impossible for the same soil in the destructured state at the same stress level. The soil can exist in this state only due to the strength and stability provided by its inter particle bonding.

Stable – contractive state, i.e. the soil could exist in the destructured state, but would contract during shear towards the constant volume critical state.

Stable – dilatant state, i.e. the soil could exist in the destructured state at the same stress level, but would expand during shear towards the critical state.

Thus if an in situ residual soil exceeds its yield stress either in shear failure or due to increasing average stress, then the strain which it will subsequently suffer depends on void ratio and stress state. Leroueil and Vaughan (1990) proposed that the state of an in situ residual soil can be assessed approximately by relating in situ void ratio to the liquid limit and optimum density as determined in the standard compaction tests.

In order to classify the residual soil according to their compression potential, Vaughan (1988) introduced the definition of relative void ratio, e_R, as

$e_R = (e - e_{OPT}) / (e_L - e_{OPT})$

where e = in situ void ratio

e_L = void ratio at the plastic limit

e_{OPT} = void ratio of soil compacted to Proctor maximum dry density

The merits of this method of classification rest on the low cost and simplicity of the compaction and atterberg limits tests while the reproducibility of e_L in the laboratory is a matter of concern.

2.4 BEHAVIOR OF COMPACTED SOILS

The structure and engineering behavior of compacted soils containing significant amount of fines will depend greatly on the method of compaction, the compaction energy (CE) and the water content at compaction. Usually the water content during compaction of the soil is referenced to the OMC as dry of the optimum, at optimum and wet of the optimum. Research on laboratory compacted samples have shown that when they are compacted dry of the optimum, soil structure is independent of whether kneading or impact method are used. Wet of the optimum water content however, the compaction method has a significant effect on the soil fabric and thus the strength and compressibility of the soil.

At the same CE, with increasing water content, the soil fabric becomes increasingly orientated (dispersed). Dry of the optimum, the soil tends to produce flocculated fabric. If the compactive effort is increased, the fabric tends to become more dispersed even though the water content remains constant.

Laboratory tests also indicated that shrinkage is less for soils compacted on the dry side of optimum due to the combination of the flocculated fabric, sensitivity to additional water at the contact points, and the lower reference for swelling. Flow of water through soil (permeability) depends both on the void ratio and fabric orientation, laboratory tests indicate that permeability decrease at constant CE and increasing water content since the dispersed fabric is less permeable. It also decreases at increased CE and constant water content also because the dispersed fabric is less permeable.

Compressibility of compacted soils is a function of both the method of compaction and stress levels subsequently imposed on the soil mass. At relatively low stress levels, clays compacted wet of optimum appear to be more compressible. At high stress level, clay compacted wet of the optimum are less compressible. This is due to the flocculated structure produced on the dry side of the optimum has a large accumulation of interparticle bonds from water deficiency compared with the dispersed fabric on the wet side of the OMC. Application of the low stresses on the dry side are not sufficient to overcome the particle bonding to first (1) reorient the fabric to a more dispersed state and then (2) squeeze particles closer together, so compressibility (reduction in void ratio) is low. On the wet side, step 1 has already been done in the compaction process so that step 2 effects predominate and the compressibility is larger.

At high stresses such that interparticle bonds are broken both steps 1 and 2 occur for the flocculated (dry side) fabric and result in a large amount of compressibility. For the wet side, factor 2 predominates with a dispersed fabric so that less compressibility is produced.

Barden et al, (1973) and Mitchell (1976) recognized four conditions necessary for collapse to occur in a soil. These are an open partially unstable partially saturated fabric, a high enough total stress so that the structure is metastable, a sufficiently large soil suction or the presence of a bonding or cementing agent that stabilizes the soil in a partially saturated condition and the addition of water to the soil which reduces the soil suction or softens or destroys the bonding agent, thereby causing shear failure at the interaggregate or at the intergranular contacts.

For any given set of conditions, the amount of collapse of compacted fine sands generally decreases with increasing precollapse moisture content, increasing pre collapse dry density and increasing overburden pressure (Cox 1978; Lawton et al. 1990). For any compacted sand, there are combinations of initial dry density, molding water content and overburden pressure at which no volume change will occur when the soil is inundated (Lawton et al, 1990). For compacted sands there appears to be a critical moulding water content at soaking above which no collapse will occur. For some soils the critical water content is above the standard proctor optimum water content. Also there is a critical degree of saturation for a given soil above which negligible collapse will occur regardless of the magnitude of the prewetting overburden pressure.

Strength is also dependent on the strain defining strength. At large strain i.e. 20 % strain, strength of compacted soils is independent of the moulding water content. Apparently the large amount of sample remoulding during strain produces the same ultimate fabric in the failure zone.

At small strain (1% - 5%) however, strength of soils compacted on the dry side of the OMC produce larger strengths. When compacted on the dry side of the OMC and later saturated, the strength is also higher than for soils compacted on the wet side and saturated, but at high strains strengths are about equal. Thus where strength at low deformation is critical and swelling is not a problem, the soil should be compacted dry of the optimum.

Although sands containing small amount of fines cannot easily be compacted using either the impact or kneading methods, these soils can be compacted using the confined static compression or a combination of confinement and vibration. However unlike soils containing significant amount of fines, density and compaction moisture content rather than the magnitude of negative pore water pressure and structure control moisture induced collapse settlement.

The arrangement of coarse grained particles can be dense or loose depending on the degree of packing. The fabric of the soil is said to be single grained and the strength of single grained soils is derived from the friction between the surfaces of the grains in contact. The area of the surfaces in contact increase with the degree of packing and therefore strength. The volume change characteristics for low fine sands compacted wet and dry of optimum water content are independent on stress path (Barden and Sides 1970).

2.5 SUCTION

Suction and applied stress are the two stress variables controlling soil behavior, Bishop and Blight (1963). The total suction is made up of two components; matrix suction and osmotic (solute) suction, but only matric suction plays a part in the mechanical behavior of the soil (Bishop and Blight 1963, Maswoswe 1985).

Matric suction is the negative gauge pressure on the soil water to which a solution identical in composition to the soil water must be subjected in order to be in equilibrium through a porous permeable membrane wall with the soil water (TRL 1993).

At a microscopic level, the concept of matric suction involves both the capillary properties of the medium, characterized by the radius of the pores and the adsorption properties of the clay minerals, which depends on the type of minerals and surface ions (TRL 1993, Fleureau et al, 1993).

Laboratory measurement of suction by the concept of axis translation was first demonstrated by Hilf (1956) and verified by Bishop and Blight (1963). The technique involves the elevation of the pore air pressure, thus raising the pore water pressure in the sample to positive values. High air entry porous ceramics are used for the independent measurement of pore air pressure and pore water pressures. The axis translation technique can be used for both the control and measurement of matric suction of samples in the triaxial apparatus and in the oedometer.

Bocking and Fredlund (1980) showed that there are some limitations to the general application of the axis translation method. They pointed out that if the soil tested does not have a continuous air phase, but air is present as occluded bubbles, then the suction can be overestimated while diffusion of air through the porous ceramic can give an under – estimation of the suction.

Schreiner and Gourley (1993) have presented a comprehensive review on the methods of measuring and controlling the various component of suction in the laboratory. The review revealed that the pressure plate system is the most direct procedure for measuring suction in the laboratory and should be the preferred routine or standard procedure to be used in the laboratory. However the investigation indicated that the equipment required is moderately complicated and expensive and in general only one measurement can be made per day per pressure plate cell.

The report also revealed that although the filter paper method is an indirect technique of measuring suction, it provides good measurement of matrix suction using the Chandler and Guiterrez (1986) calibration chart. Due to its low cost and simple procedure, it can be used on many samples.

Schreiner and Gourley (1993) show the filter paper techniques to be somewhat sensitive to the quantity of moisture pulled out of the sample by the filter paper mass, emphasizing that difficulties may be encountered in matrix suction measurement at suctions above 1000 kPa. It is believed that at such high suctions there may be inadequate liquid to liquid contact between the water phases, implying that it is highly probable that the contact procedure is measuring total suction instead of matric suction.

2.6 DIRECT SHEAR APPARATUS

Although detailed studies of the mechanics of soils are mostly based on results of triaxial tests, a major limitation of the triaxial apparatus is that in its simplest form it is a poor simulator of common field conditions (Randolph and Wroth 1981; Blight 1997).

Evidence that the triaxial apparatus is of limited relevance for following the shear plane orientation common in geotechnical construction has been provided by Hutchinson (1961) who observed that the shear strengths measured in a large in situ shear box were in considerably better agreement with the values derived from the analysis of reported land slides than the values observed from conventional triaxial tests. Bjerrum and Landva (1966) and Airey and Wood (1987) have also reported better correlation between strengths from vane tests (a direct shear type device) and strengths from back analysis of slope failures.

A soil element in situ can experience varying directions and magnitudes of principal stresses. In the triaxial cell, the applied axial stress and cell pressures are principal stresses and their directions are fixed unlike the shear box and the direct simple shear devices where shearing is associated with rotation of principal stress (Symes et al, 1984).

The pattern of stress in a direct shear test of an artificial soil (crushed glass) studied by photoelasticity method showed that the shear load is transmitted to the soil through the end walls

(Dyer 1985). This mechanism has been confirmed by pressure cell measurements on the end walls of a large direct shear apparatus (Palmeria 1987). Insignificant magnitude of stress is transmitted to the soil from the upper and lower boundaries in the test.

The consequence of load transfer through the end walls is to cause the soil sample to be subjected to an equal and opposite couple. The couple causes rotation of the sample during a conventional direct shear test to enable additional stresses to be mobilized between the soil and the side walls of the apparatus to restore moment equilibrium (Dyer 1985 and Airey 1987).

Consideration of the effect of test boundary on stress – strain behavior revealed that when a pressure bag is used to apply uniform vertical stresses to the sample, there is a greater tendency for non-uniform rotation. Non-uniform rotation in the soil enables the sample to mobilize a higher shearing resistance on the central plane. The shearing resistance may be increased further by roughening the end walls in the test. Jewell (1989) indicated that improved uniformity of stresses and strains within the sample could be achieved by firmly securing the rigid top loading platen to the top half of the apparatus so that the upper half of the apparatus moves as a unit during shear.

The tests is considered by some researchers (Jewell 1989, Dyer 1985) as meeting the requirement of plain strain tests since the sample is confined such that only lateral and vertical movements (strains) can take place.

The relatively low cost and operational ease makes the shear box preferable to the triaxial apparatus for a quick study of effective stress problems. However the direct shear box remains unpopular because of its inability to impose uniform shear stress on soil samples. The direct shear tests forces the direction and location of the failure plane. The failure plane is located at the split of the box and parallel to the horizontal load. Practically this condition may not be obtained in the field and also may not be directly applicable in design since the predetermined central plane of failure, may not be the plane of maximum shear stress in the sample (Randolph and Wroth 1981).

The shear zone in this apparatus is confined to a narrow but undetermined band defined by the split in the box. It is thus impossible to determine contractile strains due to shearing, both because the thickness of the shear band is not known as well as the limitations on making sufficiently accurate measurements of the vertical displacements that would occur due to contractile strain in this narrow band. There are, furthermore, concerns about the stress distribution within the sample (Roscoe 1953; Airey and Wood 1988).

The rigid end boundaries in a direct shear test remain fixed and impose an overall restriction of lateral strain in the soil along the central plane. Internal measurements using radiography have shown that this restriction also applies locally in the soil along the central plane in the test (Scapelli and Word 1982; Dyer 1985).

Earlier Roscoe (1953) had observed the same restriction in the Cambridge simple shear apparatus. Thus it is expected that tests in both the direct and simple shear apparatus would mobilize the direct shear angle of friction in the central plane. This has been confirmed by Stroud (1971) and later by Jewell (1989) who measured independently both the stresses and the incremental strains in Leighton Buzzard sand in simple shear. Tasuoka et al. (1988) confirmed these observations from tests on sandy soils in a Torsional simple shear device.

The behavior of soil in the direct shear box has received little numerical study. Finite element analysis of direct shear box tests in which the soil is modeled using an elastic – plastic constitutive law was reported by Potts et al, (1987). In their analysis, the stress state in the simple shear device was used as the reference state. For non strain softening soils, their analyses show that, despite the strongly non-uniform stresses generated within the box, the peak shearing resistance observed is close to the ultimate strength in simple shear. Exact agreement was obtained when the angle of dilation (ψ) is equal to soil effective angle of friction (ϕ'). On average the shear box overestimate the simple shear value by 6% when ψ is zero.

Analyses in which severe strain softening is simulated indicate that, despite the strongly non – uniform stresses within the box before failure, strains and stresses within the failure zone are surprisingly uniform.

Although the numerical work by Potts et al. (1987) provided insight to the uniformity of stress and strains, such a study cannot determine the correct method of interpreting the direct shear box test on real soil, as the validity of the soil model used must be presumed and a value for the angle of dilation assumed. In the finite element analyses, they idealized the soil as an isotropic linear elastic – plastic material operating in the drained state with its properties defined by a Young Modulus E, Poisson Ratio, μ, Angle of shearing resistance, ϕ, and Cohesion, c. In most real soil analyses, the stress - strain properties are difficult to predict and have to be determined experimentally because they are dependent on the applied stress and may be also affected by bond strength and suction.

2.7 SIMPLE SHEAR APPARATUS

2.7.1 TYPES OF SIMPLE SHEAR APPARATUS.

The simple shear apparatus emerged as a result of research attempts to modify the direct shear box to impose a uniform simple shear state on a sample. A principal requirement of any simple shear apparatus is that the sample should be uniformly strained in simple shear and plane strain.

Simple shear state of strain is a plain strain mode of straining i.e. horizontal planes through the samples are planes of zero extensions. A second requirement is that stresses on the horizontal plane must be uniform to satisfy simple shear condition.

However for shear stresses across the horizontal cross section to be uniform in simple shear, the adjoining vertical walls of the apparatus must be able to impose complementary uniform vertical shear stress on the samples. Secondly in order that the sample may be able to change in height without frictional restrictions, it is necessary that the ends of the apparatus be relatively free from shear stress i.e. the distribution of shear stresses over the top and bottom faces has to fall to zero at the ends of the samples. This can be achieved only if the resulting couple is counteracted by an opposite couple generated by a non uniform distribution of normal stresses at the top and bottom faces.

Since the degree of freedom of the vertical walls is limited, non-uniformity of boundary stresses has remain an unavoidable problem of all simple shear devices, since it is unlikely that an apparatus that can satisfy the condition of simple shear can be made.

There are two significantly different forms of devices that can impose simple shear deformation on a sample; the NGI/SGI device (Kjellman 1951) and the Cambridge device (Roscoe 1953). The first generation NGI simple shear device was inspired by the need to simulate in the laboratory the strain conditions in the field where a large flake of soil moves horizontally as a result of shear deformations in a thin soil layer (Kjellmann 1951; Bjerrum and Landva 1966). The apparatus was built to perform drained and constant volume tests. By having the samples confined in a reinforced rubber membrane and using relatively thin specimens it was hoped that uniform straining could be achieved. Later generation NGI devices focused mainly on incorporating pressure chambers, back pressure and pore pressure measurement accessories (Dyvik et al. 1987, Airey and Wood 1987).

Common to all NGI type devices is the problem of obtaining accurate measurement of the sample volume changes since the rubber membrane is not infinitely stiff and thus is likely to yield under load. This apparatus was first designed to assess the shear strength of quick clay where operationally small changes in the diameter of the specimen caused by strain in the rubber membrane resulting from changes in the horizontal normal stress are only associated with small testing errors. Roscoe (1953) however noted that the NGI device suffers from the objection that the samples are cylindrical and the shear stresses across a horizontal circular cross section cannot be uniform since they must be tangential to the circular boundary unless the vertical walls of the apparatus are capable of imposing vertical shear stresses to the sample.

In order to maintain constant volume during simple shear, especially for materials that are stiff and significantly elastic, the vertical walls of the NGI device that were reinforced at frequent intervals by metal rings which are not constrain vertically must stretch under the action of the external shear force but at the same time the rubber should be so rigid that it does not move under the action of the contact stresses with the soils. Theses two conditions are mutually incompatible.

Another concern is that volume changes are not likely to be recorded correctly owing to some yield of the rubber tubing (Franke et al. 1979).

The Cambridge simple shear apparatus was developed to facilitate detailed study of the behavior of soil particles that were participating in the narrow zone of failure in the direct shear box (Roscoe 1953). Thus the simple shear apparatus developed at Cambridge represents attempts to compel the entire soil sample to become the zone of failure. This apparatus imposed simple shear strain in a sample confined in a rigid box. Subsequent Cambridge devices (Roscoe et al, 1967; Airey et al, 1985) were equipped with arrays of contact stress transducers and load cells for the measurement of the distribution of normal and shear stress applied to the sample and the stress tensor in the central part of the apparatus. These later devices are operationally complex and are not commercially available for routine testing.

2.7.2 STRESS CHANGES DUE TO IMPOSED SIMPLE SHEAR STRAIN

The stress changes which occur during simple shear test have been discussed by several workers, both from the point of view of uniformity of the induced stresses (Roscoe, 1953; Duncan & Dunlop, 1969; Lucks et al. 1972; Prévost & Hoëg, 1976; Budhu, 1979) and also in terms of the rules governing the stress state at failure (de Josselin de Jong, 1971; Ladd & Edgers, 1972). From these studies it was assumed that the measured average shear stress and normal stress on the top and bottom faces of the sample were indicative of the stresses on the horizontal planes throughout most of the sample. At failure however, the effective stress circle will be tangent to the Mohr-Coulomb failure envelope-defined by an angle of internal friction, so the shear stress mobilized on the horizontal planes in the simple shear test at failure will not necessarily be equal to the shear strength (Parry & Swain 1977b; Randolph and Wroth 1988).

The ratio of mobilized shear stress to ultimate shear strength will depend on the angles which the planes of maximum stress obliquity make to the horizontal (Parry & Swain 1977b; Randolph and Wroth 1988).

In an undrained or constant volume simple shear test, it is believed that the applied strain increment is one of pure shear plus rigid body rotation, such that horizontal planes remain horizontal (de Josselin de Jong, 1971; Randolph and Wroth 1988). Since it is assumed that no volume change occurs, maximum shear strain takes place between the planes of zero extension, which are horizontal and vertical. In the early stages of the test, it is to be expected that the stress increments also correspond to those of pure shear, with little change in the value of the vertical and horizontal stresses (Ladd and Edgers 1972). However, as plastic strains become significant in the sample the stress increments depart from those of pure shear. For non stiff sands, plastic strains will occur right from the start of the test; for stiff sands, as the degree of overconsolidation increases, it is reasonable to expect an increasing tendency of the initial stress changes to correspond to those of pure shear and depart from pure shear at large strains.

It is generally accepted that specimens in simple shear devices are subject to boundary stress and strain concentrations, but there is controversy on whether these non uniformities significantly affect the test results (Vucetic and Lacasse 1982). Non uniformities of boundary stresses have been studied theoretically by Roscoe (1953), Luck et al, (1972) and Wright et al, (1978), and through direct experimental observation by Budhu (1979) and Airey (1984).

Direct experimental observation is however only possible in apparatus equipped with transducers which can measure the distribution of normal and shear stresses applied to the sample and this cannot be done on routine basis. Analysis performed of simple shear deformation and experimental observation that has been made, suggest that in the central path of the sample, provided that the height to length ratio is low, the inhomogeneity of stresses resulting from the non uniform boundary stresses will be low.

In general, the distribution of stresses inside the simple shear sample cannot be investigated, however the distribution of deformation within a soil sample can be studied by radiographic measurement of the position of lead markers and for granular soils, radiography gives the possibility of detecting local changes in density because the absorption of X Rays changes as the sample dilates or contracts (Roscoe 1953; Airey 1987).

Studies of ruptures and non uniformity in sand specimen indicate that once a rupture has formed, the simple shear sample is no longer homogenous. Scapelli and Wood (1982) have shown that depending on the initial state, ruptures can form before the peak stress ratio of the homogenous material. The formation of ruptures in sands is influenced by the degree of restraint that the apparatus impose on the soil and this in turn depends on the relative dimensions of the apparatus and grain size distribution of the sand samples. Studies about non uniformities of simple shear samples apply primarily at large strains. Thus it seems perfectly appropriate to present simple shear behaviour of sands at lower strains. The reference strain based on radiographic studies indicates that regions of intense shearing show a much higher rate of dilation and constant volume critical state condition is often reached at shear strain between 0.5 and 1.0.

In most commercially available apparatus, the only measurements that are possible are the total normal load and the total shear load, from which the average normal stress and the average shear stress can be computed and that is not sufficient to describe the stress tensor since these stresses which are based on forces measured at the boundaries of the apparatus, differ in magnitude from the actual normal and shear stresses applied to the soil at the centre of the sample (Budhu 1984).

Measurements with single load cells reveal that the actual stresses measured at the core of the samples are greater than the values measures at the boundaries of the apparatus. The uncertainty about the stress tensor arises because of the lack of information about the intermediate principal stress normal to the direction of simple shear deformation which is readily accounted for in triaxial deformation.

Experimental studies by Budhu (1984) on dense sands revealed that regardless of non uniformities of stress and strains, the behavior of soils in SSA and triaxial apparatus should differ because they were designed to impose different stress conditions on the soil sample. The study revealed that the often ignored intermediate principal stresses in the SSA play a significant role in the response of soils tested under plain strain conditions. The stress path followed by sands in the two apparatus differs significantly when the intermediate principal stress is taken into account. However elaborate instrumentation like the Cambridge SSA MK2 is needed to study the complete state of stresses and strains.

In such elaborately instrumented device, the stress conditions in the sample core, far away from the stress concentrations at the ends closely approximate the ideal simple shear conditions. Thus the horizontal plane on which the failure normal and shear stresses act could be a plane of maximum stress obliquity or it could be a plane of maximum shear stress or in accordance with de Josselin de Jong (1971) it could be a plane defined by vertical shear and rigid body rotation. Further evaluation by Airey et al. (1988) revealed that the mode of failure is dependent on the initial state of stress at the start of simple shear deformation.

Randolph and Wroth (1989) also attempted to fit data obtained from undrained simple shear tests into a consistent framework. It has been long realized that strengths measured in simple shear are generally lower than strengths measured in triaxail compression. Randolph and Wroth proposed a rationale for the difference based on two hypotheses;

* At failure, the ratio of the major and minor principal stresses approaches a limiting ratio which is largely independent of the value of the intermediate principal stress.
* Soil at particular water content will approach a particular value of mean effective stress as it is sheared to failure, regardless of the exact mode of shearing.

The intermediate principal stress has been written in terms of the major and minor principal stresses by making use of the parameter b (Bishop, 1966) given by

$$b = (\sigma_2' - \sigma_3')/(\sigma_1' - \sigma_3')$$

b varies from 0 for triaxial compression tests up to 1 for triaxial extension tests.

The parameter b is determined by the type of shear test and vary under different boundary conditions. For extreme values of b, the limiting stress ratio is similar for b = 0 (triaxial compression) or b = 1 (triaxial extension) (Parry, 1960). However, there is some evidence that at intermediate values of b, the effective angle of friction increases slightly.

Although the value b in triaxial tests is determined precisely by the boundary conditions ,assuming the stress state is homogeneous throughout the sample, in plane strain tests the parameter must be determined by experiment since the intermediate strain is fixed (at zero) rather than the stress. Ladd et al (1971) reported values of b of about 0.37 while other workers (Bishop, 1966; Sketchley 1973) give values between 0.4 and 0.5. Most analysis based on simple shear tests results use a value of 0.5 i.e. it is assumed that the intermediate principal stress at failure is equal to the average of the major and minor principal stresses. This value is often preferred in order to be consistent with many theoretical soil models, where the existence of a yield locus symmetric about the principal stress axis is postulated (Randolph and Wroth 1981).

2.7.3 NUMERICAL ANALYSIS OF SIMPLE SHEAR BEHAVIOUR

Airey et al, (1985) reported that in the case of significant non uniformity of stress distribution in the simple shear apparatus, one would expect to measure a higher strength for a specimen of large diameter or width if the height is kept constant, as the proportion of the sample along the edges presumably subjected to local stresses and strain concentrations decreases relative to the entire specimen volume. The stress and strain distribution within the specimen are both uniform and closer to the values calculated directly from the applied forces and displacements when the height to diameter is small and the edge effect are minimized (Airey et al, 1985).

The relationship between the boundary stresses measured in NGI and Cambridge devices and stresses on a soil element in the two devices, investigated by theoretical analysis revealed conflicting results. Lucks et al. (1972) presented an elastic three-dimensional finite element solution for undrained tests in a NGI simple shear device. From analyses with axially symmetric element loaded unsymmetrical, Lucks et al. (1972) concluded that approximately 70 % of the specimen has a uniform stress condition and that the average shear stress increment applied in the direction of the translation lies within 2 % of the horizontal shear stress in the zone of uniformity.

Shen et al. (1978) presented a numerical study of the influences of both height to diameter ratio and membrane stiffness on the stress - strain distribution. They predicted the stress strain behavior of an idealized linear elastic solid specimen in the NGI device. The computation was performed with three dimensional finite element analyses of a nonaxissymetrically loaded axisymmetric solid. Orthotropic elements were incorporated in the program to simulate the action of the wire reinforced rubber membrane.

They predicted the stress - strain behavior of an idealized linear elastic solid specimen in the NGI device. The work reported that approximately 40 % of the specimen has a uniform stress condition when the height to diameter ratio is set to 0.5. The uniformity of the stress - strain distribution however improved as the specimen height to diameter ratio decreases and as the ratio of the cross sectional area of the wire reinforcement and the rubber membrane increases. They however noted that the NGI device may introduce an error ranging from 5 % to 15 % in the shear modulus measurement. The shear modulus corresponded to a shear strain of 0.03 %. This conclusion may differ for measurements at shear strain of 3 %.

Wright et al, (1978) were even more critical of simple shear devices. They presented the stress distribution in the NGI simple shear device and concluded that the stresses varied throughout the test and that the non-uniformity of stress- strain distribution irremediably impaired the usefulness of the NGI simple shear test.

The max particle size of the soil dictates the minimum thickness of the test specimen (Chueng 1988). ASTM D 3080 specifies that the sample thickness should be at least six times the maximum grain size of the soil, and not less than 12.5 mm. In addition the specimen diameter or width should be at least twice the thickness. For testing sandy gravel in direct shear, Chueng (1988) recommended that the specimen thickness should be four to eight times the maximum grain size of the soil.

Numerical analysis of a soil sample 20mm thick and 60mm square in the first and later generations of the Cambridge device indicated that uniform conditions of normal and shear stresses could be expected over the central third of the specimen, and provided the height to length ratio is sufficiently low, the inhomogeneity of stresses resulting from the non-uniform boundary stresses will be low (Roscoe 1953; Duncan and Dunlop 1969; Airey et al, 1985)

Radiographic observations by Stroud (1971) on the variation of shear strain with depth of samples indicated that the formation of ruptures is influenced by the degree of restraint that the apparatus imposes on the soil which in turn will depend on the relative dimensions of the apparatus and of typical particles. Stroud (1971) observed that soils with higher average grain diameter – height ratio exhibited more severe rupture than soils with lower ratio. This implies that unless pure clay samples are tested, thickness of soil higher than the nominal 20 mm recommended for direct shear box tests will be required to prevent internal strain non uniformity and ruptures in the sample.

Emerging from some reported experimental works (Dyvik et al, 1988) is the observation that internal stress and strain uniformity is influenced by the height to length ratio of the apparatus and the soil average particle size (D_{50}). Thus it is difficult to specify the most suitable height to length ratio that would ensure minimal stress and strain non-uniformity for a particular soil type without running some trial tests at various sample heights.

Budhu and Wood (1979) analyzed the result of static tests on sands sheared in the NGI and Cambridge simple shear apparatus. They showed that both apparatuses developed non uniform boundary stresses on the sample and concluded that the average ratio of shear stress to normal stress on the top and bottom horizontal boundaries underestimated the ratio in the centre of the sample by about 12 % in monotonic loading.

Ladd and Edgers (1972) presented results of simple shear testing of clays in the NGI simple shear device. For clays of different plasticities, they concluded that the stress and strain appears generally uniform and that the measure values of shear strain, horizontal shear stress and vertical effective stress represents adequately the average conditions within the specimen.

Roscoe (1953) reported, based on theory of elasticity, that uniform condition could be expected over the central third of a specimen testing in the Cambridge device. Duncan and Dunlop (1969) using the finite element method have confirmed this claim. Budhu and Wood (1970) analyzed the result of static tests on sands sheared in the NGI and Cambridge simple shear apparati. They showed that both apparati developed non – uniform boundary stresses on the sample and concluded that the average ratio of shear stress to normal stress on the top and bottom horizontal boundaries underestimated the stress ratio in the center of the sample by about 12 % in monotonic loading.

Common to the theoretical studies mentioned above is the assumption of a linear elastic material behavior. Quantitative conclusions from these analyses are limited, since these analyses cannot take into account the influence of the plastic yield and the changes in stress concentration during shear of real soils.

2.8 BEHAVIOUR OF PILES IN SANDS

2.8.1 STRESS CONDITIONS ALONGSIDE A PILE SHAFT

Irrespective of the method of installation, it is generally assumed that pile set up effects are negligible as any pore pressure changes that might develop during installation would be quickly dissipated in permeable deposits. However Tevenas et al (1970) found the compressive capacity of concrete piles in a medium dense uniform sand to increase with elapsed time after installation.

Pile installation in a granular soil can rearrange the fabric of the soil, change the soil density and crush the grain to result in a different grain size distribution. The stress-strain and dilatancy / collapse behavior are affected by soil fabric, which is the spatial arrangement of solid particles and associated void space. Installation process is likely to orient particles along the shaft with the long axis of the particle parallel to the shaft. Data obtained by Oda and Koishikawa (1977) and Ochiai and Lade (1983) indicate that fabric changes alongside the pile shaft, should result in a reduction less than 5 degrees in friction angle measured with the particle alignment to the major principal axis. The effect of fabric changes on the stress-strain response could be important to shaft resistance. Tests by Miura and Toki. (1982) showed that fabric affects toe resistance and that the effect is directly related to difference in measured friction angle which varies by less than 2 degrees at any given relative density.

The effect of grain crushing is expected to be smaller for finer sized particles and for less angular particles. Also for open end pipes, the zone of influence for density changes and grain crushing, as well as fabric changes is affected more by the wall thickness of the driving shoe than the pile diameter. The zone of influence along the shaft under these conditions is expected to be too small, only a few inches. The effect of density changes and grain crushing on toe resistance, which would be controlled by the mass of the soil within two diameters of the toe, is small for low displacement piles, and any beneficial effect should be counted on in design, because the degree of plugging during driving is not predictable. The pattern of density changes to be complex and non uniform (Robinsky and Morrison 1964). The vibration of driving should result, with few exceptions, in permeable soil such as sands next to the pile that are as dense or denser that the undisturbed in situ soil.

The conditions near the pile play an important role in the axial capacity. In granular soils, piles and cone penetrometers are usually clean when extracted. Results of testing by Yoshimio and Kishida (1981) show that failure in granular soils may occur at the pile surface or in the soil close to the pile surface. Several studies have been performed on the shear resistance at the interface between soil and foundation materials (Potyondy 1961; Yoshimio and Kishida 1981; Acar et al, 1982) and the findings suggest that for siliceous, quartzitic and calcareous sands, the ratio of the pile soil interface angle to soil internal frictional angle is in the range of 0.6 to 0.7.

The state of stress prior to pile loading will influence the stress path during loading and the failure state can be influenced by the magnitude of the stresses. The vertical effective stress next to the pile after set up and prior to loading is normally assumed in practice to be the effective overburden stress. Shear stresses at the pile – soil interface after set up will be neglected. The influence of residual stresses on pile capacity can be neglected although knowledge of pile residual stresses is quite important in the proper interpretation of load distribution along the shaft from instrumented piles and to axial load displacement behavior. The horizontal effective stresses next to the pile after set up and prior to loading is related to the initial in situ effective overburden stress.

The stress path that an element of soil along the shaft experience during loading is a key element to understanding soil pile interaction. Based on the work of Randolph and Wroth (1978) and Martins and Potts (1982), the stress conditions next to a pile can be modeled as a plane strain condition and furthermore, the boundary conditions are similar to that of a simple shear tests in which there is no volume change. For these conditions pore pressure may not develop, at least over extended loading duration, but the total lateral stresses can change to maintain the no volume change condition under shear.

2.8.2 CASE HISTORIES OF EXCESSIVE PILE SETTLEMENT

The first major study of deep foundation piles in unstable soils was on the large scale load testing of test piles from platforms in the Bass Straits Carbonate Beds. These tests illustrated that the frictional capacity of steel piles can be very low and use of conventional theory and design values may result in unconservative design (Angemeer 1973).

When the North Rankin platform was installed off Northwest Australia, unexpectedly low driving resistance indicated that the piles might not provide the design shaft friction capacity. The North Rankin platform was installed in bioclastic sediments formed by the deposition of skeletal bodies which predominates in the continental shelf environment and are commonly cemented with calcium carbonate deposited at the grain contacts and in the void spaces (King and Lodge 1988; Coop and Atkinson 1993).

On being subjected to shearing as the piles were installed, the soil formed from hollow spherical grains broke down leaving a much smaller bulk volume; this resulted in a zone alongside the piles filled with a dilute suspension of carbonate particles, across which virtually no stress could be transferred. The failure occurred despite extensive site investigation, which included in situ and laboratory tests as well as in situ pile element tests. A similar failure mechanism has been used to explain the subsidence of the Ekofisk oil platform installed in a chalk deposit of the North Sea from 1979 to 1985 at a rate of about 400mm/year (Potts et al, 1988).

The Motola formation of Southern Mozambique coastal plain is characterized by 15m to 20m thick mantle of red iron oxide cemented medium and fine sandy residual soils underlain by up to 6m thick pectenid – rich calcarenite and bioclastic limestone unit (McKnight 1999).

Preliminary design of pile foundation for a large aluminum smelter complex on the outskirts of Maputo on the Motola formation had to be re-evaluated when field tests revealed severe (13%) collapse settlement potential at saturation load of 1000 kPa. The pile load test program had to be modified to assess pile performance at working load when subjected to shaft and toe saturation (McKnight 1999).

Francis and Aucamp (1995) noted two cases of pile foundation failure in KwaZulu Natal. In the bluff formation of KwaZulu Natal, considerable settlement of pile foundation of an industrial complex led to design and construction re evaluation, when it was observed that all test piles were settling at average rate of 600 mm/year even though significant portion of the pile elements are below shallow water table. In the Berea formation, the construction of an Annex to the Administration buildings of Durban Institute of Technology had to be suspended twice following excessive settlement (<< 600mm) of the pile foundation.

The end bearing capacity of piles in a uniform deposit of non cohesive soil might not be expected to be directly proportional to the local vertical effective stress, and thus would increase approximately with depth. However research by Vesic (1975) and Fleming et al. (1992) has shown that the end bearing pressure appears to reach a limiting value beyond which no further increase is observed with further penetration of the pile. This limit assumed to be 11 MN/m^2 by Tomlinson (1980) and 15 MN/m^2 by Coyle and Castello (1981), was at one time attributed to some form of arching effect. However further evaluation (Fleming et al, 1992) indicated that more rational explanation lies in the variation of the rigidity index (the ratio of stiffness to strength) and friction angle with confining pressure.

The implication of this for bearing capacity is that as the embedded length of the pile increases, values of end bearing pressure increases at a gradual decreasing rate, and for practical pile lengths appear to asymptote towards values in the range of 11 MN/m^2 to 15 MN/m^2 (Coyle and Castello 1981; Fleming et al.1992).

The natural variation of pile end capacity with depth is non linear, generally showing a gradual reducing rate of increase with depth similar to those found in pile load tests. Only at great depths and for dense deposits of sands do end capacity exceeds 20 MN/m^2. It was noted by Fleming et al. (1992) that low values of end bearing pressure should be adopted for calcareous or very siliceous sands due to more compressible nature of these soils.

The appropriate value of K used for the determination of skin friction is related to the in-situ earth pressure coefficient, method of installation of the pile and the initial density of sand. Piles driven into loose to medium deposits of sands provide considerable compaction effect as the volume compresses. For dense deposits the greater tendency of the sand to dilate will limit the amount of densification close to the pile shaft.

The value of δ, the angle of friction between pile and soil will depend on the roughness of the pile surface. Kishida and Uesugi (1987) have reported a detailed study of the effects of surface roughness and shown how the interface friction angle may be related to the friction angle of the soil in terms of a normalized roughness coefficient, defined as the maximum roughness of the pile surface over a gauge length of D_{50} for the soil normalized by the value of D_{50}. For typical pile surfaces i.e. oxidized mild steel or concrete, the normalized roughness coefficient will exceed 0.05 and the coefficient of friction at the interface will lie in the range of 0.075 to 1 times that of the soil.

Generally the expressions given for calculating the shaft capacity of driven piles should be used with caution in certain types of cemented sands where the in-situ void ratio is high or in sands containing crushable particles. Breaking down of the cementation or and crushing of the soil particles during pile installation can cause liquefaction of sands. This can lead to very low values of effective normal stresses acting on the pile shaft and values of skin friction which are less than 20 % of those normally associated with silica sands possessing similar values of frictional angle (Fleming et al. 1992).

2.8.3 DETERMINATION OF PILE CAPACITY.

Although some very important trends have been interpreted from data bases of some pile load tests, including the apparent tendency for the average shaft shear stress to stop increasing below a critical depth and the dependence of local shear stress on pile radius, very little reliable data has been reported to date on the magnitude and distribution of the effective stresses acting on the shafts of driven piles in sands.

While many test piles in sand have been instrumented to determine the variation of axial load with depth, in most cases interpretation of test results has been hampered by large shifts in strain gauge out puts caused by pile driving. Unless special measures are taken such as those described by Gregersen et al, (1973), where embedded strain gauges were used to monitor accumulated internal stresses in the pile material during load testing, the residual post installation loads are generally indeterminate. If neglected, these can cause the compressive shaft capacity to be overestimated significantly and result in large errors in the evaluated shear stress distribution with depth (Lehane et al, 1993).

For these reasons most design methods for piles in sands have remained empirical and are highly approximate, for although there now exists highly sensitive pile instrumentation such as the Imperial College Test Pile used for in situ measurement of shear and radial effective stresses, it is still not possible to develop reliable and broadly applicable design methods without correlation and incorporation of data from well designed laboratory model experiments.

Research program at Imperial College in conjunction with the industry since the last decade to redress the above concern has led to a further development and improvement on the new design method initially developed by Jardine and Chow (1996). The updated version, the new design methods for piles in clays and sands by Jardine et al (2004) has found world wide application in offshore marine and on shore projects. The design method for driven piles based on recent work at Imperial College using the instrumented Imperial College steel piles, follows an effective stress approach and uses the Mohr – Coulomb failure criterion at the pile soil interface. .The difficulty in measuring radial forces and pore pressures at the pile soil interface has led to the simulation of the bored piles. Simulated bored piles of steel with grout annulus of 0 – 5mm has been tested. Since shearing takes place at the soil grout interface and the stress measurement are made at the grout steel interface, the success of the simulation depends on the grout being strong enough to with stand shearing while being not so stiff as to reduce excessively the total stress measurement sensitivity. The magnitude and time of effective stress equalization and the nature of the stress at the interface i.e. whether restrained dilation occurs during loading were factors that added to the reliability of the new method of pile design.

Because of huge cost, many field tests on drilled and grouted piles are not taken to failure, design values for peak skin friction determined from pile load tests have later been found to vary by a wide margin from the failure values. A few reported cases, i.e. the CFA piles installed in Durban Harbour Beds (Francis and Aucamp 1995) and foundation piles in the carbonate beds of Bass Straits in North Western Australia (Williams et al, 1988) indicated large discrepancies between estimated and measured values of settlement and pile capacity.

2.8.4 PREDICTION OF PILE CAPACITY

There have been two fundamental methods of predicting the load settlement characteristics of piles.

The first considers the distortion of the whole body of soil caused by the movement of the cylinder within it. Typical of this approach is the integration of the Mindlin (1936) equations for a pile divided into elements (Poulos and Davies 1968), approximate closed form solution for pile settlement (Randolph and Wroth 1978) and the finite element method. Common to these analytical methods is the difficulty of modifying the solutions obtained to take account of inhomogeneities or nonlinearity in the behavior of the ground that may occur at any particular site.

The second approach involves the division of a pile into elements and the determination of the load and settlement at each element that are compatible both with equilibrium and with the assumed shear stress – displacement relationship at the soil – pile interface. Seed and Reese (1957) proposed a method of analysis in which the soil resistance is approximated by continuous uncoupled soil springs, analogous to a Winkler foundation. This method effectively attributes stress- displacement characteristics to the soil – pile interface while the soil continuum is assumed to be effectively rigid. Actually the soil continuum response is empirically incorporated into the soil pile interface behavior through correlation with field measurement. Coyle and Reese (1965) and Coyle and Sulaiman (1967) extended the correlation by incorporating results from large triaxial cell studies of model piles and have developed procedures for determining the soil spring behavior of normally consolidated clay and sand respectively.

This approach is now widely used for the prediction of pile behavior. A limitation of the technique is the explicit assumption of a constant horizontal stress i.e. constant normal stress condition, in the estimation of interface stress – displacement characteristics, since the axial capacity of an elastic rod embedded in a soil formation which dilates or collapses during deformation may be considerably more or less than that indicated by the peak strength of the embedding soil formation. It was considered important to evaluate the possible stress conditions alongside the pile as these may affect the capacity.

Murff (1980), Scott (1981), Luker (1988) and Lehane et al, (1993) have identified three different soil interface stress conditions that are relevant in understanding the soil pile interaction and mobilized pile capacity. These are the constant normal stress, constant normal stiffness and constant volume stress conditions.

The constant normal stress condition represents the stress conditions where the average normal stress acting at the interface between the soil and the foundation material remains constant through out the test duration. The constant normal stress condition has been successfully used in the back analysis of some landslide problems by Hutchinson (1961). This stress condition may also be applicable to pile shafts installed by fast jacking into soft deposits (Scott 1981).

A constant stiffness condition rather than the constant normal stress condition is the pile interface stress conditions mostly encountered in the field since the plastic zone close to the pile surface may either contract or expand due to imposed shear strain, implying that the normal stress acting on the interface decreases or increases during shearing.

Esrig and Kirby (1978) and Wroth et al, (1979) noted that constant volume condition may prevail at the pile interface due to arching of the outer soil. They noted that pile interface stress condition represented by constant volume test is the stress state in the plastic zone close to the outer face of piles driven into offshore carbonate soils. The constant volume strength approximates the undrained strength under certain applied load conditions.

A closed form analytical solution of the influence of pile flexibility and stress deformation characteristics of the interface zone soils was presented by Murff (1980). It was revealed that while pile flexibility has a dominant influence on capacity for cases where the interface bond is uniform linear elastic – plastic, for a strain softening interface however, shaft capacity is controlled by changes in horizontal stresses.

The use of laboratory scale test procedures that were modeled to simulated stress conditions around a vertically loaded pile has been reported by a few researchers. Luker (1988) used shear stress data from direct shear box for the load transfer modeling of pile interface behavior of a vertically loaded pile based on the Winkler criteria.

A ring shear device, a direct shear type apparatus, was used by Ovando - Shelley (1995) to model the influence of earth quake loading on the capacity of friction pile in Mexico Clay. The decay in skin friction due to cyclic loading was modeled using a hyperbolic function of the spatial variation of pore pressure with time.

Series of cyclic interface tests and comparative testing of carbonate sand before and after mechanical crushing using the direct shear box showed no reduction in soil friction angle, thus such testing devices were not able to provide significant insight into the low shear stress measured in driven piles which is due to low normal stress on the pile wall (Murff 1987; Dutt et al, 1986).

A suggested reason for the low shaft capacity is that the coefficient of friction between carbonate soils particles and the pile wall is low. Valent (1979) and Berengen et al, (1982) have however shown that the coefficient of friction between calcareous sediment and steel and concrete surfaces is not significantly different from those of quartz sands, thus supporting the premise that the low shaft capacity is most likely due to reduced normal pressure on the pile wall rather than a reduced coefficient of friction.

2.9 SUMMARY OF LITERATURE REVIEW.

Problems associated with the prediction of settlement of structures founded in these soils may be linked to their complex mechanical behavior.

Residual soils have undergone varying degrees of weathering which have either altered or completely erased their stress history. This means that void ratio and stiffness alone cannot completely characterize the mechanical behavior of these soils. Consideration has to be given to weathering state, bond strength, suction and mineralogy (Vaughan et al, 1988).

The mechanical behavior of compacted soils is influenced by mineralogy, compaction moisture content, suction and relative density. Vaughan et al, (1988) indicated that the mechanical behavior of weakly bonded soils is best considered in relation to the behavior of the same soil in the unbonded destructured state, as it is towards this state that the soil will tend as it is subjected to large strains. Fredlund and Morgenstern (1977) proposed the use of independent stress state variables for an unsaturated soil. The use of the effective stress law was criticized by Jennings and Burland (1962). The extreme sensitivity of the parameter χ to such factors as soil structure, the cycle of wetting and drying or of stress change and the occurrence of volumetric strain of opposite sign to those predicted by changes in the apparent effective stress were matters of concern. Bishop's proposal was extended to interpret volume change behavior by Blight (1965).

The bond strength of the undisturbed deposits is dependent on the degree of weathering and can vary by significant margins within a deposit (Vaughan et al, 1988). The significant variation in bond strength within deposits of residual tropical soils makes the study of the mechanical properties of undisturbed tropical soils costly and difficult. A critical state framework for explaining the behavior of unsaturated soils was put forward by Toll (1990). The general equation is of the form. Suction and applied stress are the two stress variables controlling soil behavior, Bishop and Blight (1963).

Although detailed studies of the mechanics of soils are mostly based on results of triaxial tests, a major limitation of the triaxial apparatus is that in its simplest form it is a poor simulator of common field conditions (Randolph and Wroth 1981; Blight 1997). The relatively low cost and operational ease makes the shear box preferable to the triaxial apparatus for a quick study of effective stress problems. However the direct shear box remains unpopular because of its inability to impose uniform shear stress on soil samples. The direct shear tests forces the direction and location of the failure plane. The failure plane is located at the split of the box and parallel to the horizontal load. Practically this condition may not be obtained in the field and also may not be directly applicable in design since the predetermined central plane of failure, may not be the plane of maximum shear stress in the sample (Randolph and Wroth 1981).

The shear zone in this apparatus is confined to a narrow but undetermined band defined by the split in the box. It is thus impossible to determine contractile strains due to shearing, both because the thickness of the shear band is not known as well as the limitations on making sufficiently accurate measurements of the vertical displacements that would occur due to contractile strain

The simple shear apparatus emerged as a result of research attempts to modify the direct shear box to impose a uniform simple shear state on a sample. Non-uniformity of boundary stresses has remained an unavoidable problem of all simple shear devices, since it is unlikely that an apparatus that can satisfy the condition of simple shear can be made.

There are two significantly different forms of devices that can impose simple shear deformation on a sample; the NGI/SGI device (Kjellman 1951) and the Cambridge device (Roscoe 1953).

Common to all NGI type devices is the problem of obtaining accurate measurement of the sample volume changes since the rubber membrane is not infinitely stiff and thus is likely to yield under load. It is generally accepted that specimens in simple shear devices are subject to boundary stress and strain concentrations, but there is controversy on whether these non uniformities significantly affect the test results

The relationship between the boundary stresses measured in NGI and Cambridge devices and stresses on a soil element in the two devices, investigated by theoretical analysis revealed conflicting results. Common to the theoretical studies mentioned above is the assumption of a linear elastic material behavior. Quantitative conclusions from these analyses are limited, since these analyses cannot take into account the influence of the plastic yield and the changes in stress concentration during shear of real soils.

The stress path that an element of soil along the shaft experience during loading is a key element to understanding soil pile interaction. Based on the work of Randolph and Wroth (1978) and Martins and Potts (1982), the stress conditions next to a pile can be modeled as a plane strain condition and furthermore, the boundary conditions are similar to that of a simple shear tests
Murff (1980), Scott (1981), Luker (1988) and Lehane et al, (1993) have identified three different soil interface stress conditions that are relevant in understanding the soil pile interaction and mobilized pile capacity. These are the constant normal stress, constant normal stiffness and constant volume stress conditions.

Capacity of vertically loaded piles can be predicted by the integration of the Mindlins' equations for a pile divided into elements (Poulos and Davies 1968), approximate closed form solution for pile settlement (Randolph and Wroth 1978) and the finite element method. Common to these analytical methods is the difficulty of modifying the solutions obtained to take account of inhomogeneities or nonlinearity in the behavior of the ground that may occur at any particular site. The t – z method is widely used for the prediction of pile behavior because of its ability to account for inhomogeneities or nonlinearity of soil mechanical properties in some deposits. A limitation of the t – z method is the explicit assumption of a constant horizontal stress i.e. constant normal stress condition, in the estimation of interface stress – displacement characteristics.

2.10 RESEARCH PLAN

The aim of this research is to investigate whether or not contractile strains are induced in low density unsaturated tropical sands subject to simple shear strain and then to assess the effect of such contractile strains on the shear and radial stresses around a pile shaft, pile load settlement behaviour and capacity of vertically loaded piles in sands.

The sequence of the series of investigations that embodies this research work are presented below;

- Determination of the magnitude of moisture induced collapse settlement of compacted Berea Red Sands by oedometer tests and determination of shear induced volume compression of Berea Red Sands compacted to low density in the conventional direct shear box device.
- The formulation of stress equation for a specimen subject to simple shear strain.
- Development and construction of a simple shear apparatus.
- Laboratory study of constant volume, constant stiffness and constant normal stress simple shear behaviour of compacted Berea Red Sands in the new simple shear apparatus as well as the evaluation of the effect of changes in matric suction on magnitude of mobilized simple shear stress.
- Evaluation of the effect of imposed simple shear strain on the magnitude of contractile strain in sands compacted to low density.

- Determination of the load settlement behaviour and capacity of piles related to changes in radial stress associated with contractile strain in the plastic zone close to the pile surface.

Typical of residual sands are weakly bonded structure, high void ratio and high suction conditions and depending on the applied load and mineralogy, residual sands may exhibit moisture induced collapse. Moisture induced collapse can also occur in unsaturated silty and clayey sands compacted to low density, thus compacted sands of the Berea formation was chosen for this research.

Berea Sands are quartzitic sands containing small quantities of iron oxides and kaolinites, which exhibit reduction in bulk volume when inundated at a range of densities and applied load. Unsaturated samples of Berea Red Sands compacted to low density were used for this research. The use of compacted samples of Berea Sands facilitates the production and testing of fairly identical samples thus minimizing the problem of sample variability commonly associated with undisturbed samples of residual sand deposits.

The strain path undergone by the soil as the pile is loaded axially is similar to that undergone by soil in a simple shear test where the radial stress normal to the pile shaft is simulated as the vertical normal stress on the sample in the simple shear box. Hence in order to deduce the ultimate stress state of the soil close to the pile surface, it is logical to study the stress changes in the same soil in the simple shear device.

Thus a primary objective of this research was the development of a simple shear apparatus for the measurement of stresses and contractile strains in compacted Berea Red Samples. The design and development of the device would be facilitated by the formulation of simple shear stress equation.

The above formulation and solutions would provide guidance for the selection of the dimension of the shear box. The results of the numerical evaluation would be compared with the values determined experimentally. Preliminary photographic evaluation of the shape of the sheared samples prepared at different thickness would be conducted to facilitate the selection of sample thickness.

To facilitate the interpretation of the simple shear tests results, moisture induced oedometer collapse, matric suction and direct shear box tests would be carried out on the same soil. The oedometer tests would be conducted in order to determine the influence of the initial soil conditions e.g. initial saturation, water content, void ratio, on the magnitude of collapse. The direct shear box tests would be conducted primarily to establish whether samples of Berea Sand compacted to low density will exhibit bulk volume compression during shear and to compare stress strain behaviour in direct and simple shear apparatus.

The matric suction tests would be conducted on samples tested in the new simple shear device to provide some understanding on magnitude of suction existing in unsaturated sample states and the effect of changes in matric suction on one dimensional and shear induced volume compression.

In order to simulate the stress conditions in the shear zone close to the surface of a vertically loaded pile, series of constant normal stress CNS, normal stiffness NS and constant volume CV simple shear tests would be conducted. The broad range of tests were chosen to accommodate the different suggestions on the actual stress conditions that best represents the stress state in the shear zone close to the surface of a pile vertically loaded to failure.

Semi - empirical and theoretical methods of generating the t – z curves are available. However unlike the semi empirical method, the theoretical method uses soil parameters that can be directly correlated to changes in horizontal stress in the development of the t – z curves. The effect of shear induced compression of the plastic zone on the horizontal stress normal to the pile shaft, load settlement behaviour and pile capacity would be evaluated through the modification of the theoretical t – z method.

CHAPTER THREE

PHYSICAL PROPERTIES OF BEREA RED SANDS

3.1 INTRODUCTION

Compacted samples of the Berea Red Sands formation were used for the series of laboratory tests conducted in this study. The present chapter deals with sampling and sample preparation methods appropriate for tropical soils as well as the determination of particle size, specific gravity and compaction properties of Berea Red Sands. These soil properties facilitated the physical description of Berea Red Sands and provided some guide for the selection of initial state at which subsequent samples used for the study of moisture induced and simple shear induced compression behaviour were prepared.

3.2 SOIL SAMPLING

While saturated clay is generally able to retain significant capillary suction and thus the insitu moisture content after sampling without desaturating, the large structural voids found in insitu deposits of Berea Sands mean that large quantity of pore water cannot be retained in Berea Red Sands after sampling, thus the sampling methods appropriate for clays and sedimentary type soils are not applicable to Berea Sands.

Berea Sands present a structure that is highly brittle and thus sampling and trimming often lead to significant disturbance of the in situ structural state of the sampled soil. In addition weathering of most tropical soil deposits results in significant variation in void ratio and dry density implying that it is impossible to obtain by sampling, undisturbed identical sets of soil samples for series of laboratory tests needed for the study of the mechanical behavior of Berea Sands.

Because of the above reasons only compacted samples of Berea Sands were used in the series of laboratory tests conducted in this study. The major advantage of using compacted samples was that trimming to specified testing sizes was no longer necessary as the samples were compacted inside the respective testing instruments. It is possible to prepare carefully and test sets of samples at selected values of moisture contents and dry densities.

Samples used for the present investigation were taken from a depth of about 2.0m below the surface, in a site within the Howard College campus of the University of Kwazulu Natal. The campus was built on the Berea, a ridge of elevation between 70m to 90m above mean sea level; the immediate landscape is generally undulating, well drained ridge formation (Brink 1984).

3.3 SAMPLE PREPARATION AND COMPACTION

The soil was collected in four large bags, remixed on the floor of the laboratory to ensure uniformity of soil type and source and then sealed up at the existing moisture content and stored in a room in the laboratory where temperature is maintained at temperature of 20 ± 1 °C.

Two of the major oxides found in Berea Red Sands are oxides of aluminum and iron. The waters of hydration in the sesquoxides of iron and aluminum may be driven off by oven drying at 105°C, which is the standard temperature for testing sedimentary type soils (TRRL1988). Similar to meta and tetra halloysites, sesquoxides of iron and aluminum are mainly meta hydrates and tetrahydrates of iron and aluminum. There is no significant difference in shear strength of sands containing different hydrates of the sesquoxides , thus the structural water play no part in the engineering behavior of the soil but is reflected in the test results as the held water is given off at temperature of 50°C. In order to evaluate the susceptibility of the Berea Sands to the effects of re wetting, sensitivity tests were conducted on the bulk samples of Berea Sands.

Two bulk samples of 200 grams each were prepared for moisture content determinations. One sample was oven dried at 105°C until successive weighing show that no further loss in mass is taking place, and the moisture content was then determined. The other sample was oven dried at 50°C until successive weighing indicated no further weight loss and the moisture content then determined. The two results were compared and no significant difference in mass was found, indicating that Berea Sand samples did not contain structural water, which may be due to low concentration of hydrated sesquoxides of iron and aluminum in the soil. (Brewer 1988).

All the four bags were not oven dried at the same time. For the series of different tests conducted, estimated quantities of soil were oven dried at the standard temperature of 105°C and then the samples are used immediately. The above approach prevented the problem of redrying samples that may contain trapped moisture due to long storage after oven drying.

The oven-dried samples were sieved through 2.0 mm sieve and only the particles passing the sieve were used for the tests.

For the laboratory study of the mechanical properties of Berea Sands, the oven dried samples were compacted to the required dry densities at high moisture content and then slowly dried down to the target moisture value at the laboratory controlled temperature of 20°C, the process lasting for up to four days depending on the target moisture content. Barden and Sides (1970) have shown that the structure of a compacted fine soil is greatly influenced by the moulding water content and for fine soils compacted wet and dry of the optimum moisture content the volume change characteristics are different. Barden et al, (1973) and Brewer (1988) related the above behavior to the different fabric alignment of the soil grains. Compacting the soil samples at high moisture content essentially creates soils of similar initial fabric and structure. The effect of slowly drying very wet samples of compacted Berea Sands was investigated in detail in the next chapter.

Several methods of soil compaction have been proposed based on the finding that sample uniformity is strongly dependent on the method of applying the compaction force. The three main compaction methods are dynamic, static and kneading. Investigation by Maswoswe (1985) on Lower Cromer till (a collapsible silty clay soil) revealed that provided the height to diameter ratio is not greater than 1:2, the static method of compaction gives the most uniform specimens, the investigation however revealed that kneading compaction is more representative of field conditions. TRL (1993) however recommended that for tropical soils, static compaction rather than kneading or dynamic compaction ensures that the initial grain constitution is retained and not significantly destroyed by the compaction process.

All the samples tested in this research were statically compacted into the testing apparatus in layers to the desired densities. The reproducibility of the static compaction method was evaluated by testing a set of identically prepared samples in the respective devices.

3.4 PHYSICAL CHARACTERISTICS OF BEREA SANDS

To facilitate proper description of the physical characteristics of Berea Red Sands, series of particle size distribution, specific gravity and soil compaction tests were carried out. Brief descriptions of these tests are given below.

3.4.1 PARTICLE SIZE DISTRIBUTION

Tests to determine the particle size distribution were carried out to also assess the effect of oven drying and extended soaking period in a deflocculant on the shape of the grading curves.

These tests were carried out on fresh samples from a single batch of 4.0 kg mass, thoroughly mixed and then divided into two halves, samples 3PS1 and 3PS2 respectively. Two test procedures were used. The conventional procedure appropriate for residual and sedimentary soils is briefly outlined as follows;

1. Sample 3PS1 was oven dried for 24 hours at temperature of 105 °C, moisture content was determined. The material was then mechanically sieved through the 0.425mm sieve.
2. The proportion retained was then washed through the 0.425 mm sieve, using distilled water and the material retained was oven dried and sieved through a nest of sieve 2.0 mm and 0.425 mm.
3. Any material passing the 0.425 mm sieve was oven dried and added to the soil fines obtained from step 1 and mixed thoroughly.
4. 100 grams of fines and 100ml of standard sodium hexametaphosphate were then added to a standard cylinder, the fines were then deflocculated for four hours using a disperser and the 20 seconds, 40 seconds and 1hr hydrometer test carried out.

The modified procedure, unique for cemented residual soils differs from the standard procedure outlined above. The modified procedure applied to sample 3PS2 is outlined below;

1. Sample 3PS2 was dispersed in sodium hexametaphosphate, solution for 48hrs and then washed through the 0.425 mm sieve.

2 Any material passing the 0.425mm sieve was slowly dried by evaporation at the controlled temperature. This aspect took four days to dry the material down to 20 % moisture content.

3 Estimated dry mass of 100 grams is further dispersed in the distilled water containing 100 ml of standard sodium hexametaphosphate solution for two days and then tested using the hydrometer.

In order to uncouple the effects due to oven heating from that due the defloculant, preliminary hydrometer tests were run on fresh samples of soil from the sealed bag without oven drying the sample. The moisture content of the soil from the sealed bag is 9.50 % and three sets of 100 grams of fractions passing the 0.425 mm sieve was used for hydrometer test. Also three sets of 100grams of oven dried fines was also tested. The hydrometer values for the fresh samples range from 4.0 % to 6.0 %. While values of 3.0 % to 5.0 % was obtained from a series of hydrometer tests on oven dried samples.

The particle grading curves of the average of three set of tests conducted on the untreated and deflocculated samples are shown in Fig 3.1

The results on grain size tests conducted on untreated and deflocculated samples shown in Fig 3.1 clearly indicate that Berea Red Sands are fine clayey sands. Although the effect of the oxalate resulted in marginal increase in the fines fraction, the bulk of this change occurred in the clay fractions. The silt fraction remains unchanged despite rigorous agitation of the oxalate solution.

Also shown is the grading envelope for Berea Sands from field investigation reported by Brink (1984). The grading envelope presented by Brink (1984) show the upper and lower particle size envelope for Berea Sands sampled from different sites in Kwa Zulu natal province. Brink (1984) related the high fine fractions of Berea Sands found in some deposits to high degree of weathering. The grading curves plot slightly above the lower boundary of the grading envelope reported by Brink (1984) implying that the Berea Red Soil used for the above tests have not undergone considerable weathering.

Although some researchers (Townsend 1985; Toll 1988) have reported some form of transition from one soil type to the other for residual soils due to oven drying and application of diffloculant, it took the combined effect of non oven drying and five days of physical and chemical dispersion to produce the change in particle sizes indicated in Fig 3.1. The marginal change in particle size is influenced by the small in situ clay fraction associated with early stage of weathering. It is thus decided that tests for the study of the mechanical properties of Berea Red Sands should be carried out on oven-dried samples.

3.4.2 PARTICLE SPECIFIC GRAVITY

Specific gravity tests following BS 1377 (1986), were carried out on sets of 50 grams and 400 grams mass of samples passing the 0.25 mm, 0.425 mm and 2.0 mm sieves. The aim of the above series of tests is to assess the effect of variation in mass of test sample and particle size on the specific gravity. The samples mixed with distilled water in a density-bottle were gently boiled and de aired in a vacuum desiccator using a 100 kPa suction pump. The results are presented in Table 3.1

The Data shown in Table 3.1 indicate that more than half of the tests conducted with 50 grams samples and with 400 grams samples gave specific gravity values between 2.70 and 2.71. With respect to variation in particle size, a slight increase in specific gravity with decrease in particle size was indicated for tests conducted with 50 grams samples and 400 grams samples. The specific gravity values are also consistent within the same particle size. The specific gravity of Berea Red Sand can be represented by the specific gravity of samples passing the 2mm sieve and the value may be taken as 2.70 by considering the results of tests on 50 grams and 400 grams samples.

The specific gravity of quartz is 2.65, Kaolinte is 2.61 and that of Hematite is between 4.90 and 5.30. Berea red sand is predominantly fine quartz (silicon dioxide) coated with iron oxide and so the measured values of 2.70 is due to the contribution from the iron oxide. For a soil consisting predominantly of fine quartz sand fractions, it is expected that the effect of specific gravity of iron oxide on the specific gravity of iron oxide coated quartz sand will be significant, leading to specific gravity values higher than that of quartz.

However data showing increase in specific gravity with increasing particle size has been reported for tropical red soils by Toll (1988), reflecting a higher iron oxide – quartz ratio in the larger particle size under the same weathering condition. It must however be noted that Toll (1988) used the BS 1377: 1986 guidelines of 10 grams dry samples in 50ml pycnometer for the determination of specific gravity.

3.4.3 COMPACTION

To prevent excessive modification of the initial particle size distribution, low energy compaction effort i.e. Standard Proctor compaction test was carried out. This compaction effort entails the application of 55 blows per layer on three layers by a 2.495 kg tamper dropping from a nominal height of 304.8 mm to the soil in a standard compaction mould of 152 mm diameter and 115 mm high.

Each point on the compaction curve was established with a fresh sample after which the sample is discarded.

The Proctor compaction tests provide a guide on dry densities and moisture content ranges at which the soil may exhibit significant collapse settlement under applied stress. The compaction curves are presented in Fig 3.2.

Data obtained from the tests and shown in Fig 3.2 indicate average optimum moisture content of 9.2 % at the average maximum dry density of 1785 kg/m^3. The compaction curves show marginal variation in the dry density at low moisture content and this may be due to differences in granular friction resulting from differences in degree of lubrication at low moisture content

At low moisture content, high inter particle surface friction results in high shear strength and a low dry density. Additional water content increases surface lubrication thereby increasing density. Soil strength is also increased by the formation of water menisci leading to increase in effective particle contact area. At peak point, the balance between lubrication and the formation of water menisci is reached. Additional water will result in more menisci causing an increase in effective stress and a decrease in dry density (Maswoswe 1985).

The OMC and MDD value of 9.20 % and 1785 kg/m^3 respectively are marginally higher than values reported by Maud (1968) and Brink (1984).

As it is well known that samples compacted dry of the optimum moisture content tend to exhibit moisture induced collapse, the compaction curves of Fig 3.2 provided a guide for the selection of the initial states (moisture contents and dry densities) for the preparation of soil samples used for the study of moisture induced and shear induced compression behavior. All the samples used for the study of the mechanical properties of Berea Red Sand were prepared at dry densities below 1785 kg/m^3.

In summary, the results on grain size tests conducted on untreated and deflocculated samples shown in Fig 3.1 clearly indicate that Berea Red Sands are fine clayey sands. Although the effect of the oxalate resulted in marginal increase in the fines fraction, the bulk of this change occurred in the clay fractions. Oven drying at 50°C and at 105°C indicate that Berea Sand samples did not contain structural water, which may be due to low concentration of hydrated sesquoxides of iron and aluminuim in the soil.

The grading curves plot slightly above the lower boundary of the grading envelope reported by Brink (1984) implying that the Berea Red Soil used for the above tests have not undergone considerable weathering. The marginal change in particle size is influenced by the small in situ clay fraction associated with early stage of weathering. It is thus decided that tests for the study of the mechanical properties of Berea Red Sands should be carried out on oven-dried samples.

The specific gravity of Berea Red Sand can be represented by the specific gravity of samples passing the 2mm sieve and the value may be taken as 2.70 by considering the results of tests on 50 grams and 400 grams samples.

The specific gravity of quartz is 2.65, Kaolinte is 2.61 and that of Hematite is between 4.90 and 5.30. Berea red sand is predominantly fine quartz (silicon dioxide) coated with iron oxide and so the measured values of 2.70 is due to the contribution from the iron oxide.

The OMC and MDD value of 9.20 % and 1785 kg/m^3 respectively are marginally higher than values reported by Maud (1968) and Brink (1984).

TABLE 3.1; SPECIFIC GRAVITY VALUES OF BEREA RED SANDS

Samples passing sieve size (mm)	Mass of dry sample (grams)	Specific Gravity (G_s)
2.0	50	2.708
2.0	50	2.679
0.425	50	2.737
0.425	50	2.708
0.25	50	2.783
0.25	50	2.708
2.0	400	2.704
2.0	400	2.708
0.425	400	2.700
0.425	400	2.706
0.25	400	2.783
0.25	400	2.779

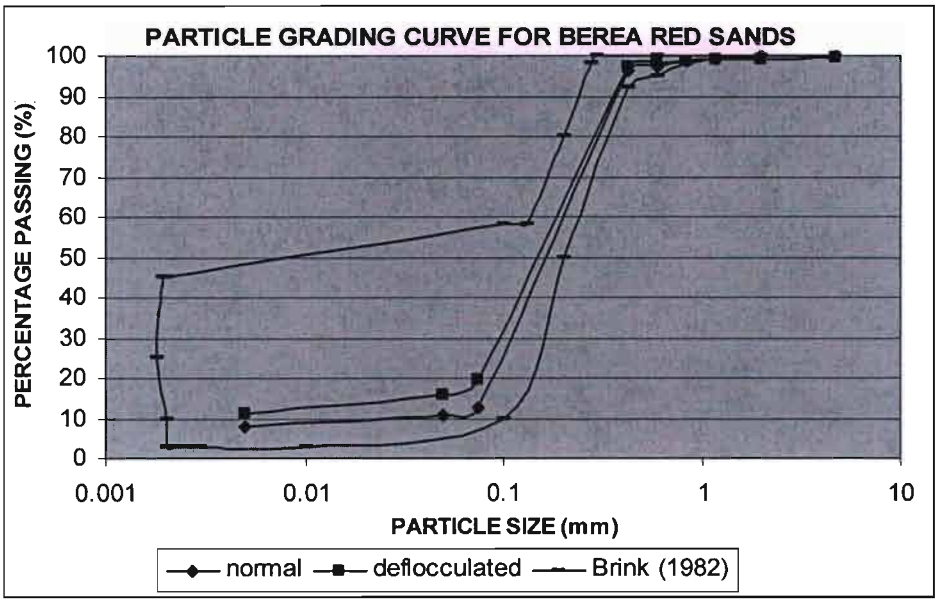


FIG 3.1, PARTICLE GRADING CURVE FOR BEREA RED SANDS.

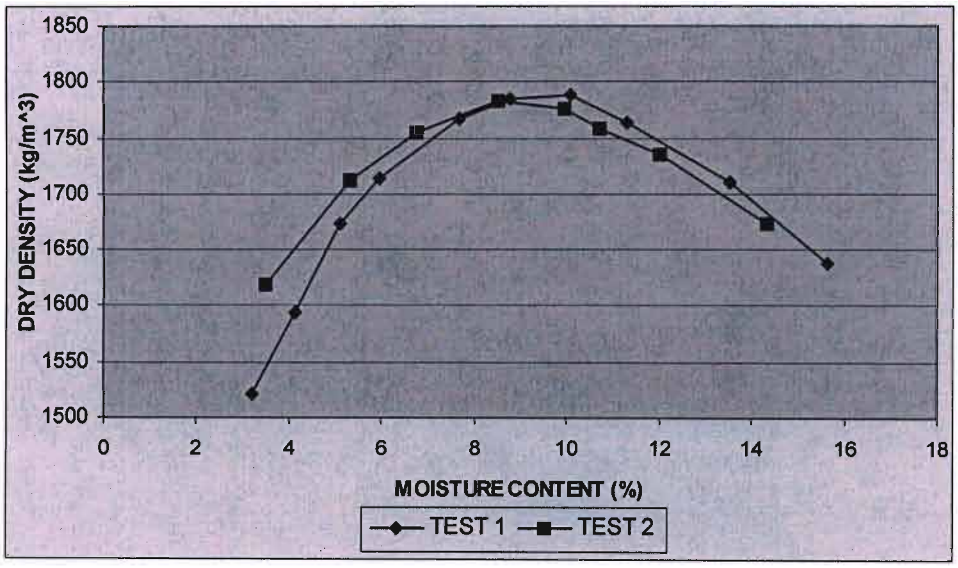


FIG 3.2, STANDARD PROCTOR COMPACTION CURVE FOR BEREA RED SANDS.

CHAPTER FOUR

MOISTURE INDUCED VOLUME CHANGE BEHAVIOR OF BEREA SANDS

4.1 INTRODUCTION

Laboratory study of the volume change behavior of compacted Berea Red Sands in one dimensional compression is necessary to ascertain whether significant volume change can be expected when compacted Berea Red Sands samples are compressed in one dimension, and also to ascertain the effect of compaction moisture content and dry density on magnitude of compression and collapse settlement.

Detailed laboratory study of the compression and moisture induced collapse behavior of Berea Red Sands are presented in this chapter.

There are two widely used laboratory methods of determining the collapse settlement of soils.

The first is the double oedometer method in which one of two identical samples is saturated before subjecting both samples to series of applied stresses. Moisture induced collapse settlement of Berea Red Sands was studied by the second method. This method entails increasing the stress on a sample up to a specified value followed by inundation at the specified stress. The advantage of this method of determining collapse settlement is that field compression stress paths can be directly simulated.

4.2 TEST CONDITIONS AND PROCEDURE.

The samples used for these tests were statically compacted into standard oedometer rings of 19mm height and 76mm diameter at initial moisture content of 20%, placed in the oedometer pots between air dried porous stones. The inner surface of the oedometer rings was lubricated with the lubricant Q10 before sample compaction to reduce the effect of side shear in the sample during compression.

To avoid moisture loss from the samples, no filter papers were used and the samples were sealed with plastic bags, which were attached to the oedometer pots with adhesive tape. For any given applied stress, 24 hrs were allowed for the consolidation of all samples used for the compression and moisture induced collapse settlement tests. Depending on the test, at a given applied stress, collapse was induced by gradually filling the oedometer pots with distilled water, after removing the plastic bag.

The sample was then allowed to stand for a further 24 hrs after which time the deflection was noted. Depending on the stage of test, either additional stress increments would be applied or the sample would be unloaded in stages. For all tests, samples were loaded in stages by doubling the magnitude of stress applied from stage to stage till the desired stress was reached. Preliminary tests to examine the effect of rate of increase in the magnitude of stress applied on the shape of the compression curves were investigated using two loading conditions, one as described above, where the magnitude of applied stress is double the previous load, and a second loading condition in which the increase in the magnitude of applied stress was kept constant. The result indicated no change in the shape of the compression curves as a result of change in the rate of loading.

Twelve oedometer loading units are available in the civil engineering laboratory, thus a set of up to twelve samples can be tested over the same period. The variables are void ratio, degree of saturation and the applied stresses during soaking. Changes in sample void ratio corresponding to full compression for each load increment was obtained by back calculation from the final thickness reading. The moisture content values of interest in the series of compression tests were the values determined after the end of the unloading phase, while the moisture content values ascribed to the collapse settlement tests are the values estimated from the bulk mass of the samples before the initial compression loading.

A preliminary oedometer test was conducted on samples prepared at 20 % moisture content and 1300 kg/m^3 dry density. It was intended to dry down the samples to selected moisture content, but at the initial states of 1300 kg/m^3 and 20 % moisture content, it was practically impossible to compact the samples as they were essentially slurry with heights below 19mm, thus the soil samples used for settlement tests were prepared at the initial state of dry density of 1400 kg/m^3 - 1430 kg/m^3 and moulding water content of 20 %.

The phenomenon of moisture-induced collapse was investigated at moisture content range consistent with in-situ conditions applicable to Berea Red Sands Formation. As the range fall within 6 % and 10 %, all the samples were tested at this range of moisture content. At a given vertical stress, the samples were soaked by gradually filling the oedometer pots with distilled water such that the air in the soil voids are displaced by the rising water and the amount of collapse noted.

In this research, the amount of collapse is referred to as collapse settlement and is defined as the ratio of the change in height ΔH, of a sample after inundation, to the initial height H_o, expressed in percent. The collapse settlement of a sample at any applied stress expressed in percentage was given by Jennings and Knight (1957) as

$$\frac{\Delta e}{1+e_o} \times 100\,\% \qquad 4.1$$

where Δe is the change in void ratios on saturation and e_o is the initial void ratio of the sample.

4.3 RESULTS

The results of series of tests conducted to investigate compression and moisture induced collapse settlement behaviour of Berea Red Sands compacted at different compaction moisture content to different dry densities and tested under different normal stress were presented in tables 4.1 – 4.9.

Test series 4.1

A set of samples were compacted into the oedometer ring at moulding water content of 3.44% - 14.37% to dry densities of 1403 kg/m^3 – 1437 kg/m^3 and subjected to different effective normal stresses within the range of 10 kPa – 1200 kPa and subsequently unloaded to 10 kPa. The compression curves for five different normal stresses are shown in Table 4.1 and Fig 4.1.

All the five samples tested, show increasing compressibility with increase in moisture content.

Upon unloading, the samples did not indicate any change in void ratio and the unloading curves are straight lines parallel to the effective normal stress axis. The general trend observed from Fig. 4.1 is that the compressibility of Berea Red Sands compacted to the same dry densities increases with increase in degree of saturation.

Test series 4.2

All the samples tested so far were compacted at selected values of moisture content to some desired range of dry densities. The reproducibility and degree of accuracy of the method of testing was evaluated by assessing the collapse settlement of Berea Red samples compacted at moisture content range of 14.83 % - 17.04 % to dry densities of 1424 kg/m^3 - 1476 kg/m^3 and then gradually dried down to and tested at moisture content range of 4.01 % - 5.26 %. All the samples were compressed in stages to normal stress of 200 kPa and then soaked.

The results of nine samples shown in Table 4.2 indicate a mean collapse settlement of 11.31% with variation coefficient of ± 1.13 or ± 10 %. The low coefficient implies that collapse settlement behavior from a given set of samples initially compacted at high moulding moisture content and tested in the oedometer are repeatable to the accuracy of ± 10 % and significant collapse can be expected from samples compacted to dry density range of 1424 kg/m^3 - 1476 kg/m^3 and soaked at 200 kPa.

Test series 4.3 and 4.4

The effect of compacting samples at moisture content higher than the values at which the samples were loaded on the compressibility and collapse settlement was examined below.

By drying down a sample from a high moisture content Sasitharan et al, (1993) noted that the evaporating fluids would leave drainage tracks and the sample would be more likely to wet up uniformly upon soaking without forming localized liquefied spots within sample of essentially similar fabric. Fabric is the physical constitution of the soil as expressed by the spatial arrangement of the solid particles and associated void (Brewer 1968), and soil fabric orientation may be non-existent, weak, moderate or strong, depending on the percentage of the particles that are orientated with their principal axes within 30 degrees of each other. Barden (1974) examined the fabric of soils compacted at the OMC and also at moisture content wet and dry of the OMC and reported that the samples compacted wet of the OMC indicated a strong orientation. If such a sample is dried down to moisture content below OMC, loaded and inundated, significant collapse is expected due to the loss of the strong fabric orientation.

Individual particles of fine soils tend to attract one another edgewise as they settle in suspension. This results in the formation of a loose and porous assemblage of honey combed soil particles Oloo (1994). Honey combed structures can carry large loads due to the arching effects of the honey comb, but are also subject to sudden collapse when loading is not axis-symmetric (Oloo 1994).

It is conceivable that the above behavior may be exhibited by low density Berea Sand, with about 12% clay and silt fractions.

The above hypothesis was tested using two sets of samples. The two sets of samples were subjected to the same magnitude of applied stresses.

In one set, the samples were compacted to dry densities of 1424 kg/m^3 – 1436 kg/m^3 at moisture content of 14.95 % - 16.61% and slowly dried down to moisture content of 4.40 % - 4.88 % at laboratory temperature of 20 °C. In the second set the samples were compacted to dry densities of 1424 kg/m^3 – 1435 kg/m^3 at moisture contents of moisture content range of 4.38 % - 4.88 %.

The compression curves of the two sets of tests are shown in Fig. 4.2 and Fig. 4.3. The compression curves are in terms of externally applied stress because the positive effective stress component due to suction is unknown. Although the effective stress conditions in the unsaturated samples are indeterminate, it can definitely be said that, at the specific void ratio, the effective stress in a sample is greater than the applied stress acting on the sample. It can be seen from Fig. 4.2 and Fig. 4.3 that the void ratio of the unsaturated samples is always greater than the void ratio of the saturated sample at any applied stress.

This is clearly against the principle of effective stress. If the unsaturated samples of Berea Sands had obeyed the principle of effective stress, their void ratio should have been less than the void ratio of the saturated samples at any specific applied stress (Blight 1967).

The test results plotted in Fig. 4.4 indicate significant differences in collapse settlement behavior due to method of sample preparation.

In situ state of tropical soil has evolved through cycles of infiltration and evaporation. Drying down the samples that were initially compacted at high moisture content was intended to recreate the in-situ particulate structure with strong fabric orientation and a matrix of air and pore water channels consistent with in-situ conditions.

It is interesting to note however that once collapse had occurred, the samples in Fig. 4.3 and Fig. 4.4 follow more or less the same consolidation curves when loading is resumed. This would seem to suggest that regardless of initial moisture content once the samples were saturated they follow the same virgin consolidation curve. Allowing for any experimental error in determining the initial void ratio of the samples, they all return to a unique saturated compression curve on soaking. The behavior appears to be independent of the value of the applied load at which soaking was initiated. The rebound curves however are not similar for all two set of curve and appear to be dependent on the pressure at which soaking was initiated. The higher this pressure is the less the rebound.

Test Series 4.5 – 4.9

Based on the above findings, the effect of changes in initial saturation and compaction water content on magnitude of collapse was investigated. Here sets of six identical samples were soaked under applied stresses of 50 kPa, 100 kPa, 200 kPa, 400 kPa, 800 kPa and 1200 kPa.

Fig. 4.5 shows the collapse settlement curves of samples of Berea Red Sands compacted into the oedometer rings at average moisture content of 15.17 % - 17.61 % to dry densities of 1407 kg/m^3 – 1427 kg/ m^3 and tested at moisture content range of 4.42 % - 4.66 %.

Fig. 4.6 shows the collapse settlement curves test of samples of Berea Sands compacted into the oedometer rings at average moisture content of 14.17 % - 16.13 % to dry densities of 1410 kg/m^3 – 1434 kg/m^3 and tested at moisture content range of 5.95 % - 6.22 %.

Fig. 4.7 shows the collapse settlement curves of samples of Berea Red Sands compacted into the oedometer rings at average moisture content of 14.21 % - 15.89% and tested at moisture content range of 6.05 % - 6.44 % and dry densities of 1.717 Mg/m^3– 1.733 Mg/m^3.

Fig. 4.8 shows the collapse settlement curves of samples of Berea Red Sands compacted into the oedometer rings at average moisture content of 14.21% - 15.89% and tested at moisture content range of 8.40% - 8.80% and dry densities of 1.401 Mg/m^3 – 1.434 Mg/m^3

Fig. 4.9 shows the collapse settlement curves of samples of Berea Sands compacted into the oedometer rings at average moisture content of 14.21 % - 15.89 % and tested at moisture content range of 8.06% - 9.02% and dry densities of 1.716 Mg/m^3 – 1.729 Mg/m^3

To facilitate interpretation of the collapse behavior, Fig. 4.5 to Fig. 4.9 were replotted as curves of percentage collapse settlement against effective normal stress. These curves are shown in Fig. 4.10 to Fig. 4.12.

It can be seen from Fig. 4.5 to Fig. 4.9 that for the same compaction water content and soaking stress, the higher the initial void ratio, i.e. a lower initial saturation, the greater the amount of collapse. After collapse, all the samples again appear to follow a common saturated compression curve. Also the rebound curves of all the samples did not indicate any change in void ratio during the unloading stages. The constant void ratio indicated throughout the rebound stages is the same in magnitude as the final void ratio of the samples under the maximum applied stresses.

It can be seen from Fig. 4.11 – Fig. 4.13 that the samples with an initial saturation less than 20% seem to have a soaking stress at which percentage collapse is a maximum and the lower the initial saturation the higher is the soaking stress at which maximum collapse occurs. The above observation is true for all samples tested at initial moisture content between 4.48% and 6.78%.

For the stress range considered, the general trend indicates that the amount of collapse increases with soaking stress, however, as the applied stress increases even more, the soil void will be reduced to such an extent that the amount of collapse would start to decrease. For samples tested at average moisture content of 8.45% and shown in Fig. 4.13, for the stress range considered, the general trend indicates that the amount of collapse decreases with increase in soaking stress.

It has already been shown in Fig. 4.11 that the compressibility of compacted Berea Red Sands increase with increase in initial moisture content due to increased lubrication of the surfaces of the sand grains resulting in closer packing of sand grains subjected to external stress.

The samples compressed at average moisture content of 4.42 % to 6.78 % did not indicate significant volume change i.e. limited compressibility at low values of applied stress, which must be due to low degree of lubrication and thus the increase in collapse settlement indicated subsequently must be due to increase in applied stress for the range of applied stresses up to 400 kPa. Beyond 400 kPa, the significantly reduced void ratio results in reduced collapse settlement upon inundation.

The samples compressed at average moisture content of 8.45% shown in Fig 4.12 exhibited significant volume compression from the beginning of the loading stage due to increased lubrication of soil granular surfaces. The compressibility increases with increase in applied stress, thus the collapse settlement decreases with increase in applied stress.

Fig. 4.10 to Fig. 4.12 was used to plot constant soaking stress curves of percentage collapse against initial saturation. These curves shown in Fig. 4.13 indicate that for the same soaking stress, the lower the initial saturation, the greater the collapse. These curves also clearly indicate that for samples with initial saturation lower than 20 %, there is a tendency for the amount of collapse, at a specific initial saturation, to decrease with increasing soaking stress. This is again due to the higher applied stress reducing the collapse potential of an initially very loosely compacted sample by reducing the available void.

The general pattern emerging from Fig. 4.13 is that a combination of low initial saturation of 13 % and applied stress of 400 kPa result in the greatest collapse. For samples soaked at an applied stress less than 400 kPa, they only show significant collapse if the initial saturation is less than 26%. The general trend is that in order to be sure of avoiding collapse, at any stress range, the soil most be compacted at a degree of saturation higher than 26%.

For the saturation range considered, the greatest amount of collapse for samples compacted at 8.44 % water content, occurs at initial saturation of 23.1% and a vertical soaking stress of 50 kPa, whereas for the 6.8 % moisture content samples, the greatest amount of collapse occur at an initial saturation of 17.2 % but at a higher soaking stress of 200 kPa. For samples compacted at

4.2 % moisture content, the maximum collapse occurs at initial saturation of 10.3 % and at a much higher soaking stress of 400 kPa.

The above trend demonstrates the effect of sample compressibility on collapse settlement of loosely compacted samples of Berea Red Sands. The general trend is that the more compressible the samples, the lower the magnitude of collapse settlement and the soaking stress at which the maximum collapse settlement occur.

The presence of light cementation at the particle contact points of compacted residual sand may contribute to high void ratios as well as increased capacity for large volume changes depending on applied stress and initial saturation, provided the cementation at the particle contact point can be destroyed by the combination of inundation and applied stress. For particles compacted and dried from initially high moisture content, the strength of the bond at the particle contact points generally increases with decrease in moisture content at which samples are tested. For Berea Red Sands, the strength of the bond due to clay and silt bridges at the contact point increases with decrease in moisture content, thus the samples are less compressible at lower moisture content, but exhibit significant collapse settlement because of the combination of low fine content, low dry density and thus high void ratio.

The degree of cementation affects collapse settlement response. Well-cemented sands, i.e. laterites have lower void ratio as the pore spaces have become predominantly filled with significant amount of clay particles, so collapse settlement after soaking results in small volumetric strains. However unlike laterites, the percent fines, mineralogy of the fines and the degree of coating of the quartz grains in Berea Red Sands favors significant collapse settlement response when inundated at low water content.

4.4 SUMMARY

Moisture induced collapse settlement of compacted Berea Red Sands is dependent on the following three major conditions; partially saturated soil voids, sufficiently large suction and a high enough total stress so that the structure is metastable.

For any given set of condition, the amount of collapse of compacted Berea Red Sands generally decreases with increasing precollapse moisture content, increasing precollapse dry density and decreasing applied stress.

For compacted Berea Red Sands there are combinations of initial dry density, moulding water content and overburden pressure at which insignificant volume change and at which maximum collapse will occur when the soil is inundated. Samples compacted above dry density of 1716 kg/m^3 at average molding water content of 7.8 % will exhibit minimal collapse settlement irrespective of soaking pressure while samples compacted at dry density of 1407 kg/m^3 at average molding water content of 4.2 % will exhibit maximum collapse settlement of 12 % - 14 %. The maximum collapse settlement occurs at soaking stress of 400 kPa.

For Berea Sands there appear to be critical molding water content at soaking above which minimum collapse settlement will occur. The average value is found to be 7.8 %.

There is a critical degree of saturation above which negligible collapse will occur regardless of the magnitude of the pre wetting soaking stress. For Berea Sands this value is 30 %.

Having establish the range of compaction moisture contents and dry densities within which significant moisture induced collapse settlement occur in compacted Berea Red Sands, it is now necessary to investigate whether significant volume compression might occur in Berea Red Sands compacted to low density and subjected to direct shear, and in addition to establish the effect of moisture content on direct shear induced volume compression.

TABLE 4.1

TEST SERIES 4.1: Compression tests on samples of Berea Red Sands compacted and tested at moisture content range of 3.44 % - 15.92 %.

test no	soil moisture content after test (%)	dry density before test (Mg/m^3)	dry density after test (Mg/m^3)	void ratio after test (e)	void ratio before test (e)	initial degree of saturation (%)
4.11	3.44	1.416	1.514	0.7834	0.9074	8.43
4.12	7.50	1.428	1.554	0.7370	0.8910	18.05
4.13	9.93	1.404	1.578	0.7116	0.9232	24.76
4.14	12.52	1.424	1.614	0.6730	0.8964	30.30
4.15	15.92	1.438	1.652	0.6339	0.8779	37.74

TABLE 4.2

TEST SERIES 4.2: Collapse settlement test of samples compacted into the oedometer rings at moisture content range of 14.83% - 17.04% and tested at 4.01% - 5.26%

test no	soaking stress (kPa)	moisture content after compaction (%)	moisture content before test (%)	dry density before test (Mg/m^3)	collapse settlement (%)
4.21	200	17.04	4.72	1.476	10.10
4.22	200	15.58	4.26	1.436	11.90
4.23	200	16.23	4.78	1.424	11.43
4.24	200	15.05	5.13	1.457	10.98
4.25	200	15.02	4.72	1.436	10.10
4.26	200	15.37	5.26	1.436	11.90
4.27	200	17.33	4.78	1.424	12.03
4.28	200	15.98	4.13	1.427	10.98
4.29	200	15.03	4.01	1.472	12.45

TABLE 4.3

TEST SERIES 4.3: Collapse settlement test of samples of Berea Sands compacted into the oedometer rings at average moisture content of 14.95% - 16.61% and tested at moisture content range of 4.40% - 4.88%.

test no	soaking stress (kPa)	moisture content after compaction (%)	moisture content before test (%)	dry density before test (Mg/m^3)	dry density after test (Mg/m^3)	void ratio after test (e)	void ratio before test (e)	collapse settlement (%)	degree of saturation (%)
4.31	100	17.54	5.72	1.436	1.761	0.5331	0.8808	10.10	13.60
4.32	200	15.58	5.26	1.436	1.732	0.5588	0.8799	11.90	12.50
4.33	400	16.23	5.78	1.424	1.766	0.5289	0.8961	12.43	13.99
4.34	800	14.95	5.13	1.427	1.722	0.5683	0.8915	10.98	12.34

TABLE 4.4

TEST SERIES 4.4: Collapse settlement tests of samples of Berea Sands compacted into the oedometer rings and tested at moisture content range of 4.38% - 4.88%

test no	soaking stress (kPa)	soil moisture content before test (%)	dry density before test (Mg/m^3)	dry density after test (Mg/m^3)	void ratio after test (e)	void ratio before test (e)	collapse settlement (%)	degree of saturation (%)
4.41	100	4.51	1.435	1.778	0.5187	0.8809	9.69	13.82
4.42	200	4.45	1.435	1.767	0.5280	0.8809	10.51	13.65
4.43	400	4.38	1.424	1.751	0.5424	0.8958	11.68	13.21
4.44	800	4.88	1.425	1.727	0.5630	0.8951	10.32	14.73

TABLE 4.5;

TEST SERIES 4.5: Collapse settlement test of samples of Berea Sands compacted into the oedometer rings at average moisture content of 15.17% - 17.61% to dry densities of 1407 – 1427 kg/m3 and tested at moisture content range of 4.42% - 4.66%.

test no	soaking stress (kPa)	moisture content after compaction (%)	moisture content before test (%)	dry density before test (Mg/m^3)	dry density after test (Mg/m^3)	void ratio after test (e)	void ratio before test (e)	collapse settlement (%)	degree of saturation (%)
4.51	50	17.16	4.57	1.407	1.749	0.5435	0.9185	9.46	13.44
4.52	100	15.32	4.45	1.429	1.745	0.5469	0.8900	10.86	13.49
4.53	200	17.61	4.66	1.403	1.716	0.5737	0.9244	11.54	13.62
4.54	400	17.27	4.34	1.419	1.714	0.5750	0.9028	12.79	12.99
4.55	800	15.17	4.42	1.429	1.708	0.5809	0.8890	10.14	13.41
4.56	1200	15.18	4.64	1.427	1.748	0.5443	0.8924	9.46	14.05

TABLE 4.6:

TEST SERIES 4.6: Collapse settlement test of samples of Berea Sands compacted into the oedometer rings at average moisture content of 14.17% - 16.13% to dry densities of 1410 kg/m^3 – 1434 kg/m^3 and tested at moisture content range of 5.95% - 6.22%.

test no	soaking stress (kPa)	moisture content after compaction (%)	moisture content before test (%)	dry density before test (Mg/m^3)	dry density after test (Mg/m^3)	void ratio after test (e)	void ratio before test (e)	collapse settlement (%)	degree of saturatioi (%)
4.61	50	16.13	5.95	1.410	1.710	0.5788	0.9155	8.01	17.54
4.62	100	14.63	6.06	1.421	1.741	0.5509	0.9000	8.83	18.18
4.63	200	14.12	6.27	1.420	1.721	0.5691	0.9018	10.11	18.76
4.64	400	15.25	6.22	1.420	1.725	0.5649	0.9008	10.00	18.63
4.65	800	14.17	6.12	1.429	1.738	0.5532	0.8894	6.01	18.58
4.66	1200	14.37	6.04	1.434	1.707	0.5816	0.8826	5.11	18.49

TABLE 4.7

TEST SERIES 4.7: Collapse settlement test of samples of Berea Sands compacted into the oedometer rings at average moisture content of 14.21% - 15.89% and tested at moisture content range of 6.05% - 6.44% and dry densities of 1.717 Mg/m^3– 1.733 Mg/m^3

test no	soaking stress (kPa)	moisture content after compaction (%)	moisture content before test (%)	dry density before test (Mg/m^3)	dry density after test (Mg/m^3)	void ratio after test (e)	void ratio before test (e)	collapse settlement (%)	degree of saturation (%)
4.71	50	14.66	6.44	1.726	1.885	0.4324	0.5647	2.14	30.80
4.72	100	15.89	6.44	1.734	1.903	0.4186	0.5575	2.14	31.17
4.73	200	14.21	6.33	1.733	1.924	0.4033	0.5580	2.69	30.61
4.74	400	15.38	6.13	1.717	1.901	0.4199	0.5721	3.59	28.92
4.75	800	15.78	6.10	1.731	1.863	0.4493	0.5601	2.74	29.39
4.76	1200	15.10	6.05	1.735	1.845	0.4633	0.5560	2.52	29.36

TABLE 4.8

TEST SERIES 4.8: Collapse settlement test of samples of Berea Sands compacted into the oedometer rings at average moisture content of 14.21% - 15.89% and tested at moisture content range of 8.40% - 8.80% and dry densities of 1.401 Mg/m^3– 1.434 Mg/m^3

test no	soaking stress (kPa)	moisture content after compaction (%)	moisture content before test (%)	dry density before test (Mg/m^3)	dry density after test (Mg/m^3)	void ratio after test (e)	void ratio before test (e)	collapse settlement (%)	degree of saturation (%)
4.81	50	16.96	8.76	1.401	1.677	0.6098	0.9276	5.37	25.51
4.82	100	16.05	8.84	1.426	1.681	0.6059	0.8932	3.52	26.71
4.83	200	17.12	8.62	1.422	1.685	0.6021	0.8985	3.14	25.91
4.84	400	16.01	8.81	1.423	1.691	0.5971	0.8977	2.21	26.51
4.85	800	14.54	8.40	1.428	1.716	0.5732	0.8907	2.72	25.47
4.86	1200	14.72	8.80	1.434	1.673	0.6137	0.8833	1.95	26.89

TABLE 4.9

TEST SERIES 4.9: Collapse settlement test of samples of Berea Sands compacted into the oedometer rings at average moisture content of 14.21% - 15.89% and tested at moisture content range of 8.06% - 9.02% and dry densities of 1.716 Mg/m^3– 1.729 Mg/m^3

test no	soaking stress (kPa)	moisture content after compaction (%)	moisture content before test (%)	dry density before test (Mg/m^3)	dry density after test (Mg/m^3)	void ratio after test (e)	void ratio before test (e)	collapse settlement (%)	degree of saturatio (%)
4.91	50	14.88	8.46	1.716	1.828	0.4768	0.5735	2.13	39.82
4.92	100	15.97	8.47	1.714	1.828	0.4771	0.5753	2.11	39.74
4.93	200	13.91	9.02	1.726	1.879	0.4373	0.5647	2.68	43.11
4.94	400	15.58	8.85	1.706	1.870	0.4435	0.583	3.56	40.98
4.95	800	15.71	8.78	1.728	1.861	0.4508	0.5626	2.74	42.15
4.96	1200	14.50	8.06	1.729	1.840	0.4675	0.5616	2.51	38.74

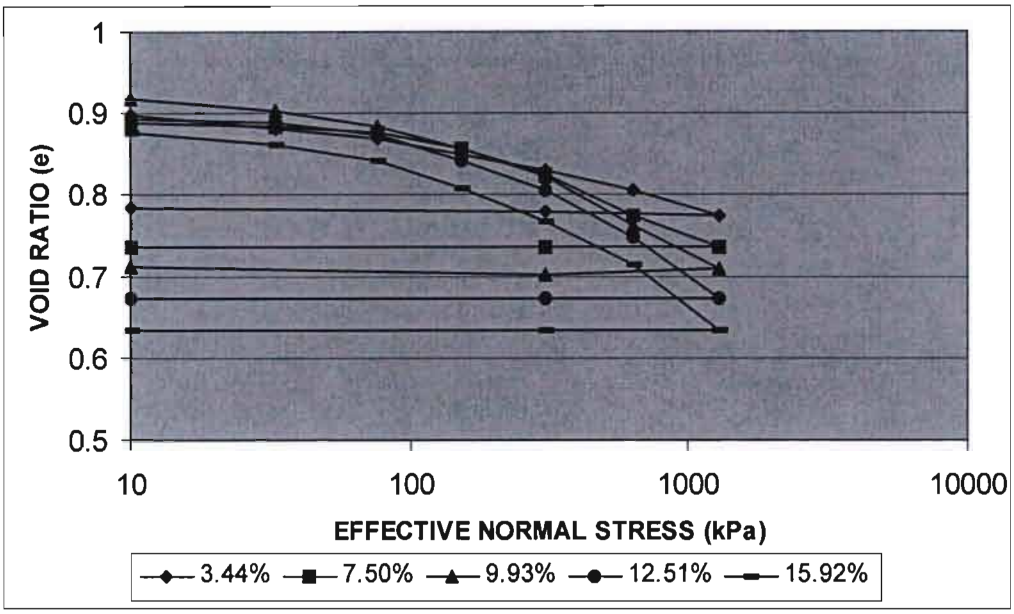


Fig. 4.1: Compression curves of Berea Red Sands samples compacted to dry densities of 1404 kg/m^3 – 1438 kg/m^3 at moisture content range of 3.44% - 15.92%.

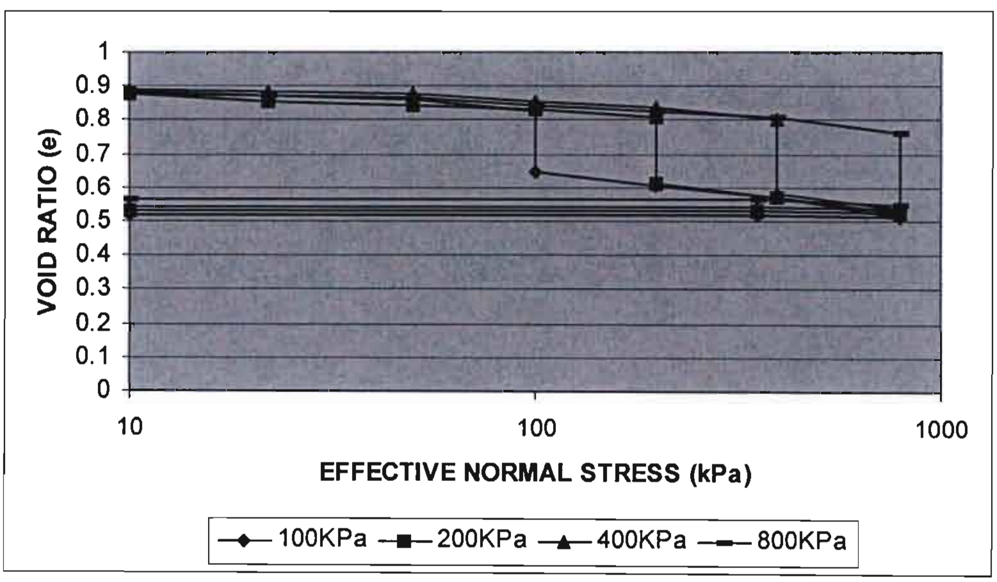


Fig. 4.2: Collapse settlement curves of Berea Sands compacted into the oedometer rings at average moisture content of 14.95% - 16.61% and tested at moisture content range of 4.40 % - 4.88 %.

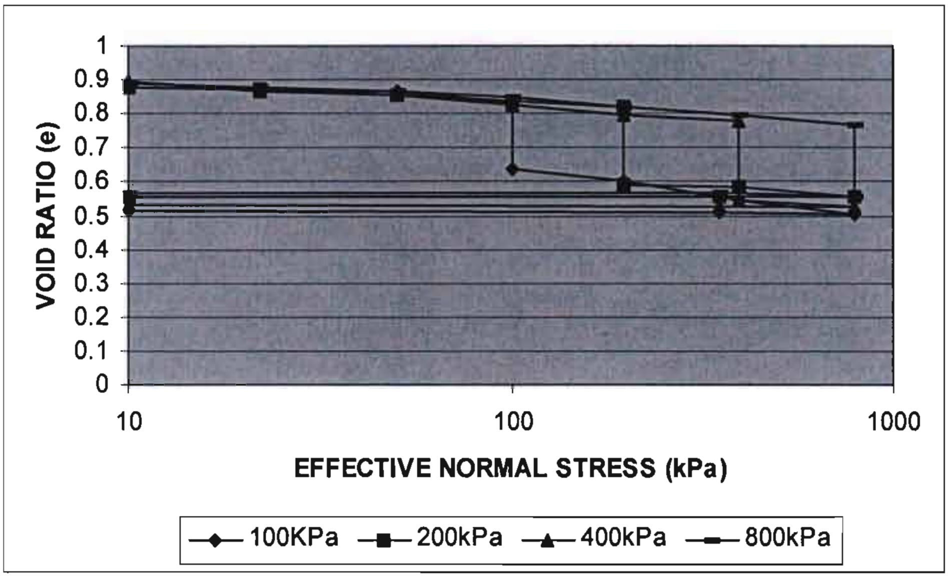


Fig. 4.3: Collapse settlement tests of samples of Berea Sands compacted and tested at moisture content range of 4.38 % - 4.88 %.

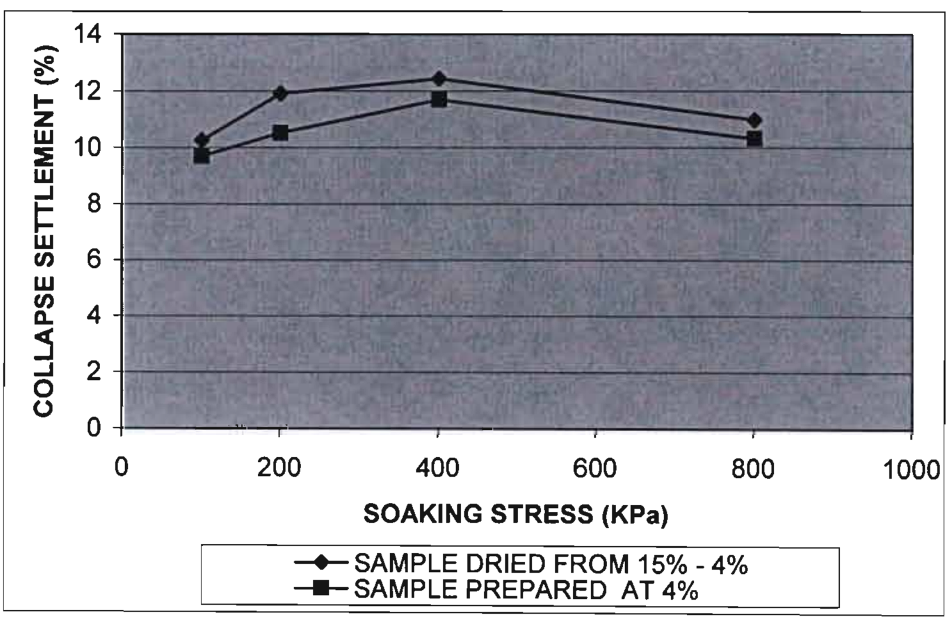


Fig. 4.4: Collapse settlement curves of compacted Berea Red Sands samples showing the effect of moulding water content on collapse settlement. The samples compacted at moisture content of 15 % and then subsequently dried down and tested at moisture content of 4.40 % - 4.88 %.

4 % exhibited more collapse settlement than samples compacted and tested at moisture content of 4.38 % - 4.88 %.

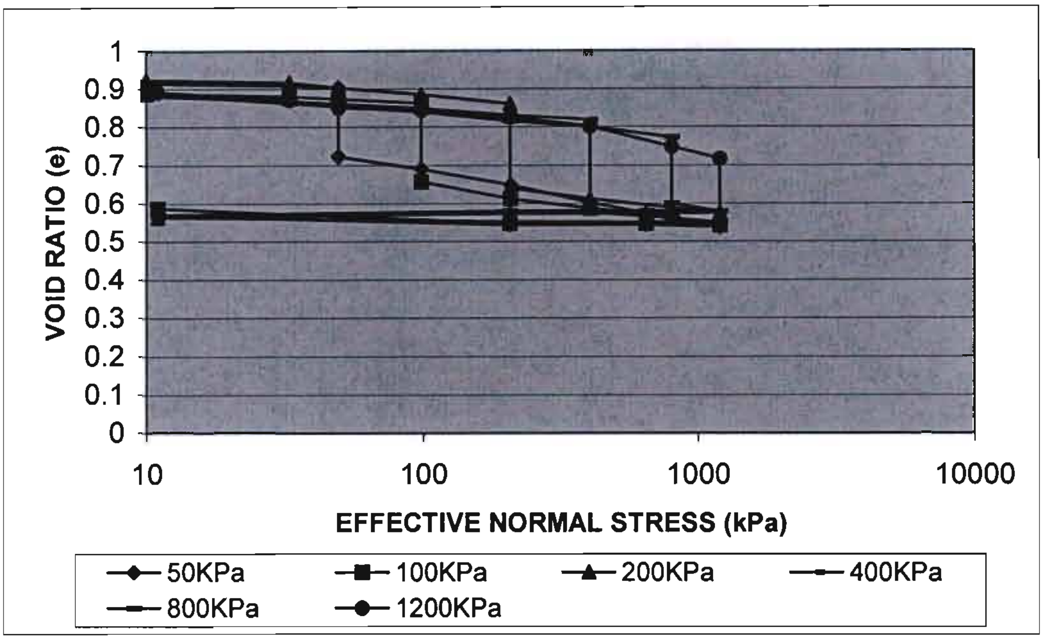


Fig. 4.5: Collapse settlement curves of compacted Berea Red Sands tested at moisture range of 4.1 % – 4.5 %, degree of saturation of 11.9 % - 12.5 % and dry density of 1400 kg/m^3 – 1403 kg/m^3

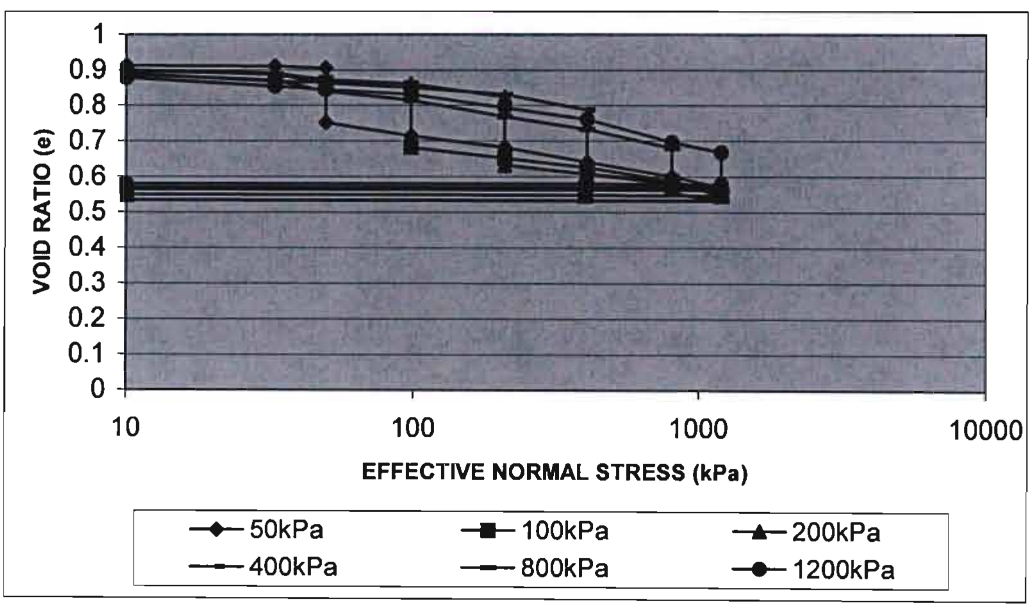


Fig. 4.6: Collapse settlement curves of compacted Berea Red Sands tested at moisture range of 6.15 % – 6.3 %, degree of saturation of 17.9 % - 18.32 % and dry density of 1403 kg/m^3– 1409 kg/m^3

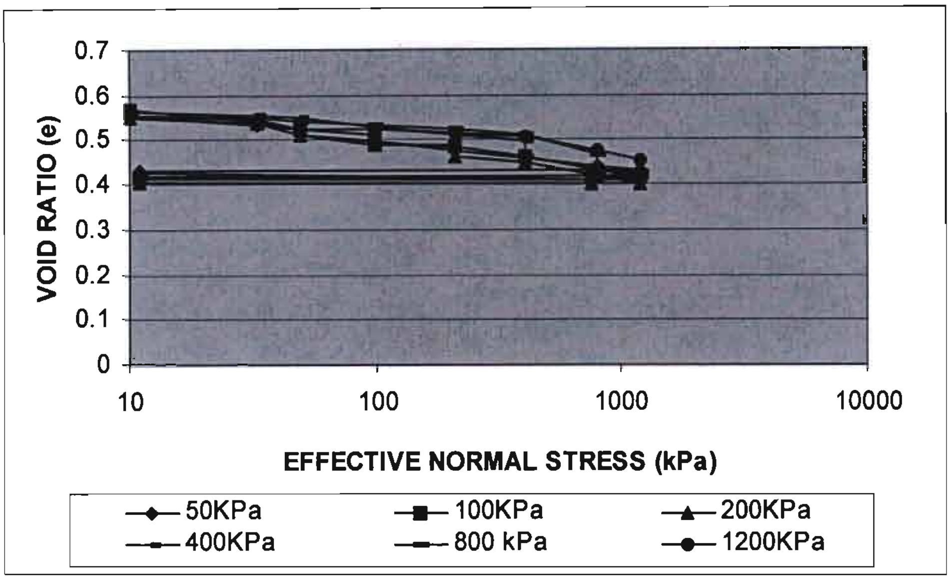


Fig. 4.7: Collapse settlement curves of compacted Berea Red Sands tested at moisture range of 5.7 % – 6.1 %, degree of saturation of 27.9 % - 28.3 % and dry density of 1712 kg/m^3 – 1714 kg/m^3

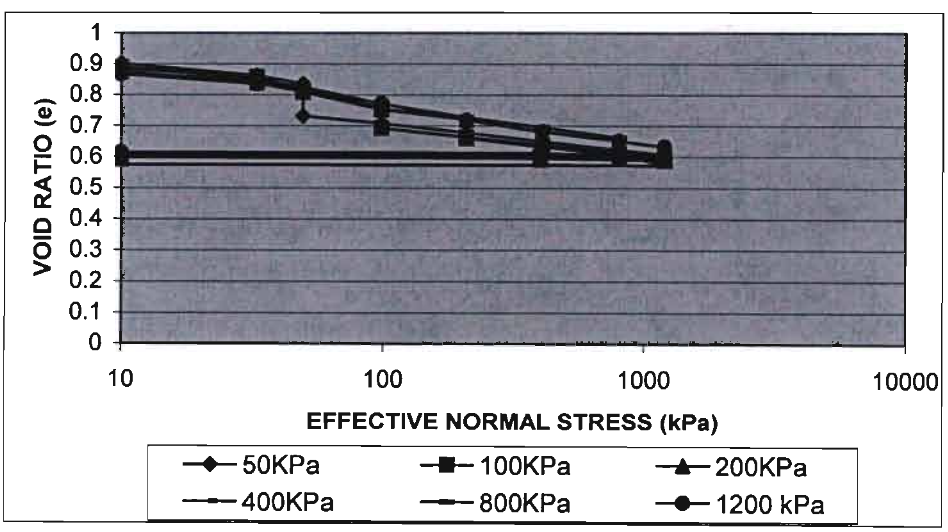


Fig. 4.8: Collapse settlement curves of compacted Berea Red Sands tested at moisture range of 8.04 %– 8.21 %, degree of saturation of 23.4 % - 23.88 % and dry density of 1402 kg/m^3 – 1407 kg/m^3

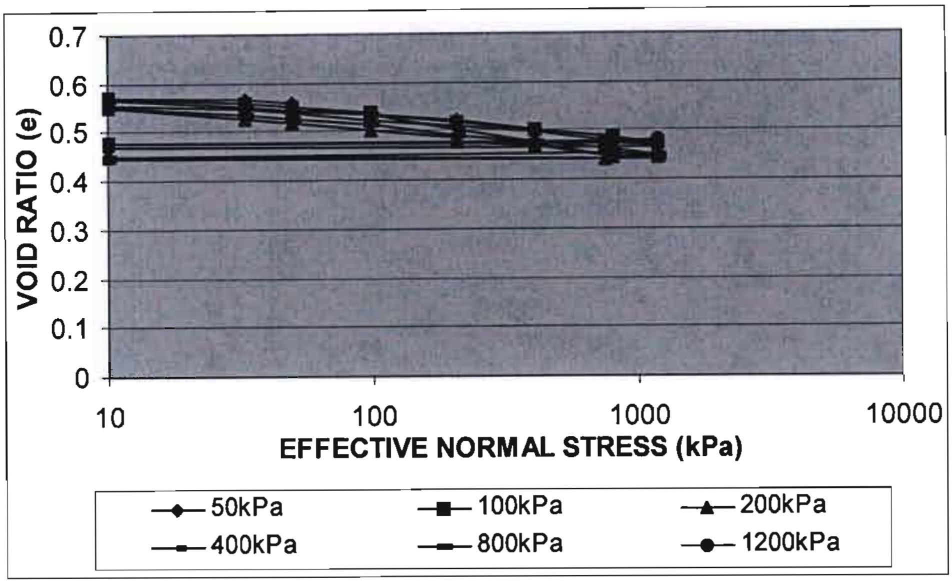


Fig. 4.9: Collapse settlement curves of compacted Berea Red Sands tested at moisture range of 7.8 % – 8.1 %, degree of saturation of 37.2 % - 38.2 % and dry density of 1712 kg/m^3– 1714 kg/m^3

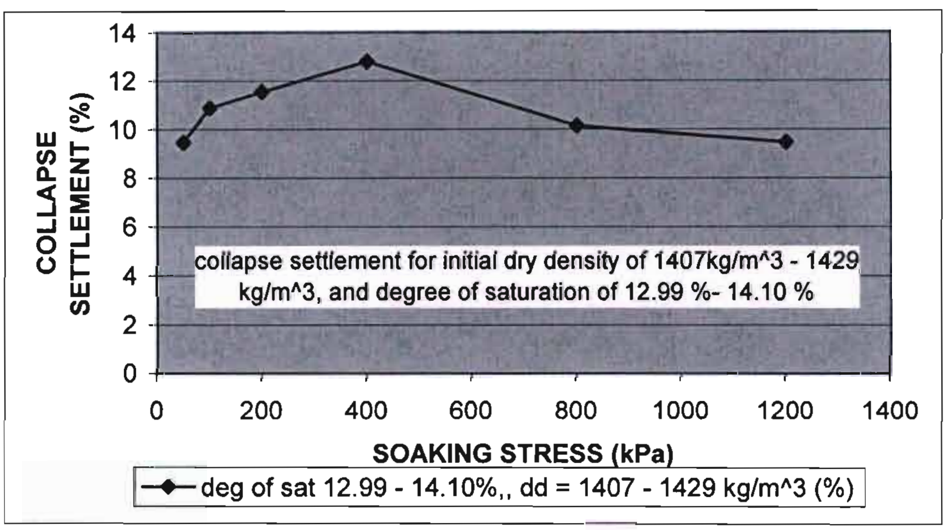


Fig. 4.10: Collapse settlement – soaking pressure curves of compacted Berea Red Sands tested at average moisture content of 4.2 %.

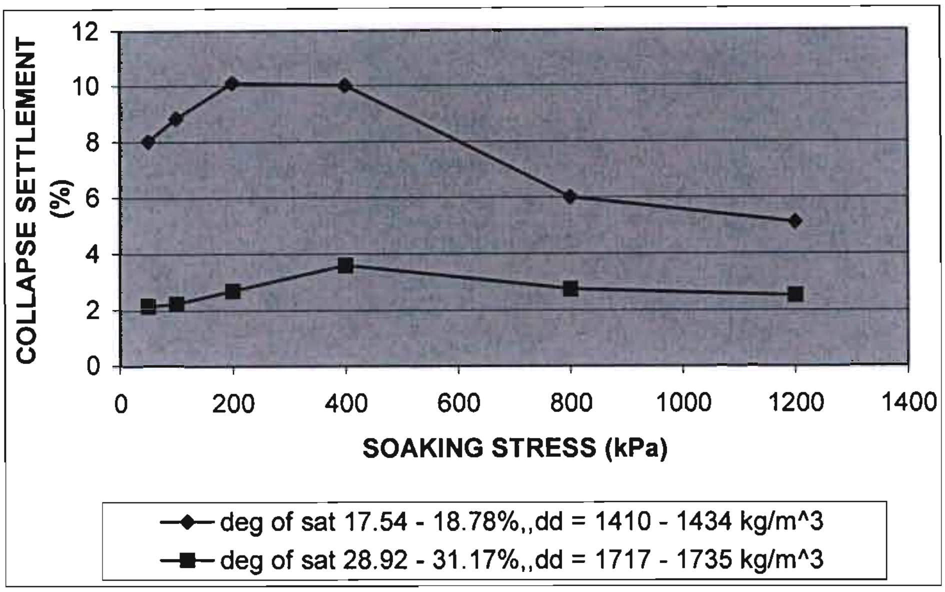


Fig. 4.11: Collapse settlement – soaking pressure curves of compacted Berea Red Sands tested at average moisture content of 5.8 %

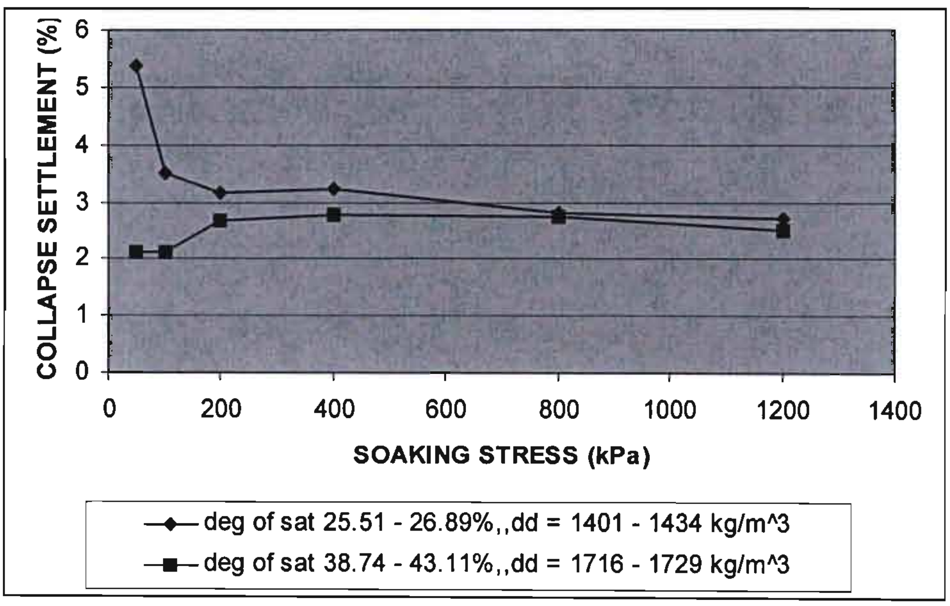


Fig 4.12: Collapse settlement – soaking pressure curves of compacted Berea Red Sands tested at average moisture content of 7.8 %

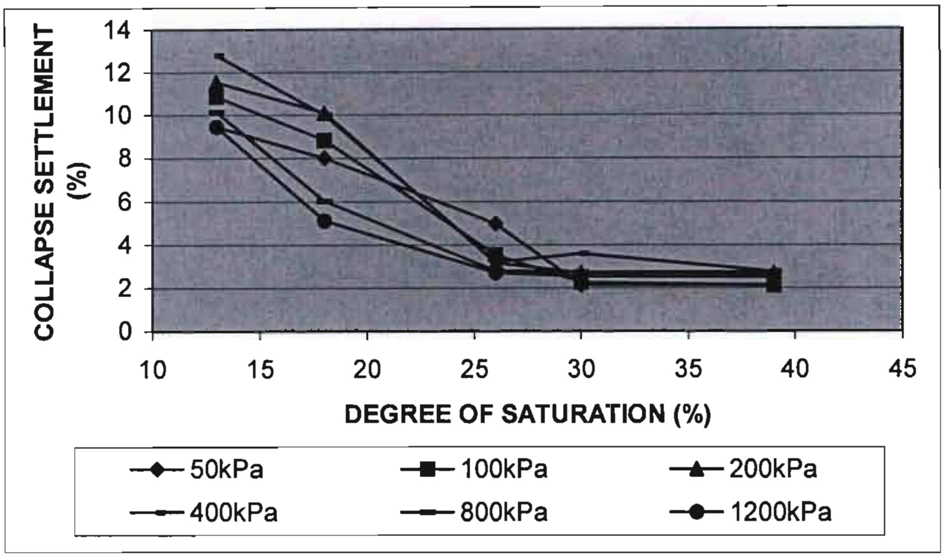


Fig. 4.13: Collapse settlement – saturation curves of compacted Berea Red Sands soaked at different normal stresses. Maximum collapse settlement at minimum degree of saturation is associated with soaking pressure of 400 kPa. The soaking pressure at which maximum collapse occur decreases with increase in degree of saturation. Stepwise downward trend from 400 kPa to 200 kPa to 100 kPa with increasing degree of saturation is indicated. For samples tested at any given degree of saturation, minimum collapse settlement is associated with high soaking pressures of 800 kPa and 1200 kPa.

CHAPTER FIVE

DIRECT SHEAR INDUCED COMPRESSION BEHAVIOUR OF BEREA RED SANDS

5.1 INTRODUCTION

Having established that Berea Red Sands compacted to low density exhibit significant moisture induced collapse settlement, it is thus necessary to establish whether significant volume compression might occur in Berea Red Sands compacted to low density and subjected to direct shear in the conventional shear box apparatus.

The boundaries of the direct shear box impose a pre-selected failure plane close to the mid height of the soil samples tested in direct shear box apparatus. Because a horizontal plane of failure which may not be the plane of ultimate shear stress is imposed on the sample by the boundaries of the shear box apparatus, the results of shear box tests are difficult to interpret in terms of Mohr Coulomb criterion. However if Berea Red Sands compacted to low density in the direct shear box exhibit significant volume compression in direct shear, then it would be necessary to study the shear induced volume compression behavior in simple shear where shear strength is mobilized by the whole sample and volume change can be used to determine contractile vertical strain and soil modulus.

Detailed laboratory study including test conditions, procedures, test results and discussions, of the volume change behavior of compacted Berea Red Sands in direct shear is presented in this chapter.

5.2. TEST PROCEDURE

The direct shear box apparatus available in the laboratory have internal base dimensions of 60 mm by 60mm and internal height of 50mm. The direct shear system available in the departmental laboratory is a linear assembly of three shear boxes driven at the same strain rate by motorized gear system connected by pulleys and a single chain. Three soil samples can be tested at the same time. The shear box devices are equipped with load cells to measure the resistance of the samples under applied normal stresses due to imposed horizontal displacement.

The samples that were tested were first compacted into the boxes at moisture content range of 16% - 18% and then slowly dried down in the laboratory to pre-selected moisture content values. The shear box tests were run on samples of 60mm by 60mm by 30mm thickness with half the thickness of the sample in the lower half of the box.

In their typical configuration, the conventional direct shear box devices can only run drained tests. The variables of interest are moisture content, vertical displacement and direct shear stress.

All the samples tested were subjected to a constant strain rate of 1mm per hr. The same strain rates were used in all the simple shear tests. The low dry density and high void ratio of the samples that were tested permits easy draining of moisture from the soil with little likelihood of pore pressure build up at the strain rate of 1mm per hr.

Series of tests were run on samples at various normal stresses and moisture contents. Four sets of samples were compacted into the direct shear box apparatus to dry densities of 1402 ± 20 kg/m^3 and moisture content of 16% - 18%. Each of the four sets were then gradually dried down to lower average moisture contents 4.2%, 7.8%, 10.3% and 13.8% respectively. Samples from each sets were selected and tested at normal stresses of 20 kPa, 40 kPa, 60 kPa, 100 kPa, 200 kPa and 400 kPa at constant strain rate of 1mm per 1hr. The samples were compacted into the shear box, dried to lower moisture content and tested at the above values of normal stresses. All tests were repeated, the curves examined for consistency, where necessary a third test was conducted and finally the mean results were plotted. A total number of sixty five direct shear tests were conducted. The moisture contents values reported here were determined after the direct shear tests.

5.3 DIRECT SHEAR TEST RESULTS

The results of the four sets of samples presented in test series 5.1 – 5.4 are shown in Fig. 5.1 to Fig. 5.4, and the failure envelopes are shown in Fig. 5.5.

It can be seen from the curves that within the range of imposed displacement and moisture content at which all the samples were tested, compacted Berea Red Sands exhibited elasto-plastic stress - displacement behavior with defined peak shear stress value in direct shear at low applied normal stresses up to 100 kPa. At normal stresses higher than 100 kPa, the stress displacement curves did not indicate peak values of shear stress within the range of displacement imposed by the direct shear apparatus. The slight non linear increase in shear stress with increasing horizontal displacement indicated at high normal stresses may be due to a combination of decreasing effective normal area of the sample and increasing granular friction as a result of decreasing void ratio resulting from volume reduction of the sample leading to increased internal resistance of the sample.

For samples tested at the same normal stress, the curves indicated increase in both stiffness (slope of the stress- displacement curves) and shear stress with increasing normal stress. Since the soil samples were compacted into the direct shear box to low density, increased stiffness at low normal stresses should not be expected from the samples as would have been expected from undisturbed weakly bonded samples.

While significant cohesion is not expected from the low kaolin clay fractions of the Berea Red Sand samples, it was however expected that significant cohesion due to matric suction would be indicated at least for samples tested at low degree of saturation. The failure envelopes shown in Fig 5.5 did not indicate significant cohesion even for the samples tested at low moisture contents where matric suction effects would dominate.

A general trend of decreasing angle of friction, from 31° to 28°, with increase in moisture content at which samples were tested is indicated in test series 5.10 – 5.40 and is due to increasing lubrication of the surface of the sand grains. Brink (1984) tested Berea Sands in the triaxial apparatus and reported angle of friction in the range 30° - 33°. Thus the direct shear box angle of friction is slightly lower than the triaxial values reported by Brink (1984). The particle size curve of Berea Red Soil samples which contain 4 % silt and 8 % clay content, used for the direct shear box tests shown in Fig. 4.1 plots very close to the lower boundary of the particle size envelope of Berea Sands reported by Brink (1984). Soils plotting near the lower envelope are highly granular, had experienced low degree of weathering, and are associated with high frictional angle.

The change in the volume of Berea Red Soils due to imposed direct shear strain is also shown in Fig. 5.1 to Fig. 5.4. It can be seen that within the range of moisture content and dry density of interest, compacted Berea Sand exhibited a decrease in sample thickness due to imposed direct shear strain. The shear-induced compressions tend to increase with increase in normal stress. This is clearly indicated in samples tested at moisture contents dry of the OMC. The above trend is not clearly indicated in samples tested at average moisture content of 13.5 %.

It must be noted that shear stress mobilization in the direct shear box is limited to a narrow band of soil close to the pre selected shear plane at the middle of the box. The actual thickness of this band of soil is uncertain but since shear induced compression increases with increase in normal stress, it is logical to state that the thickness of the narrow band of soil also increases with increase in normal stress.

The shear induced compression is therefore not representative of the entire thickness of the sample and so contractile vertical strain cannot be computed from shear induced compression of Berea Sands in a direct shear box apparatus.

5.4 JUSTIFICATION FOR SIMPLE SHEAR INVESTIGATION.

In chapter four, the effect of changes in moisture contents and dry densities on compression and collapse settlement behavior was presented. In this chapter it has been established that Berea Red Sands compacted to low density exhibit significant volume compression in direct shear.

The test data obtained from conventional shear box apparatus cannot used for the determination of contractile strain and modulus and thus this apparatus cannot be used for the study of the effect of volume change behavior of the soil along side a pile surface on the load settlement behavior of the pile.

Available literatures on direct shear behavior of soils (Airey et al, 1985; Jewell 1989) revealed that while significant stress and strain non uniformities might be expected in samples tested in conventional direct shear apparatus, the shear stress determined from such tests are only slightly higher than the values determined from simple shear tests data. Thus for samples tested at the same values of dry densities and moisture content the shear box results would provide an important guide for shear stresses expected in simple shear tests.

TABLE 5.1

TEST SERIES 5.1: Direct shear box tests on samples at moisture content range of 3.81 % - 4.87 %

test no	normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test Mg/m^3	shear stress (kPa)	angle of friction (degrees)
5.11	20	15.81	3.81	1.426	17	31
5.12	40	16.03	4.39	1.457	30	
5.13	60	16.10	3.54	1.433	40	
5.14	100	16.52	4.24	1.436	76	
5.15	200	15.45	4.87	1.435	121	
5.16	400	15.84	4.01	1.445	246	

TABLE 5.2

TEST SERIES 5.2: Direct shear box tests on samples at moisture content range of 7.04% - 7.90%

test no	normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	shear stress (kPa)	angle of friction (degrees)
5.21	20	16.13	7.81	1.424	15	30
5.22	40	15.59	7.18	1.452	28	
5.23	60	16.89	7.24	1.433	37	
5.24	100	16.52	7.04	1.436	58	
5.25	200	13.51	7.44	1.435	112	
5.26	400	17.42	7.90	1.436	233	

TABLE 5.3

TEST SERIES 5.3: Direct shear box tests on samples at moisture content range of 9.58 % - 10.60 %

test no	normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	shear stress (kPa)	angle of friction (degrees)
5.31	20	16.86	9.89	1.424	13	
5.32	40	15.00	9.92	1.452	25	
5.33	60	17.32	10.60	1.433	34	
5.34	100	16.52	9.49	1.436	54	
5.35	200	15.81	10.44	1.435	108	
5.36	400	17.09	9.58	1.436	223	29

TABLE 5.4

TEST SERIES 5.4: Direct shear box tests on samples at moisture content range of 13.70% - 14.53%

test no	normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	shear stress (kPa)	angle of friction (degrees)
5.41	20	18.43	13.70	1.411	12	
5.42	40	15.27	13.99	1.424	24	
5.43	60	16.89	13.84	1.433	32	
5.44	100	19.48	13.82	1.408	51	
5.45	200	18.10	14.53	1.407	102	
5.46	400	16.24	14.02	1.454	213	28

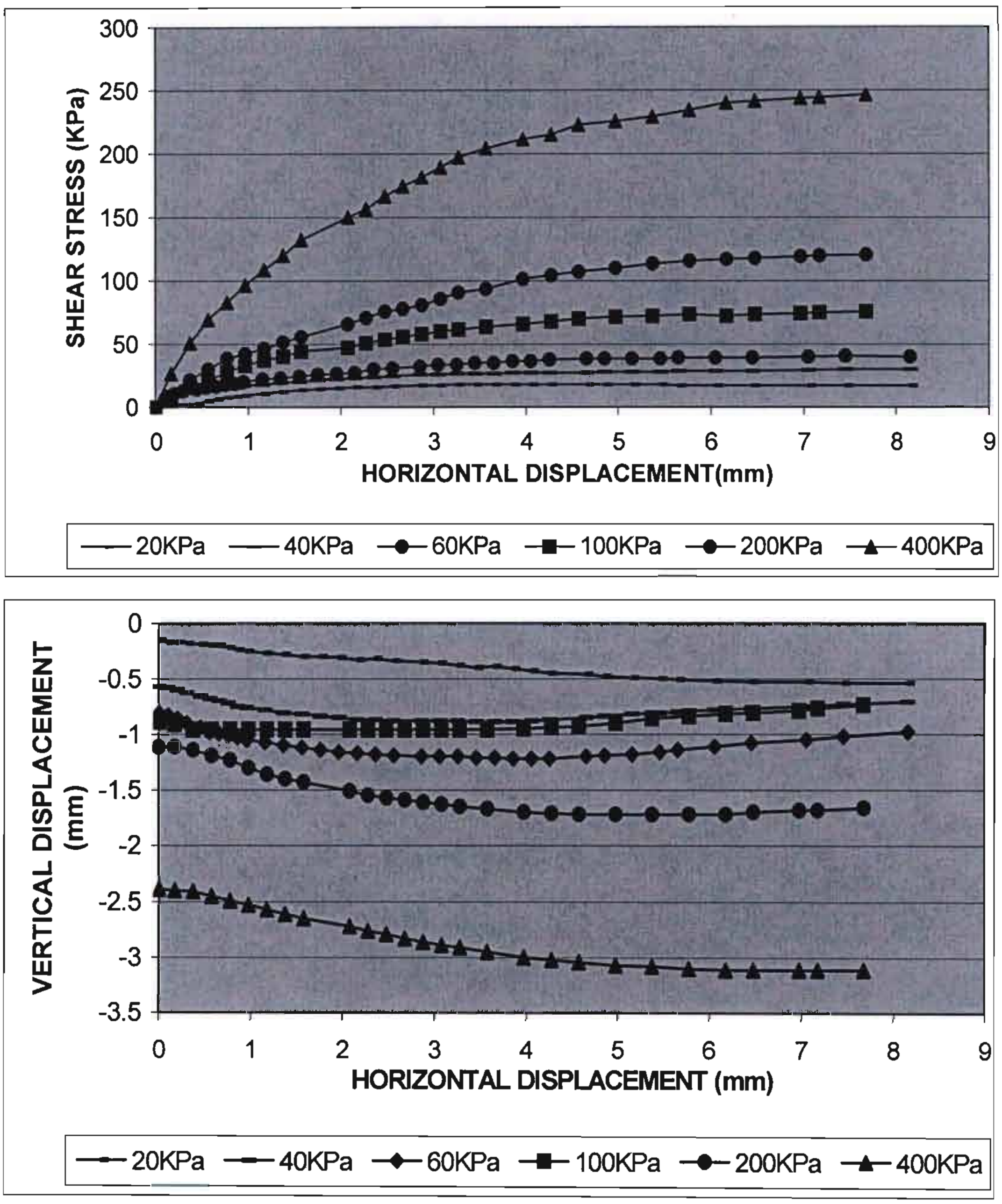


FIG 5.1: Direct shear curves of compacted Berea Red Sands. Samples were compacted to 30mm height, dry density of 1426 kg/m^3 – 1457 kg/m^3 and moisture content of 15.45%-16.03% and tested at moisture content of 3.81 %.- 4.87%

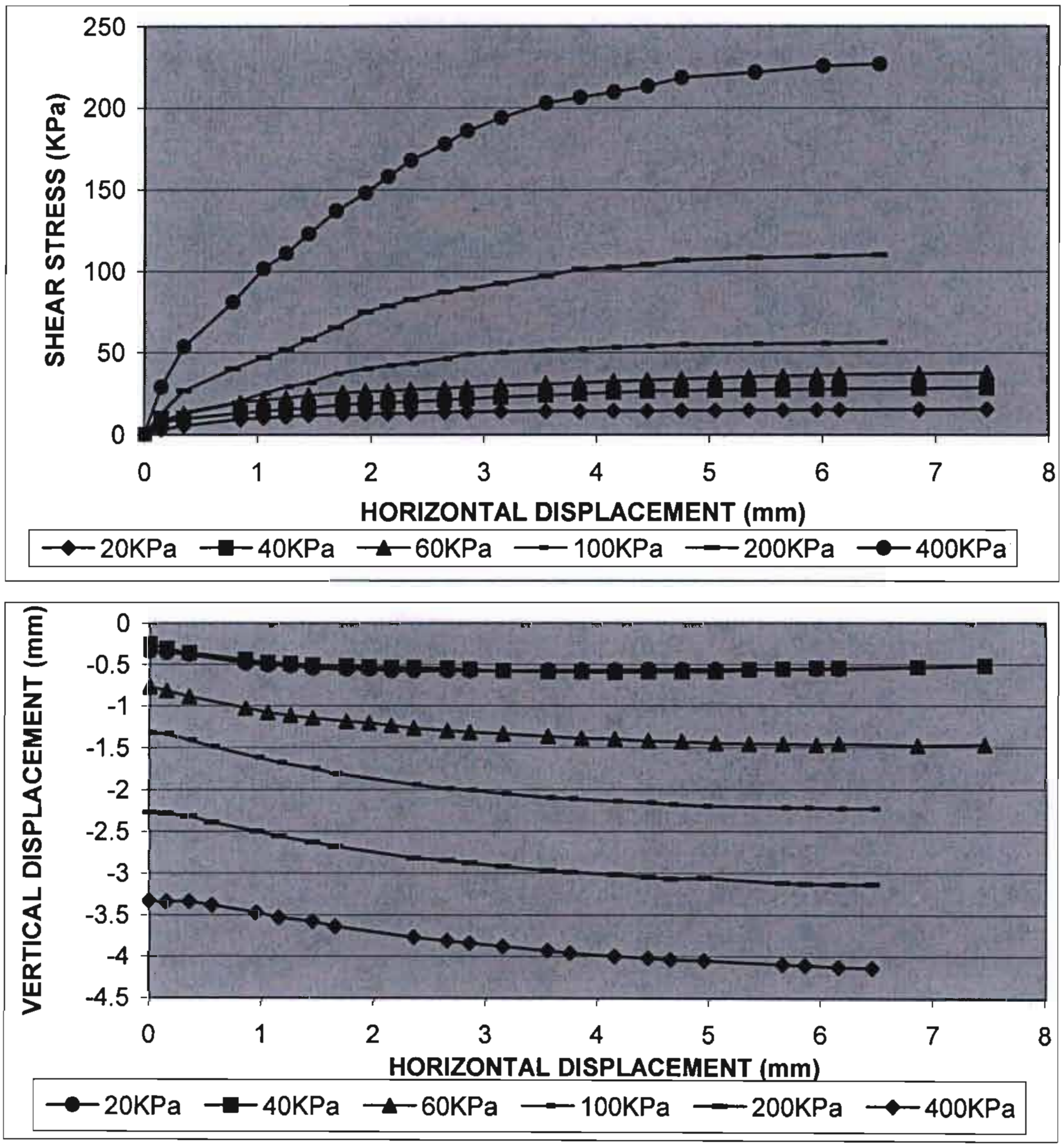
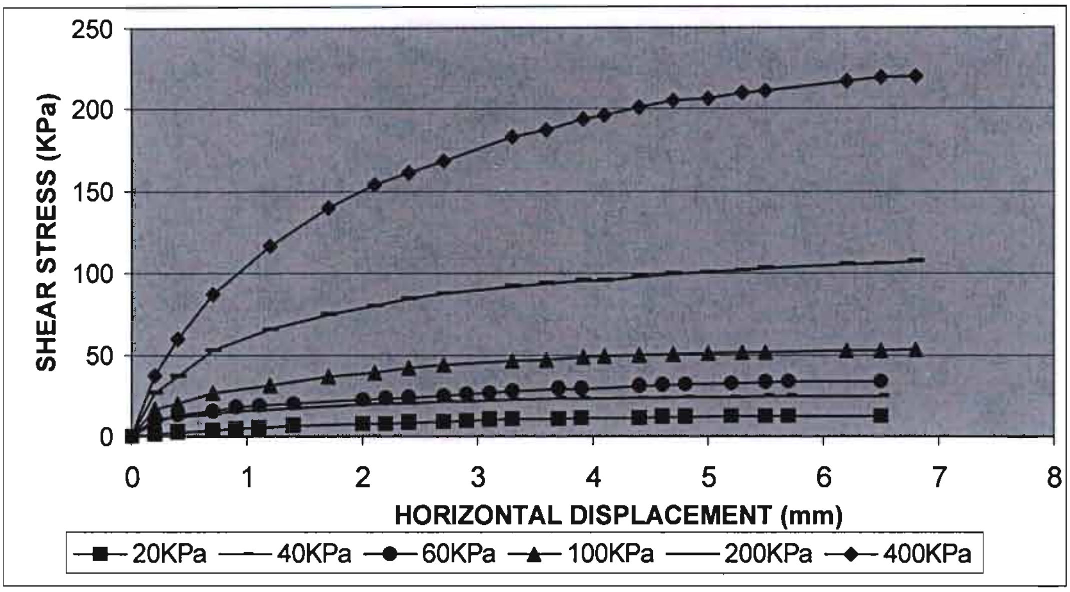


FIG 5.2: Direct shear curves of compacted Berea Red Sands. Samples were compacted to 30mm height, dry density of 1424 kg/m^3 – 1452 kg/m^3 and moisture content of 15.59%-17.42% and tested at moisture content of 7.04 %.- 7.90%



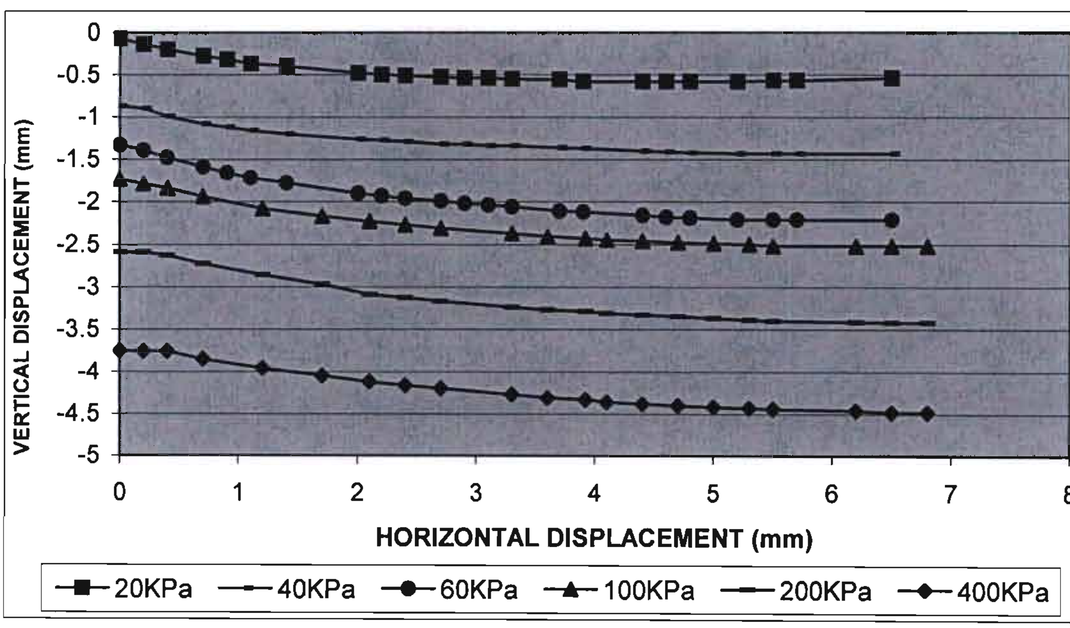
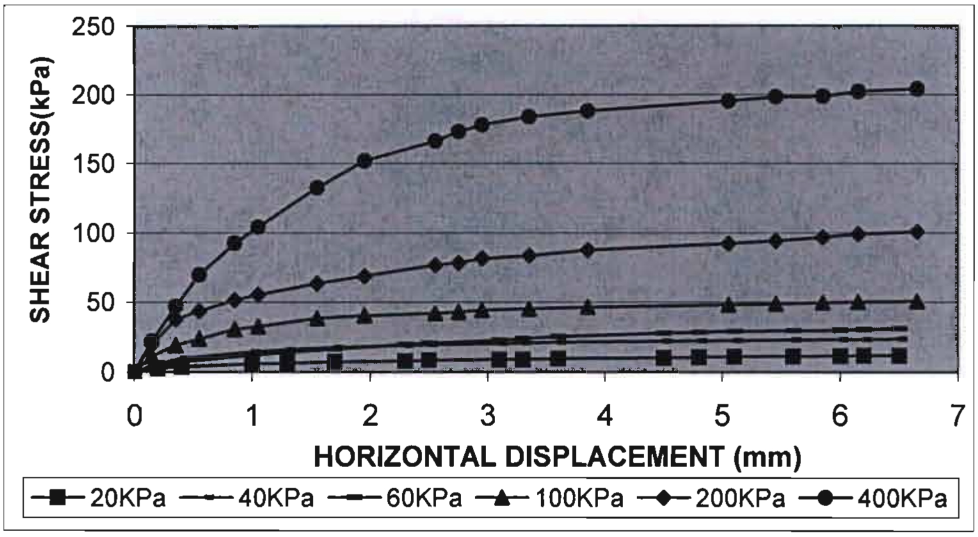


FIG 5.3: Direct shear curves of compacted Berea Red Sands. Samples were compacted to 30mm height, dry density of 1426 kg/m^3 – 1457 kg/m^3 and moisture content of 15.00%-17.32% and tested at moisture content of 9.49 %.- 10.60%



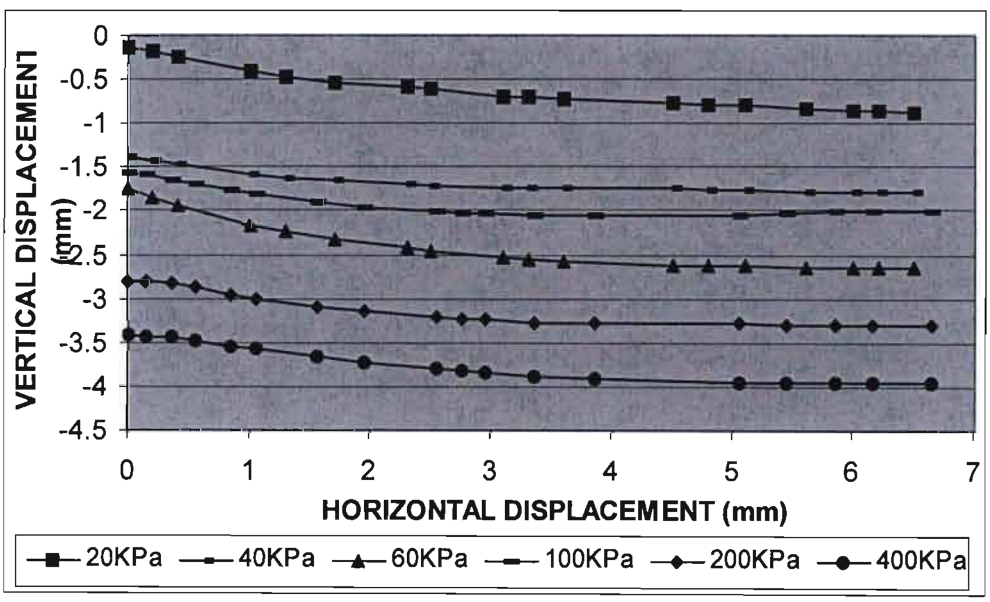


FIG 5.4: Direct shear curves of compacted Berea Red Sands. Samples were compacted to 30mm height, dry density of 1426 kg/m^3 – 1457 kg/m^3 and moisture content of 16.27%-17.48% and tested at moisture content of 13.70%.-14.53%

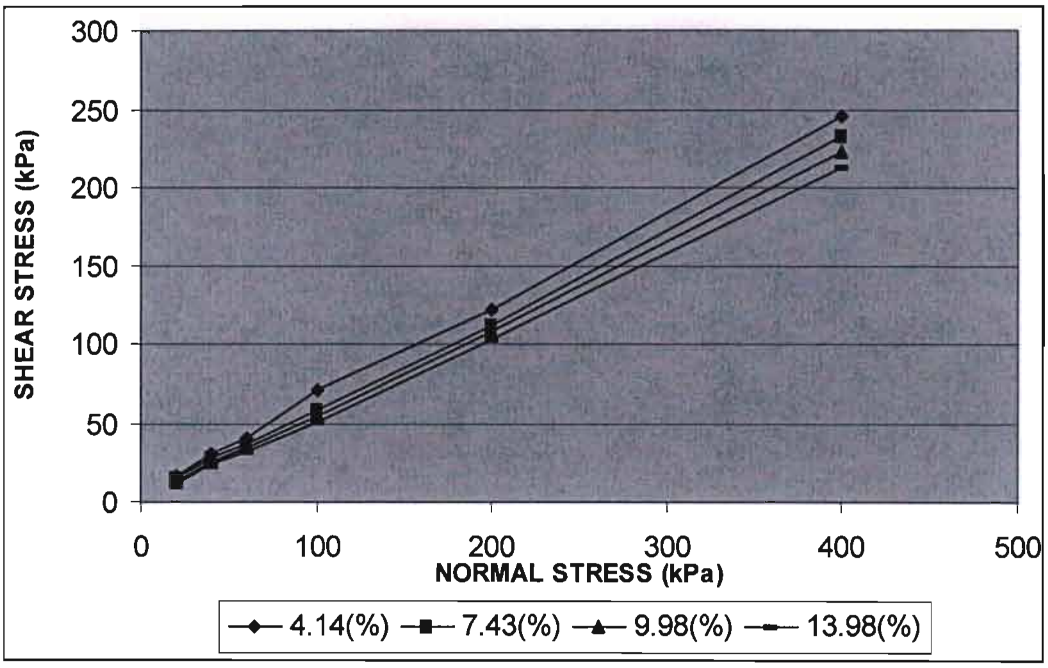


FIG 5.5: Strength envelopes for compacted Berea Red Sands compacted to height of 30mm, dry density range of 1424 kg/m^3- 1457 kg/m^3.and moisture content range of 4.14% - 13.98%

CHAPTER SIX

ANALYSIS OF SIMPLE SHEAR FAILURE CONDITIONS IN SANDS

6.1 INTRODUCTION

In chapter 5, it was established that Berea Red Sands compacted to low dry density exhibit volume compression in direct shear. Direct shear induced volume compression of Berea Sands is however of little relevance to the study of load settlement behavior of vertically loaded piles in Berea Sands because the volume measured in direct shear devices cannot be converted to contractile strain and modulus. Contractile strain and modulus of the soil close to the pile surface are needed for the determination of load settlement behavior of piles in soil deposits and since the mode of deformation of the soil close to the pile surface is of simple shear mode, there is the need to study the behavior of soils in simple shear apparatus, in which soil samples are subjected to simple shear deformation, and which permits the computation of contractile strain and soil modulus from test data.

A primary requirement for the development of a new simple shear apparatus for sands is the analysis of stresses and strains imposed on sand specimen subjected to simple shear strain. Analysis of stresses and strains in simple shear deformation is necessary for the design of a simple shear apparatus.

The stress conditions in the NGI and Cambridge type simple shear apparatus differ because of the different types of imposed test boundaries. Direct computation of instantaneous changes in shear and volumetric strains in specimens tested in the NGI device is complicated by difficulty in measuring changes in the cross sectional area and volume of deformed cylindrically shaped bodies. The regular geometry and rigid nature of the Cambridge type simple shear box permits both accurate measurements of volume change in samples during shear and accurate measurement of forces applied at the boundaries of the Cambridge type device.

The analysis of sand specimens subjected to simple shear strain in the Cambridge type apparatus was carried out in two stages. Stage one entails the evaluation of horizontal and vertical stresses in the sample with respect to the stress tensors assuming uniform stress conditions exists in the entire sample during simple shear testing. For this assumption pure shear conditions exist in the specimen and thus failure was described by the Mohr Coulomb criteria. The change in the ratio of horizontal to vertical stress at different stages, before the imposition of simple shear strain, at first yield and at ultimate failure conditions, were evaluated.

Stage two is the rigorous analysis of the effect of the test boundary conditions on the stress – strain response, as it is recognized that the simple shear strain imposed on a sample compacted into the shear box does not result in pure shear conditions. The rigorous analysis assumes that plastic deformation is induced from the on set of simple shear strain. Some of the results of the first stage analysis were used in the second stage analysis for the development of the equation of simple shear stress in terms of externally applied load and the imposed boundary conditions.

Derivatives of the second stage analysis facilitated the evaluation of the effect of the height to length ratio of the specimen on the stress – strain behavior.

6.2 SIMPLE SHEAR ANALYSIS BASED ON STRESS TENSOR.

The following assumptions were made (Roscoe et al, 1967; Airey et al, 1985) for the analysis of the simple shear tests on sands at peak shearing conditions based on detailed radiographic observations of samples during tests;

a) There is sufficient uniformity for the deforming sands to be described in terms of a single state of stress and incremental strain.
b) The orientation of the principal axes of stress and incremental strain coincides as sample deform in simple shear.
c) At ultimate plastic failure, the horizontal planes are essentially sets of velocity characteristics, such planes are inclined at $\pm (45 - \psi/2)$ to the direction of the principal plastic strain increment and to the direction of major principal stress.
Ψ is the rotation of the major principal stress from the vertical axis.

For a typical soil specimen subjected to simple shear strain and constant normal stress during test, the shear strain ε_{xy} is increased from zero, while the shear stress τ_{xy} and the vertical strain ε_y are observed.

Duncan and Dunlop (1982) and Airey et al, (1985) proposed that the horizontal plane through a soil specimen on which the simple shear stress act can be represented by any of the three Mohr circles shown in Fig. 6.2, Fig. 6.3 and Fig. 6.4. For the three different failure conditions shown in Fig. 6.2, Fig. 6.3 and Fig. 6.4, the stress ratios at failure were determined and compared below.

Failure Condition A

For the failure condition in which the horizontal plane is a plane of maximum stress obliquity and the angle of friction mobilized in the sand is $\tan\phi$, the Mohr circle of stress is represented by Fig. 6.2, where point N is the pole of the circle and point H represents the stresses τ_{xy} and σ_y which acts on the horizontal plane. From the geometry of the circle shown in Fig 6.2, the stresses are derived thus;

$$\tau_{xy} = r\cos\varphi'$$

$$\sigma_y = \frac{r}{\sin\phi} - r\sin\varphi'$$

$$\left(\frac{\tau_{xy}}{\sigma_y}\right)_u = \frac{\sin\phi\cos\psi}{1-\sin\phi\sin\psi} \quad \text{.......... .... 6.1}$$

when $\psi = 0$

$$\left(\frac{\tau_{xy}}{\sigma_y}\right)_u = \sin\varphi'$$

when $\psi = \varphi$

$$\left(\frac{\tau_{xy}}{\sigma_y}\right)_u = \tan\varphi'$$

Failure Condition B

The mode of failure of the soil could be vertical failure planes plus rigid body rotation. This mode of failure offers the least resistance to simple shear deformation and is thus defined by the least stress ratio at failure. The Mohr circle of stress for the state is represented by Fig 6.3 and the stress ratio at failure is determined thus;

$$\tau_{xy} = r\cos\varphi'$$

$$\sigma_y = \frac{r}{\sin\phi} + r\sin\varphi'$$

$$\left(\frac{\tau_{xy}}{\sigma_y}\right)_u = \frac{\sin\phi\cos\psi}{1+\sin\phi\sin\psi}$$

when $\psi = 0$

$$\left(\frac{\tau_{xy}}{\sigma_y}\right)_u = \sin\varphi'$$

when $\psi = \varphi$

$$\left(\frac{\tau_{xy}}{\sigma_y}\right)_u = \frac{\sin\phi\cos\phi}{1+\sin^2\phi} \quad \text{.......} 5.2$$

Failure Condition C

The direction of the major principal stress at failure may not be consistent with that involved in ultimate plastic failure represented by Fig 6.2 and Fig 6.3. For the special case where the major principal stress is already orientated at ± (45 – $\psi/2$) to the horizontal direction when failure occurs, i.e. $\psi = \varphi$, the horizontal plane is a plane of maximum shear stress. Geometrically it can be represented by the Mohr circle shown in Fig 6.4.

$$OB = \frac{\sigma_y - \sigma_x}{2}$$

$$AO = \sqrt{(OB)^2 + (AB)^2} \,\ldots\ldots = \ldots \sqrt{\left(\frac{\sigma_y - \sigma_x}{2}\right)^2 + \left(\frac{\sigma_y - \sigma_x}{2\sin\phi}\right)^2} \ldots$$

$$\frac{\tau_{xy}}{\sigma_y} = \frac{1}{2}\sqrt{\left(1+\frac{\sigma_x}{\sigma_y}\right)^2 \sin^2\phi - \left(1-\frac{\sigma_x}{\sigma_y}\right)^2}$$

$$\frac{\tau_{xy}}{\sigma_y} = \frac{OB}{OA} = \frac{1}{2}\sqrt{\left(1+\frac{\sigma_x}{\sigma_y}\right)^2 \sin^2\phi - \left(1-\frac{\sigma_x}{\sigma_y}\right)^2} \quad \text{.......} 6.3$$

Setting $\frac{\sigma_x}{\sigma_y} = k$

then the stress ratio is,

$$\frac{\tau_{xy}}{\sigma_y} = \frac{1}{2}\sqrt{(1+k)^2 \sin^2\phi - (1-k)^2} \quad \text{.......} 6.4$$

k is the lateral stress coefficient and defines the stress state of the sample at different failure conditions with respect to the stress ratio.

The value of k when the stress ratio at failure is zero is derived below.

$$(1-k)^2 = (1+k)^2 \sin^2 \phi$$

$$\frac{(1-k)^2}{(1+k)^2} = \sin^2 \phi$$

$$\frac{(1-k)}{(1+k)} = \pm \sin \phi$$

For sinφ = +ve

$$k = \frac{(1-\sin\phi)}{(1+\sin\phi)} = k_a$$

For sinφ = -ve

$$k = \frac{(1+\sin\phi)}{(1-\sin\phi)} = k_p$$

The stress ratio at first yield has a maximum value at an intermediate value of k determined by setting the derivative of the stress ratio to zero;

$$\frac{\partial \tau_n}{\partial \sigma_y} = \frac{1}{4(1+k)^2 \sin^2 \phi (1-k)^2} * 2(1+k)\sin^2 \phi + 2(1-k) = 0$$

$$2(1+k)\sin^2 \phi + 2(1-k) = 0$$

$$k = k_u = \frac{1+\sin^2\phi}{1-\sin^2\phi} \quad \text{..6.5}$$

In summary, the stress ratio, will be zero, when

$$\frac{\sigma_x}{\sigma_y} = \frac{1-\sin\phi}{1+\sin\phi}$$

(active pressure state),

or when

$$\frac{\sigma_x}{\sigma_y} = \frac{1+\sin\phi}{1-\sin\phi}$$

(passive pressure state).

The stress ratio has a maximum value for an intermediate value of *k*= *km*:

$$Km = \frac{\sigma_x}{\sigma_y} = \frac{1+\sin^2\phi}{1-\sin^2\phi}$$

and here $\frac{\tau_{xy}}{\sigma_y} = \tan\phi$

Thus different values of $\frac{\tau_{xy}}{\sigma_y}$ when failure begins correspond to the different conditions of the principal stress directions in relation to the shear plane.

The possible stress ratios at failure in relation to *k* are given below;

For the case corresponding to

$$\frac{1-\sin\varphi}{1+\sin\varphi} \le \frac{\sigma_x}{\sigma_y} \le 1$$

Failure begins for a value $\frac{\tau_{xy}}{\sigma_y}$ in the interval $0 \le \frac{\tau_{xy}}{\sigma_y} \le \sin\varphi$. As the test proceeds both $\frac{\sigma_x}{\sigma_y}$ and $\frac{\tau_{xy}}{\sigma_y}$ increase Eventually they approach asymptotically the upper limits $\tau_{xy} = \sigma_y \sin\varphi$ at $\sigma_x = \sigma_y$ if constant volume condition is maintained at rupture.

Correspondingly, in the cases where $\sigma_y \le \sigma_x$, failure is associated with values of *k* within the range.

$$\frac{1+\sin^2\varphi}{1-\sin^2\varphi} \le \frac{\sigma_x}{\sigma_y} \le \frac{1+\sin\varphi}{1-\sin\varphi}$$

or within the range

$$1 \le \frac{\sigma_x}{\sigma_y} \le \frac{1+\sin^2\varphi}{1-\sin^2\varphi}$$

The boundary conditions imposed by the walls of the Cambridge type simple shear apparatus does not permit either K_a or K_p stress conditions at failure, since this are limiting infinite conditions.

For residual sands compacted to low density, stress conditions during constant volume simple shear tests imply that k changes from K_o to 1, i.e. $\sigma_x = \sigma_y$ since K = 1 correspond to the limiting stress value $\tau_{xy} = \sigma_y \sin\varphi$.

For the same soil tested under constant normal stress conditions, bulk volume collapse of soil structure within fixed lateral boundaries implies that K changes from K_o at start of test, then asymptotically approach 1, K_m or K_p. However bulk volume dilation of soil structure within fixed lateral boundaries implies that K changes from K_o at start of test then asymptotically approach K_a.

Thus the theoretical infinite stress ratios approached asymptotically depends on the initial stress state of the sample and mode of normal stress application. These values are never reached, are asymptotes and the function can therefore be represented by hyperbolas.

The change in k from K_o to either K_a, 1 or K_p was selected after assessing the following equations;

$$K = K_p - [\frac{(\phi-\alpha)^2}{\phi^2}] * (K_p - K_o) \qquad 6.6$$

$$K = K_o + \frac{\alpha}{\frac{1}{K_o} + \frac{\alpha}{K_x - K_o}} \qquad 6.7$$

$(0 < \alpha. < \varphi)$ and $K_x = 1$ or K_p

$$K = K_p \sin\alpha + K_o \cos\alpha \qquad 6.8.$$

$$K = K_p - K_o[1 - e^{-K_o \alpha}) \qquad 6.9$$

ϕ is the angle of friction of the sand.

α is the angle of rotation of the ends of the specimen.

The curves are shown in fig 6.5. For Curve B representing Eq 6.7 shown in fig 6.6, the change in K is asymptotic for all practical values of shear strain imposed on the sample. Fig 6.6 also show the hyperbolic curves of the lateral pressure coefficient for different mode of simple shear failure i.e. change from K_o to 1, K_m and K_p

The Curve representing Equation 6.7 was chosen because the preliminary stress strain curves predicted from Equations 6.6, 6.7 and 6.9 are neither similar in shape to stress strain curves encountered in soil mechanics tests nor are the curves similar to that determined from direct shear tests data. Stress strain curves predicted by the application of Equation 6.7 are similar to direct shear curves.

The shape of the predicted stress strain curve may or may not bear any resemblance to the hyperbolic K curves as this will depend on the amount of boundary induced deformation which can be determined analytically (Fig 7.2 A and B) and shear induced compression which can only be measured after testing of the real soil samples (Fig 8.9 Fig 8.12 and Fig 8.21)

However since the typical Cambridge type apparatus imposes simple shear strain on a specimen confined in a rigid square or rectangular wall by the rotation of the end flaps or plates, the simple shear strain imposed on a sample compacted into the shear box does not result in the pure shear conditions because of uncomplimentary stresses on the vertical faces of the specimen and thus end effects (Roscoe 1953; Duncan and Dunlop 1982; Airey et al, 1982).

The resulting non uniformity of stresses induced in the specimen imply that stresses measured at the boundaries of the simple shear device are expected to differ from stresses at the core or middle third where uniform stress and strain conditions exist. Since only stresses and strains at the boundaries of conventional simple shear apparatus are measured, these boundary parameters are evaluated below.

6.3 STRESSES AT BOUNDARIES OF CAMBRIDGE TYPE APPARATUS

Since a primary condition for the attainment of maximum stress ratio is the coincidence of α and ϕ' at yield, it can be assumed that the rotation of the vertical faces of the specimen coincides with the rotation of principal stress associated with the ideal pure shear condition in the middle third of the sample (Duncan and Dunlop 1982). The above assumption satisfies Failure Condition C since the horizontal plane is a plane of maximum shear stress, where the major principal stress is already orientated at ± (45 – ψ/2) to the horizontal direction when failure occurs.

The assumption simplifies subsequent stages of the analysis.

The stresses mobilized by a soil sample within the rigid boundaries of a simple shear device due to imposed simple shear strain can be evaluated by considering an idealized slice shown in Fig. 6.1 with infinitesimal unit thickness at the middle of the soil sample along the direction of shearing. The specimen is defined by the following;

γ' = Effective unit weight of the soil

q = externally applied normal stress

H = height of sample.

L = length of sample

K_o = coefficient of lateral pressure at rest

K_u = coefficient of lateral pressure during shear = K in Eq 6.7

K_p = asymptotic value of lateral pressure

a = simple shear strain

Before the imposition of shear strain, the stress state within the sample under applied normal stress is assumed to be the at rest stress state K_o, although for samples compacted to low density the correct value of K at rest should be slightly higher than K_o as such samples are not normally consolidated. Due to imposed strain K_o changes to K_u the coefficient of lateral pressure during shearing. At large strain or at failure K_u approaches K_p asymptotically.

For a rectangular slice of sample subject to angular strain due to the rotation of the end flaps, the stresses due to imposed shear strain was evaluated as follows;

For the cross section of a sample shown in Fig 6.1, the at rest pressure on the plane AB at depth H before simple shear strain application was evaluated as

$$P_o = K_o \gamma' H + K_o q$$

The resultant horizontal pressure on the plane AB at depth H due to simple shear strain was evaluated as

$$P = (K_u \gamma' H \cos\alpha + K_u q \cos\alpha) + (K_u \gamma' H \cos\alpha + K_u q \cos\alpha)\tan\alpha \sin\alpha$$

The net horizontal pressure Ph acting across the sample at depth H due to externally imposed strain is $P - P_o$ given as

$$P_h = P - P_o = (K_u \gamma' H \cos\alpha + K_u q \cos\alpha) + (K_u \gamma' H \cos\alpha + K_u q \cos\alpha)\tan\alpha \sin\alpha - (K_o \gamma' H + K_o q)$$

Combining and integrating the above equations over the height of the sample;

$$\int_0^H (P_h)\partial H = \frac{K_u \gamma' H^2 \cos\alpha}{2} + K_u q H \cos\alpha + \frac{K_u \gamma' H^2 \sin^2\alpha}{2} + K_u q H \sin^2\alpha - \frac{K_o \gamma' H^2}{2} - K_o q H$$

The above expression represents the force per unit width (thrust) imposed by the sample on the flap. The thrust due to the weight of the sample acts through a point H/3 from the base, while the thrust due to the applied normal stress acts through a plane H/2 from the base.

Considering equilibrium of the end flaps, the combined moments due the above thrust is equal to the moment due to the average external horizontal force at the base of the flap for any value of α. Thus;

$$FH\cos\alpha = \frac{K_u \gamma' H^3 \cos\alpha}{6} + \frac{K_u q H^2 \cos\alpha}{2} + \frac{K_u \gamma' H^3 \sin^2\alpha}{6} + \frac{K_u q H^2 \sin^2\alpha}{2} - \frac{K_o \gamma' H^3}{6} - \frac{K_o q H^2}{2}$$

Thus the externally applied force is

$$F\cos\alpha = \frac{K_u \gamma' H^2 \cos\alpha}{6} + \frac{K_u q H \cos\alpha}{2} + \frac{K_u \gamma' H^2 \sin^2\alpha}{6} + \frac{K_u q H \sin^2\alpha}{2} - \frac{K_o \gamma' H^2}{6} - \frac{K_o q H}{2}$$

The average shear stress mobilized by the sample is the externally applied force per horizontal cross sectional area of the sample (Roscoe 1953; Randolph and Wroth 1981). Thus considering unit width of sample;

$$\frac{F\cos\alpha}{L} = \frac{K_u \gamma' H^2 \cos\alpha}{6L} + \frac{K_u q H \cos\alpha}{2L} + \frac{K_u \gamma' H^2 \sin^2\alpha}{6L} + \frac{K_u q H \sin^2\alpha}{2L} - \frac{K_o \gamma' H^2}{6L} - \frac{K_o q H}{2L} \quad \text{......6.10}$$

Eq 6.10 is the simple shear stress equation for a sample subjected to simple shear strain assuming unit width of the specimen. It satisfies the conditions that no shear strength is mobilized by the specimen when subject to zero imposed simple shear strain.

For a soil specimen of given dimension, the shear strength can be determined for any value of simple shear strain α, applied stress q and dry density γ.

Eq. 6.10 is limited in that it cannot determine the effect of changes in water content of the sample on shear strength. Because the shear strength of soils decreases with increasing water content, Eq. 6.10 represents the average shear stress of the dry sample for given values of applied stress q and dry density γ.

6.4 GENERAL SUMMARY.

There are different modes of failure for a soil sample subject to simple shear strain. Failure condition C is for the special case where the major principal stress is already orientated at $\pm (45 - \psi/2)$ to the horizontal direction when failure occurs. Since the horizontal plane is a plane of maximum shear stress for this mode of failure, the change in the magnitude of the coefficient of lateral stress k was used to determine the different stress ratios at failure.

The stress ratios K_a, 1 and K_p represent theoretically failure conditions. These failure conditions are never reached in simple shear tests and thus are asymptotic.

The initial stress ratio k for a soil compacted into a Cambridge type simple shear apparatus was assumed to be equal to K_o. Thus k increases from K_o to 1 asymptotically, for constant volume simple shear deformation. However if imposed simple shear strain induce soil volume compression, then k increases from K_o to K_p asymptotically and if imposed simple shear strain induce soil volume expansion, then k decreases from K_o to K_a asymptotically. For compacted Berea Red Sands, a hyperbolic representation of the change from K_o to K_p was adopted because of established evidence in Chapter Five of shear induced volume compression of Berea Sands in direct shear.

The stress ratios were determined from Mohr circles. This essentially entails the evaluation of horizontal and vertical stresses in the sample with respect to the stress tensors assuming uniform stress conditions exists in the entire sample during simple shear testing. For this assumption pure shear conditions exist in the specimen and thus failure was described by the Mohr Coulomb criteria. In simple shear devices, stress distribution is not uniform and thus the stress at the boundaries and the average simple shear stress in the sample was evaluated.

For a soil specimen of given dimension, the shear strength can be determined for any value of imposed strain, applied stress q and dry density γ. Eq. 6.10 is limited in that it cannot be used to determine the effect of changes in water content of the sample on shear strength. Because the shear strength of soils decreases with increasing water content, Eq. 6.10 represents the average shear stress of the dry sample for given values of applied stress q and dry density γ was evaluated.

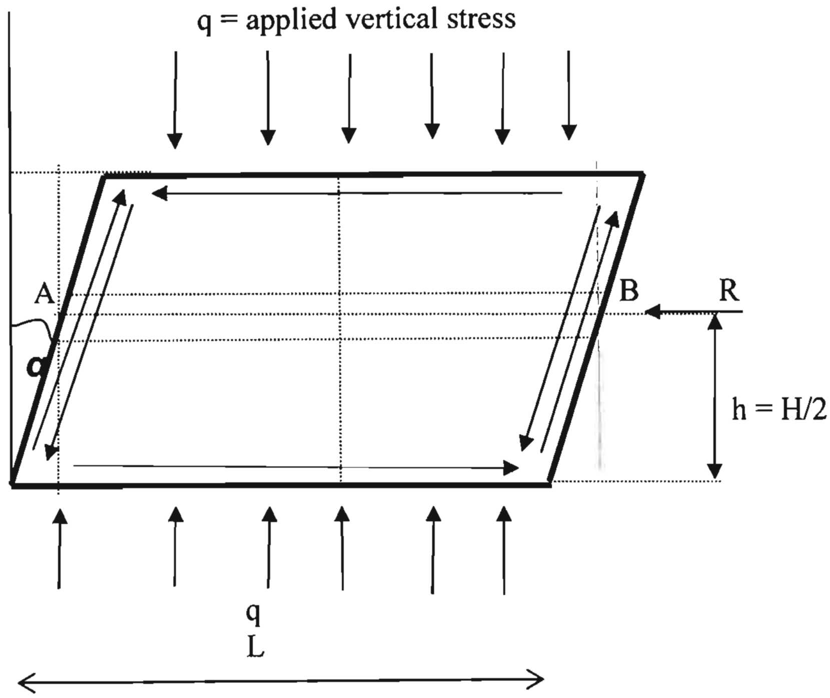


Fig 6.1A Schematic illustration of applied stresses, direction of shear stresses at the boundaries and reaction to imposed simple shear strain in a specimen. Here wall friction is significant to stress strain behavior

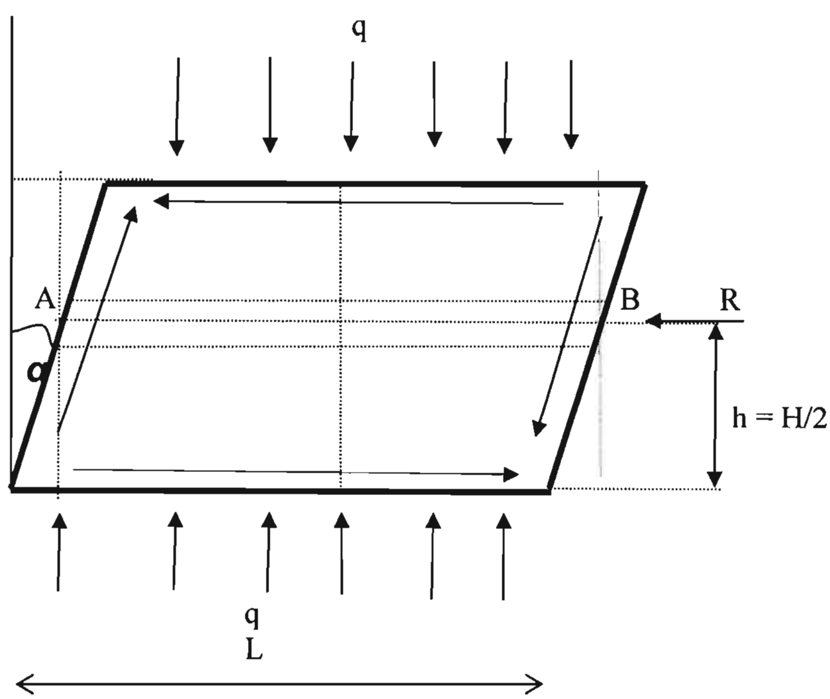


Fig 6.1A Schematic illustration of applied stresses, direction of shear stresses at the boundaries and reaction to imposed simple shear strain in a specimen. Condition at the core of the specimen where pure shear can be assumed to prevail.

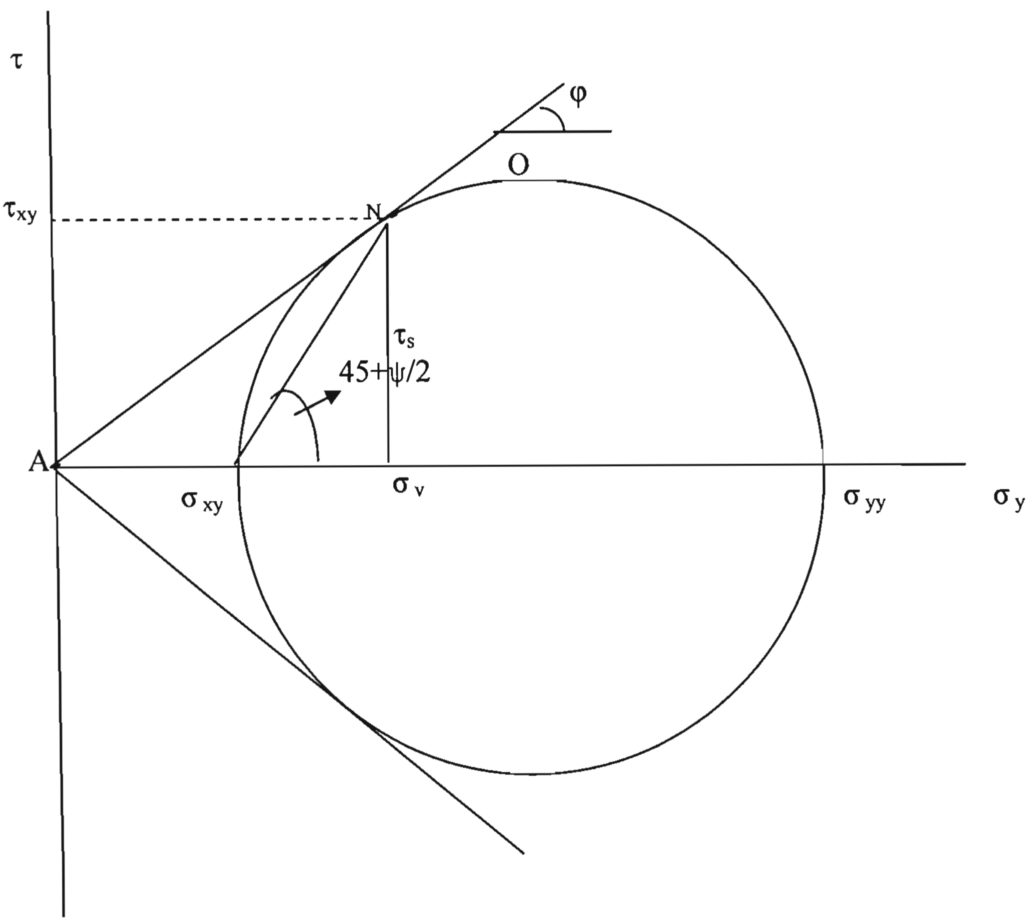


Fig 6.2 Mohr circle for simple shear state of stresses in which the horizontal plane is a plane of maximum stress obliquity i.e. Strength is related to tan ø' on the assumption of pure shear condition at the core of the samples and away from the specimen boundaries

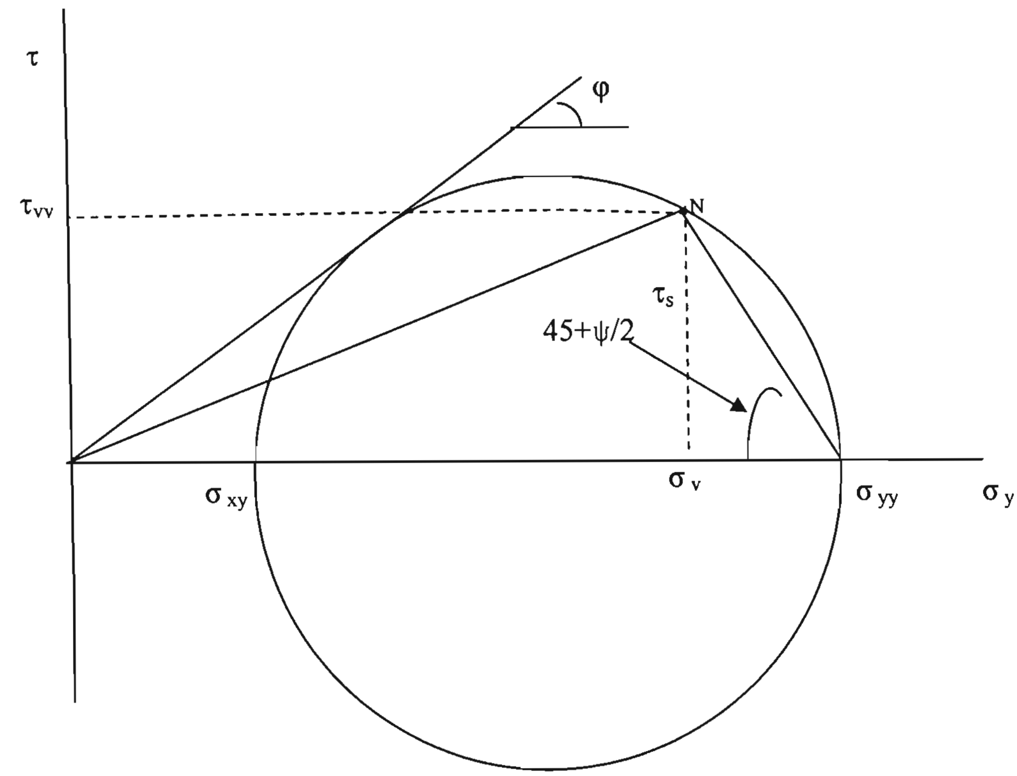


Fig 6.3 Mohr circle for simple shear state of stresses for vertical failure planes associated with rigid body rotation strength is related to $\sin\varphi'$ when $\psi = 0$ and $\dfrac{\sin\phi\cos\phi}{1+\sin^2\phi}$ when $\psi = \varphi$ on the assumption of pure shear condition at the core of the samples and away from the specimen boundaries

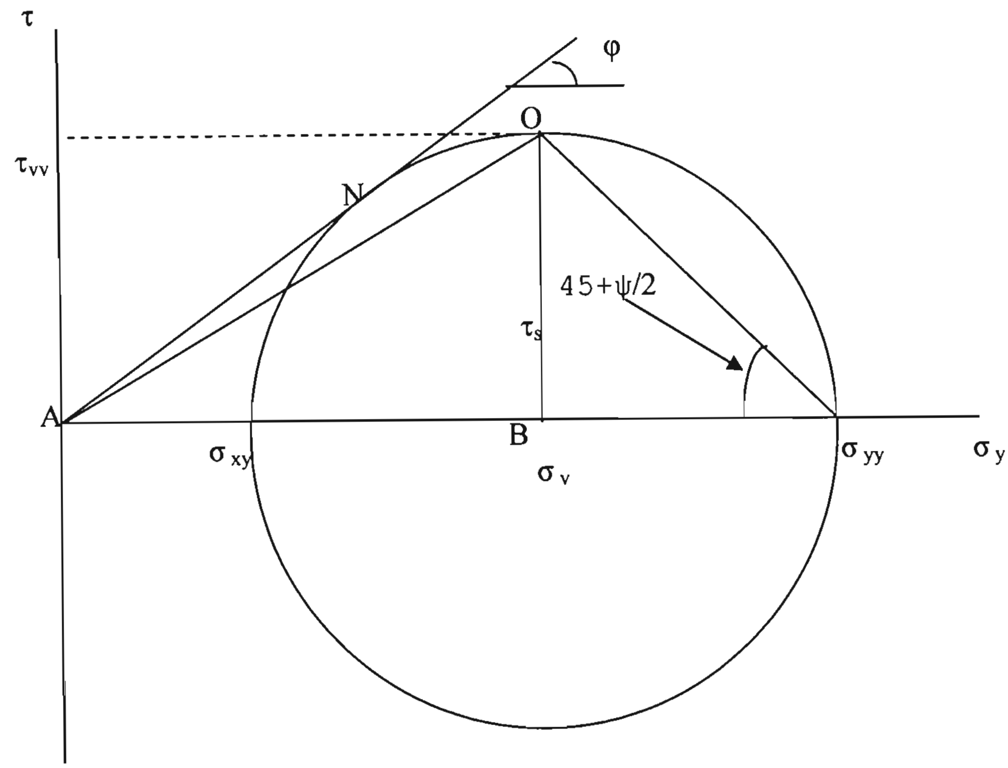


Fig 6.4 Mohr circle for simple shear state of stresses in which the horizontal plane is a plane of maximum shear stress, strength is related to $\frac{1}{2}\sqrt{(1+\frac{\sigma_x}{\sigma_y})^2 \sin^2\phi - (1-\frac{\sigma_x}{\sigma_y})^2}$ on the assumption of pure shear condition

at the core of the samples and away from the specimen boundaries

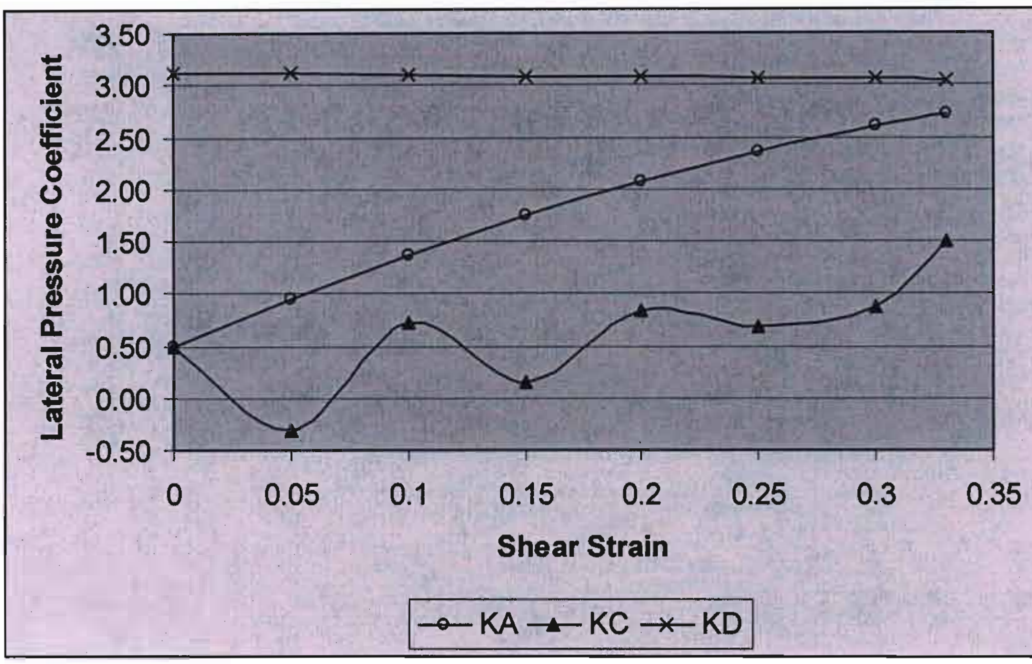


Fig 6.5; Curves showing the change in lateral pressure coefficients represented by Equations 6.6, 6.8 and 6.9 due to increase in shear strain. KA is Equation 6.6; KC is Equation 6.8 and KD is Equation 6.9.

Equation 6.7 the hyperbolic equation adopted for analysis of simple shear stress conditions in the Cambridge type device is shown in Fig 6.6

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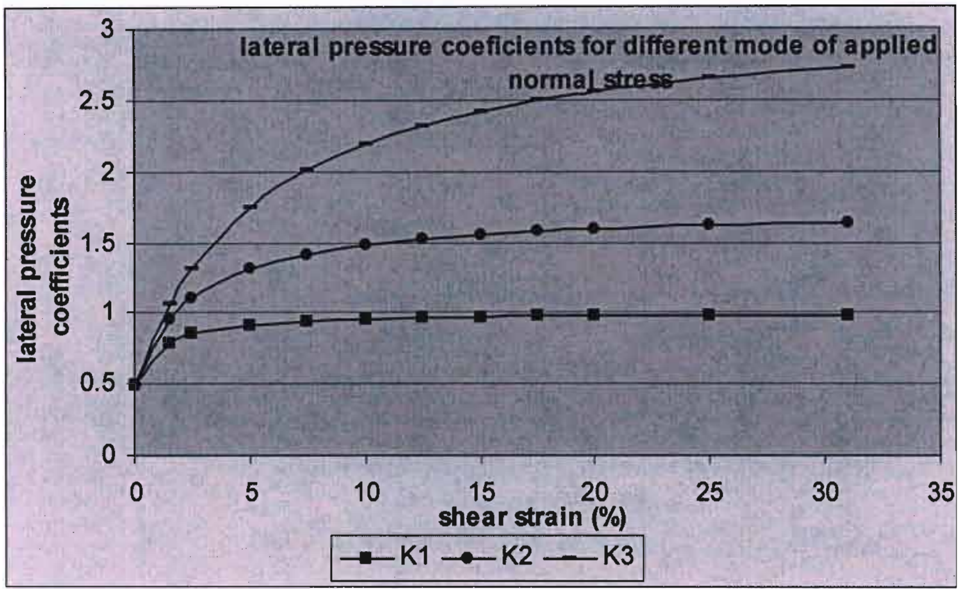


Fig 6.6; Lateral pressure coefficient for different mode of simple shear failure established from KB i.e. equation 6.7

K1 represents the change from K_o to the asymptotic value of 1 due to increase in imposed strain on the sample.
K2 represents the change from K_o to the asymptotic value of Km due to increase in imposed strain on the sample.
K3 represents the change from K_o to the asymptotic value of K_p due to increase in imposed strain on the sample.

CHAPTER SEVEN

A NEW SIMPLE SHEAR APPARATUS FOR COMPACTED SANDS.

7.1 INTRODUCTION

Details of a new apparatus, designed and developed to impose two dimensional simple shear strain on compacted sandy soils while also permitting the measurement of volume change during shear are presented in this chapter. The dimension of the apparatus was selected taking cognizance of the ratio of average particle size to minimum height of specimen and the limiting height to length ratio of the specimen within which simple shear deformation is permissible.

Two models of the simple shear apparatus were developed and evaluated. Photographs of compacted samples before and after shear deformation were compared.

7.2 PRIMARY OBJECTIVES OF THE NEW SIMPLE SHEAR APPARATUS

In order to obtain relevant and fairly reliable stress - strain and strength data that are representative of residual soils, taking cognizance of the effect of structural inhomogeneity on their engineering behavior, it will be necessary to compel the entire sample of a reasonable thickness to become a zone of shear failure. In the direct shear box, the fraction of the total volume of sample subject to shear stress deformation and thus can be described as a shear zone is indeterminate.

Pile shaft capacity is dependent on changes in normal stress, which for a vertically loaded pile configuration is in part dependent on the change in the thickness of a narrow band of soil at the pile - soil interface. Thus the device must be such that small changes in thickness under shear load can be measured within reasonable degree of accuracy. As the above condition cannot be guaranteed in the NGI type device without elaborate instrumentation, the new simple shear apparatus was modeled after the Cambridge apparatus.

While later Cambridge devices by Roscoe et al, (1967) and Airey et al, (1985) that were equipped with contact stress transducers and load cells were operationally more complex, they have not significantly improved our knowledge of stress strain and strength behavior of soils which were established using the first generation device by Roscoe (1953), and are not commercially available for routine testing. It is an objective of this research to design and develop a simple shear apparatus that is operationally simple for routine laboratory testing.

7.3 MATERIAL SELECTION

Stiff plates are more suitable for testing sandy soils and residual tropical soils than reinforced membranes. The reinforcing rings cannot ensure adequate boundary rigidity and the amount of lateral volume change of the membrane during test cannot be correctly predicted.

Fairly accurate measurement of vertical deformation is a primary objective of this research because of the need to relate shear induced vertical strain in specimen subject to simple shear strain to changes in lateral strain in the shear zone alongside the pile shaft. Stiff plates ensures that changes in sample volume can be directly correlated to changes in sample thickness and thus to changes in horizontal stress normal to the pile shaft.

The relatively lower stiffness of the steel wire - reinforced membrane of the NGI apparatus which was originally designed to investigate the shear strength of quick clays caused significant horizontal strain and bulging of the membrane when testing very stiff clays and sands (Vucetic et al, 1982).

Changes in sample volume cannot be directly correlated to changes in sample thickness and thus cannot be correlated to changes in horizontal stress normal to the pile shaft.

Based on the above reasons the new device was modeled after the Cambridge type apparatus.

7.4 SAMPLE HEIGHT AND SHEAR BOX SIZE

The average shear stress mobilized by the sample for unit width of sample, formulated in chapter six, is given below;

$$\frac{F\cos\alpha}{L} = \frac{K_u\gamma' H^2\cos\alpha}{6L} + \frac{K_u qH \cos\alpha}{2L} + \frac{K_u\gamma' H^2\sin^2\alpha}{6L} + \frac{K_u qH\sin^2\alpha}{2L} - \frac{K_o\gamma' H^2}{6L} - \frac{K_o qH}{2L} \quad ......7.1$$

The effect of the dimension i.e. the height/length ratio of the specimen on the mobilized simple shear stress was evaluated by considering the stress ratio at different failure conditions.

The applicable boundary conditions are as follows;

Condition A;

The stress ratio $\dfrac{(\frac{F\cos\alpha}{L})}{q}$ is a maximum = sin φ, when k is defined as

$$k = \frac{\sigma_x}{\sigma_y} = \frac{1+\sin^2\phi}{1-\sin^2\phi}$$

The above value of k represents constant volume simple shear deformation.
By substitution into the simple shear stress equation, Eq 7.1;

$$\frac{H}{L} = \frac{2\tan\phi}{k\cos\phi + k\sin^2\phi - K_o}$$

Condition B

The stress ratio $\dfrac{(\frac{F\cos\alpha}{L})}{q}$ is zero for $k = K_o$, i.e. specimen state before subjected to simple shear strain.

Substitution into the simple shear stress equation Eq 7.1, and solving for H/L gives;

$$\frac{H}{L} = 0$$

The range of H/L for which the specimen can deform in simple shear is given as

$$0 \le \frac{H}{L} \ge \frac{2\tan\phi}{k\cos\phi + k\sin^2\phi - K_o}$$

Thus the maximum height to length ratio of the specimen that permits simple shear deformations is dependent on the internal frictional angle of the sand specimen (Palmer and Rice 1973, Chowdhury 1978). Beyond the maximum height to length ratio, deformation of the specimen is no longer of simple shear mode. Simple shear strain involves uniform angular rotation of the vertical surfaces of speciemen that induces minimum stress concentrations at the corners of the speciemen. Simple shear strain imposed on samples of height to length ratio within the range exhibit minimum stress and strain non uniformity in the samples.

Samples of height to length ratio higher than the maximum will mobilize simple shear stresses and maximum shear stress that are lower than the values mobilized by the samples of dimension within the range given above. Palmer and Rice (1973) obtained similar expression for crack initiation and propagation in stiff soils and soft rocks in the long shear box. In their expression, K_o was replaced by the ratio of unconfined to direct shear strength.

One of the objectives of this research is to study the stress – strain behavior of Berea Red Sands subject to simple shear strain. Maud (1980) and Brink (1982) have reported values of effective angle of internal friction of Berea Sand of 31 degrees in triaxial shear.

Thus for Berea Sand, $K_o = 0.484$, $K_p = 3.126$, $K_a = 0.3198$ and $H/L = 0.69$

Samples subject to simple shear strain at the boundaries of a specimen will mobilize thesame value of shear stress if the length to height ratios H/L of the samples are less than 0.69, for soils with $\varphi = 31$. Beyond the ratio, the deformation of the sample is not a simple shear mode of deformation. Chueng (1988) noted had that the max particle size of the soil dictates the minimum thickness of the test specimen, while Dyvik et al, (1988) observed that internal stress and strain uniformity is influenced by the height to length ratio of simple shear apparatus and the soil average particle size (D_{50}).

ASTM D 3080 specifies that the sample thickness appropriate for direct shear testing should be at least six times the maximum grain size of the soil, and not less than 12.5 mm. In addition the specimen diameter or width should be at least twice the thickness. Airey et al, (1985) and Jewell (1988) used samples with dimension H/L = 0.10. Roscoe (1953) used sample dimension H/L = 0.33 for dry cohesionless soils while Roscoe et al, (1968) used samples of dimension H/L = 0.10 for the development of cam clay model.

A square cross sectional area of 70 mm by 70 mm and internal height of 62 mm was selected for the new device. The dimension chosen give a maximum H/L ratio of 0.85. Samples of smaller H/L ratios can thus be tested and the effect of sample dimension on mobilized shear stress evaluated. The choice of base area was dictated by the need to accommodate the device within the existing structure of the conventional direct shear box apparatus.

The simple shear stress equation was evaluated using a soil specimen with base area of 70mm by 70mm, initial height of 30mm, dry density of 1400 kg /m^3 and applied normal stress of 100 kPa.

Fig. 7.1 shows the simple shear stress – strain curves for different k values and applied normal stress of 100 kPa. Fig. 7.2 shows the curves of mobilized simple shear stresses for different normal stresses applied on a sand specimen. Fig. 7.3 shows the simple shear stress curves for specimen of different dry densities subject to the same normal stress of 100 kPa. The curves shown in Figs. 7.1, 7.2 and 7.3 represent constant volume simple shear deformation achieved by correcting the simple shear curves with the computed vertical strain shown in fig 7.2.

The specimen indicated compressive vertical strain due to imposed simple shear strain. The vertical strain is determined by evaluating the change from a rectangular shaped vertical cross section of the sample to one with a vertical cross section of a parallelogram due to imposed simple shear strain. The change represents the additional height by which the sample must be extended during simple shear deformation to ensure constant volume deformation (Roscoe et al, 1967; Airey et al, 1985). This is achieved by replacing the sample height H by H/cosα.

Only one such curve is presented as the vertical strain shown is due to sample deformation imposed by the boundaries of the apparatus and also since this mode of deformation is independent of the magnitude of applied normal stress.

7.5 DEVELOPMENT OF A NEW SIMPLE SHEAR APPARATUS.

The mainframe of the new simple shear apparatus consists of a steel channel section and steel end flaps. On the underside of the base are fitted two ball bearings tracks, which accommodate the travel of the simple shear box, set in a standard, commercially available direct shear box frame.

The ends of the soil sample are confined by steel end flaps, which were designed to allow for shear strain of up to 45 degrees on either side of the central position.

One of the major objectives of the current research is the development of a device with insignificant end effects in samples subjected to simple shear strain. The end effects are the main cause of excessive stress concentrations at the ends of the soil samples subject to simple shear strain and are the primary reason for the uncertainty of stress strain data obtained from tests using rigid boundary simple shear apparatus.

A problem common to many Cambridge type simple shear devices is that the axis of rotation of the end plates is not exactly at the intersection of the faces of the end plates and the base. Ansell and Brown (1978) approached the problem by allowing the end steel plates to pivot about the axes of their rounded lower edges in semi cylindrical grooves in the base plate. Cylindrical shaped grooves were machined into each end of the rounded edge of the plate and these engage the with spring loaded axles mounted in the sides plates so as to position the rounded edges in the semi cylindrical slots cut in the end of the channel forming the lower platen.

The clearance between the concentric semi cylinders is such that it allows the end plates to pivot about their lower edges while maintaining a close fit at the corners of the specimen. Ansell and Brown (1978) suggested that for the testing of dry granular soils in the Cambridge type apparatus, the end effects could be reduced by shaping the base of the end flaps into locating cones, which are fitted into cylindrical grove at the base of the apparatus. They however noted that the coincidence between the axes of the rotation of the end plates and the corners of the soil sample was not exact.

The above method of mounting the end plates is simpler than the approach adopted by Roscoe (1953), whose device used external hinges to bring the effective axes of rotation of the end plates into close coincidence with the corners of the specimen. Roscoe (1953) suggested that the end effect could be reduced by the use of external hinges connected to the end plates, while the inner walls of the shear box is covered by a 0.5mm thick lubricated membrane.

Both Roscoe (1953) and Ansell and Brown (1979) acknowledged that the coincidence was not exact in their systems.

In the new apparatus, the above problem was overcome by using a bearing centered on the intersection of the faces of the end plates and the base, the bearing being housed in circular sidewall sockets. Clearance was provided beneath the end plates to allow sufficient rotation to take place. The arrangement shown in Fig. 7.4a and Fig 7.4b provided exact coincidence and is much cheaper and simpler than the end plates fitting systems devised by Roscoe (1953) and Ansell and Brown (1979). The top edges of the two end plates were connected so that the two plates remained parallel and equidistant at any value of shear strain.

Another consideration for the minimization of end effects was the use of membranes and lubricants on the inner walls of the apparatus. The effect of membranes on stress – strain behavior of Berea Sands in simple shear will be investigated in the next chapter.

In order to ensure that the sample within the simple shear apparatus displace in a mode compatible with simple shear deformation, the top edges of the two end plates were connected with two rods. The arrangement ensures that the shear load applied to the apparatus is effectively transferred to the soil sample and points on the boundaries of the sample displace in a manner that the relative motion between the sample and the upper and lower plates is uniform.

In addition to the above, it is also necessary to ensure that adequate friction is developed between the sample and the upper and base plates. The technique suggested by Airey et al, (1985) was to coat the top and bottom platens with a thin layer of epoxy resin. Investigation of the effect of inadequate roughness of the top and bottom plates was conducted using samples of dense Ottawa Sands by Finn et al, (1971). Three types of surfaces were investigated; plain sand blasted hardened steel plates, plates with glued grains of Ottawa sand and ribbed steel plates. The investigation revealed no difference in the stress – strain behavior of Ottawa sand from series of tests conducted with the three surfaces. Finn et al, (1971) concluded that surface friction of the top and base plate does not influence the stress strain behavior of sands (Finn et al, 1971). The pressure pad of the new apparatus was however fitted with porous stone to provide the necessary friction and drainage of the sample.

Two types of the new apparatus were developed and are classified on the basis of mode of application of shear strain. The dimension of the simple shear apparatus is shown in Fig 7.5. In the two types of the new device, the resistance is measured by load cell connected to a stiff horizontal rod, which is in turn connected to the motorized worm gear. The stiff horizontal rod applies a horizontal force to the vertical edges of the box. The changes in the height of the sample as well as the horizontal displacement of the box are measured at the external boundaries by dial gauges.

In the type A device shown in plate 7.1 and Fig 7.6, a pair of cables is connected to the tip of the pressure pad. The other ends of the cables are connected to wall brackets and can be set at different heights depending on the selected thickness of the sample.

Normal pressure is applied to the sample through the pressure pad. The cable connected to the pressure pad prevents the top surface of the sample from moving during the lateral movement of the box, which is driven by the worm gear. The relative movement of the base in relation to the top surface of the sample imposed rotational motion on the end plates. The internal horizontal cross sectional area of the box is slightly larger than the area of the top platen thus there is no contact between the two parts. The resistance per unit horizontal area of the sample is a measure of the average shear stress on the sample due to the imposed rotation of the end plates. The rotation of the end plates is expressed in degrees equivalent to the ratio of the horizontal displacement of the top of the sample to the initial sample height.

The performance of type A device was evaluated by conducting trial tests and visual examination of photographs of soil specimens before and after the tests. Thin visible vertical threads were sealed to the sides of the sample before assembly in the simple shear device and visually inspected after each test. The change in alignment of the thread provided visual evidence of simple shear deformation.

Preliminary tests was conducted on Berea Red soils compacted into the simple shear box to dry densities of 1401 kg/m^3 – 1423 kg/m^3 at moisture content range of 4.3 % – 4.5 %. The selected samples height is 30mm and the normal stresses are 40 kPa and 400 kPa. The soil samples after tests shown in Plates 7.2 revealed that the samples did undergo simple shear deformation. The stress - strain curves are shown in Fig 7.7. The stress strain curves indicate that shear stresses increases with increase in end plate rotation. At end plates rotations above 5 degrees, shear stresses equal to the applied normal stresses are mobilized. There is no contact between the rotating end plates and the edges of the top platen during the straining process, so the abnormally high values of shear stress was not due to contact.

The abnormally high shear stresses measured at the boundary were evaluated by representing the loading configuration of the type A device in a force diagram shown in Fig 7.8. The force diagram indicates that the force inducing the rotation of the end flap is the resultant of the vertical stress on the sample made up the sample weight and the normal stress and the horizontal stress on sample top surface due to pressure pad/soil interface friction. The line of action of resultant stress falls on the end plate BC, and thus induces rotation of the plate. However because of the line of action of the resultant stress, the moment arm is much smaller than the height of the flap and thus can only induce limited rotation. Further lateral movement imposed by the worm gear set the resistance equal to the normal stress, since with increasing rotation, the line falls on the base and rotation of the flaps theoretically stops, as the moment arm now tends to zero. However as rotation of the flap is caused by the horizontal movement of the base against the non movement of the reaction block the rotation cannot in reality tend to zero.

Rotation of the end plate can only be effectively sustained by a loading configuration that results in a significant moment arm. That can be achieved by setting the cables at some angles above the horizontal plane shown in Fig 7.8. The associated problems here are that the applied normal stress are no longer uniformly distributed on the surface of the specimen and it may not be possible to apply constant normal stress of any specified magnitude to the specimen.

Although the design concept of the type A simple shear apparatus is elegant, the stress – strain curves imply that for all values of applied normal stress, soil samples will mobilize shear stresses equal to applied normal stresses. This finding is not consistent with established knowledge of soil behavior in simple shear, so another loading model, the type B, was developed.

The Type B apparatus applies the reaction to the right hand end plate via a horizontal rod as shown in Fig 7.9 and Plate 7.3. The end of the rod, which acts against the end plate, is free to slide on the end plate such that the point of action is at a fixed height above the sample base. The moment arm thus varies with the rotation of the end plates and is equal to Hcosa. The other end of the rod is fixed to a rigid vertical part of the direct shear box assembly. Evaluation of the sample is thus required to ascertain whether the sample deform in simple shear mode i.e. whether there is compatibility between the externally imposed strain and the mode of deformation of the sample.

Typical stress – strain curves from the type B device are shown in Fig 7.10 while photographs of samples sheared at different thickness are shown in plate 7.4 and plate 7.5. The type B device was adopted as the main testing device because the stress – strain curves indicate shear stress lower than the applied normal stress and while there is no indication of a peak shear stress value within the range of imposed strain applied, the curve indicated that an initially high value of shear modulus associated with the elastic range of deformation followed by a gradual decrease in modulus. Also a decreasing rate of increase in the shear stress is clearly indicated. In plate 7.4 and plate 7.5, the shape of the sample before and after deformation is different. Simple shear mode of deformation is clearly indicated in plate 7.5. The new device imposes simple shear strain on soil samples.

The stress strain curves determined from series of tests in the type B device are similar in shape to the shear stress – horizontal displacement curve determined from series of tests in the direct shear apparatus presented in chapter 5 and direct shear tests curves reported by Boey and Carter (1988) on residual carbonated sands.

It was noted by Vucetic and Lacasse (1982) and Airey et al (1988) that while the Cam clay model was developed from series of tests conducted using the First generation Cambridge simple shear device (1953) subsequent modification and advances in later Cambridge devices by Roscoe et al, (1967) and Airey et al, (1985) that were equipped with contact stress transducers and load cells were operationally more complex, they have not significantly improved our knowledge of stress strain and strength behavior of soils which were established using the first generation device by Roscoe (1953), and are not commercially available for routine testing

Luker (1988) and Boey and Carter (1988) have noted that pile load settlement curves determined from controlled direct shear tests predicted closely the load settlement curves determined from pile load test

Neither the shear stress values indicated in Figure 7.7 nor the shear stress values indicated in Figure 7.10 can be compared with the shear stress values of Figure 7.1 – 7.3 because of the differences in moulding water content. In particular the shear stress values indicated in Figure 7.7 were determined from series of test in the type A device and cannot be compared with the predicted shear stress values of Figure 7.1 – 7.3 as the type A device could not impose simple shear deformation on test samples. In addition the predicted simple shear stress results can best be compared with simple shear stress results for samples tested at zero moisture content as the simple shear equation cannot predict changes in shear stress due to changes in moisture content.

Similar consideration was made by Roscoe (1953) who compared the stress strain results of completely dry sands with numerical values predicted by 3D elasticity theory.

7.6 STRAIN RATE

The shear rate appropriate for a drained test depends on the drainage conditions of the apparatus and the permeability of the soil sample. In their typical configurations, both the shear box and the direct simple shear apparatus can only run the drained test. The nature of shear box test in many cases does not allow the sample to reach either a fully drained or fully undrained condition in a constant rate of shear tests (Blight 1997). However in many practical situations it is possible to select a shearing rate such that the deviation from the ideal condition is not significant. Based on the investigation by Gibson and Henkel (1954), Head (1982) recommended a time to failure, t_f, for drained direct shear tests of;

$t_f = 12.7t_{100}$ ....7.2.

where t_{100} is the time to 100% of primary consolidation. The value of t_{100} can be obtained by extrapolating the linear portion of the square root of the time plot of the consolidation phase of the test.

When t_f has been determined, the maximum permissible rate of shearing in a direct shear test can be estimated from:

Rate of shearing $< \delta_f/t_f$ 7.3

Where δ_f is the horizontal displacement of the shear box at failure. This value is not known a priori and has to be estimated. The top laminae of a soil of 30 mm high have to displace by 6.4 mm to impose a shear strain of 12 degrees. This is equivalent to 21 % strain. Standard laboratory shear strength tests are kept within strain of 20 %. Thus δ_f is taken as the horizontal displacement that gives 20 % strain, which is equal to 6.09 mm.

To estimate t_f, Berea Sand samples were compacted to dry densities of 1403 kg/m^3 – 1407 kg/m^3 at moisture contents of 15.0 % - 15.34 % in oedometer rings, the pots were filled with water and the samples compressed with normal pressures of 40 kPa and 400 kPa. Typical consolidation curves are shown in Fig 7.11. The estimated value of t_f corresponding to 100% primary consolidation is 10 minutes. Thus the maximum rate of shearing to prevent pore pressure effects is 1mm per 21minutes.

A much slower rate of I mm per 60 minutes was used for all the tests in the new device and direct shear box. Preliminary investigation of the effect of strain rate on mobilized shear stress in simple shear was conducted with samples compacted into the simple shear box to density of 1431 kg/m^3 – 1470 kg/m^3 dried to moisture content of 7.2% - 7.9% and tested at strain rates of 1mm/hr and 0.1mm/hr. The result shown fig 7.12 indicates that strain rate has no effect on the stress – strain behavior of low density Berea Sands in simpler shear.

The appropriate rate of shearing is dictated mainly by pore pressure effects, which may have a dominant influence with saturated specimen. For granitic residual soils with D_{50} varying from 0.2 to 2.0 mm, Chueng et al, (1988) found no significant differences in the strength parameters when the shear rate was varied from 0.007 mm to 0.6 mm per minute. Bjerrum and Landva (1966) recommended a shear rate of 1mm per 0.8 -1.2 hours for Norwegian brown clay after a set of preliminary tests.

7.7 SAMPLE COMPRESSION DUE TO SIMPLE SHEAR STRAIN.

A very important objective of this research is to ascertain whether samples of Berea Red Sands compacted to low density exhibit volume reduction due to imposed simple shear strain. A soil sample in the new apparatus will experience two phases of deformation. The first phase is the deformation imposed by the boundaries of the apparatus which deforms a rectangular shaped vertical section of the sample into a parallelogram. This phase of deformation is not associated with volume reduction but a reduction in the initial height of a sample from H to H$\cos\alpha$, where α is the shear strain. The second phase of deformation depends on the properties and initial state of the soil and is evaluated as any change from H$\cos\alpha$ as measured by the vertical dial guage. Thus volume dilation or contraction is referenced to H$\cos\alpha$ and not H.

To determine whether compacted samples of Berea Red Sands exhibit shear induced volume compression in simple shear, samples were compacted to 30 mm height, dry density of 1401

kg/m^3 - 1405 kg/m^3 and moisture content of 4.5 % - 4.8 % and were tested under constant normal stresses of 40 kPa - 400 kPa in the type B simple shear apparatus. The stress – strain curves are shown in Fig 7.13.

The samples exhibited shear induced volume compression within the range of applied normal stresses. The soil indicated total compression of 2.27 mm – 1.65 mm of which the shear induced compressions are 0.74 mm – 0.12 mm for applied normal stress from 40 kPa – 400 kPa. Thus shear induced volume compression decreases with increase in applied normal stress. Further evidence of shear induced volume reduction was presented in chapter 7.

7.8 ASSEMBLY FOR CONSTANT NORMAL STIFFNESS AND CONSTANT VOLUME TESTS

The constant stiffness and the constant volume simple shear tests were carried out with different test assembly. The constant stiffness assembly is shown in Fig 7.14 and Plate 7.6 while the constant volume assembly is shown in Fig 7.15 and Plate 7.7.

The assembly consists of two threaded vertical stiff rods connected to the mainframe at the two sides of the simple shear device. A thick reaction frame, with holes drilled at the ends is slotted to the top of the vertical rods and held in position by means of nuts. The springs are placed between the reaction frame and a pair of short pieces of rod below which an S - shape load cell is connected.

The initial normal stress is applied by the two nuts on top of the stiff reaction bar and is transmitted from the load cell to the samples through a set of ball bearings resting on top of a slightly modified pressure pad. Springs of different stiffness were used for the constant stiffness tests. The constant volume tests were run using a rigid assembly of rods where normal stresses are applied and controlled by screws.

7.9 SELECTION OF SPRING STIFFNESS.

The angle of friction between soil and pile was found to be consistent with that measured in simple shear tests (Airey et al 1982, Fleming et al, 1992). For the determination of pile capacity, using laboratory simple shear parameters, the simple shear tests must reflect the stress conditions along side a vertically loaded pile. For determination of pile capacity under constant normal stiffness condition, normal stiffness simple shear strength parameters are required, i.e. the samples are subjected to simple shear strains under constant normal stiffness that is equal to the stiffness of the soil surrounding the pile.

The normal stress on the pile shaft is related to the effective vertical stress at any depth by the coefficient K, K depends on the insitu earth pressure coefficient K_o, method of pile installation and the initial density of sand. Approximate value of K for sands recommended by Fleming et al (1992) is K = Nq/50. For Berea Red Sand with triaxial value of $\varphi = 31°$, K = 0.8. The range of normal stresses acting on pile shaft over embedment length up to 40 m can thus be was computed.

Jardine et al, (1985) suggested that the initial stiffness parameter G_{init} of the insitu soil can be determined at very low strain on the undisturbed soil sample if accurate measurement is made on the specimen. The G_{init} was however determined from the stress - strain curves of samples of compacted Berea Red Sands subjected to constant normal stresses within the range of 77 kPa and 260 kPa. The mobilized shear stress at shear strains of 0.33 % was used to determine G_{init}. The elastic modulus of the spring selected is equal to G_{init} . The range of strain of the springs when subjected to different magnitude of loads were also noted.

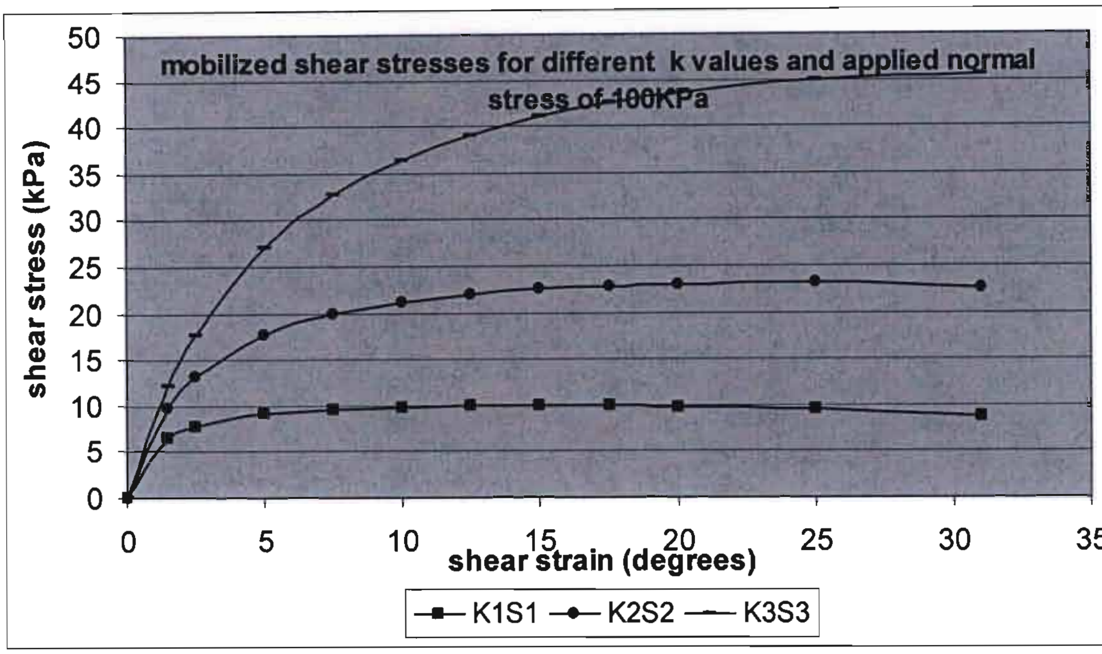
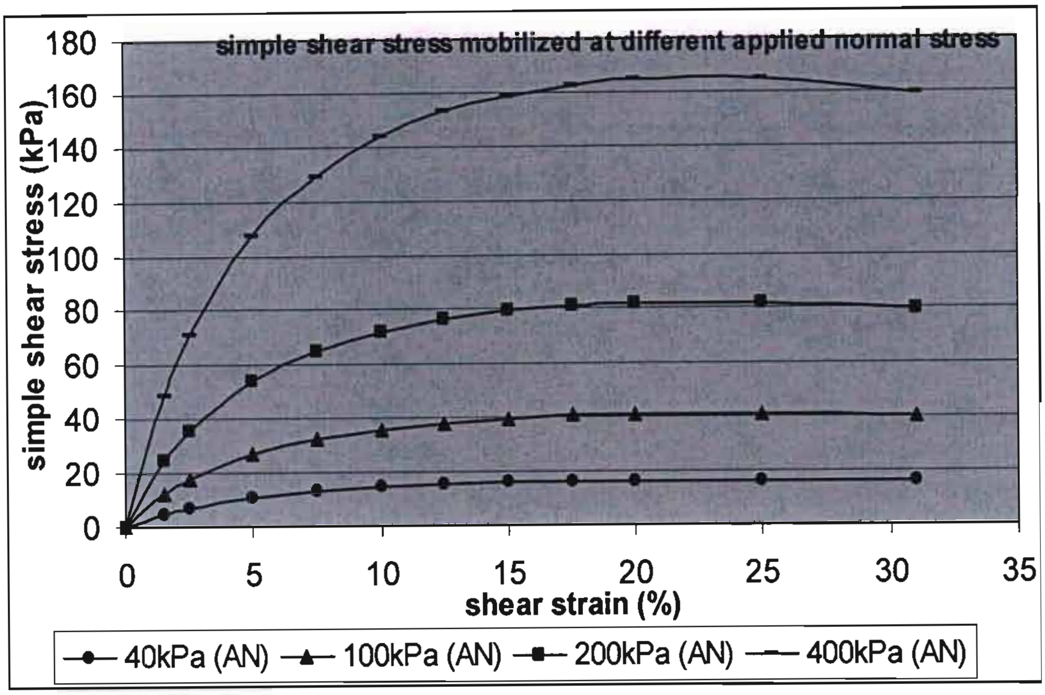


Fig. 7.1 simple shear stress-strain curves for different k values and applied normal stress of 100 kPa.

K1S1 represent shear stress mobilized by sample subject to normal stress of 100 kPa. The mode of failure is defined by transition from K_o to the asymptotic value of 1 due to increase in simple shear strain on the sample.

K2S2 represent shear stress mobilized by sample subject to normal stress of 100 kPa. The mode of failure is defined by transition from K_o to the asymptotic value of Km due to increase in simple shear strain imposed on the sample.

K3S3 represent shear stress mobilized by sample subject to normal stress of 100 kPa. The mode of is defined by transition from K_o to the asymptotic value of K_p due to increase in simple shear strain on the sample.



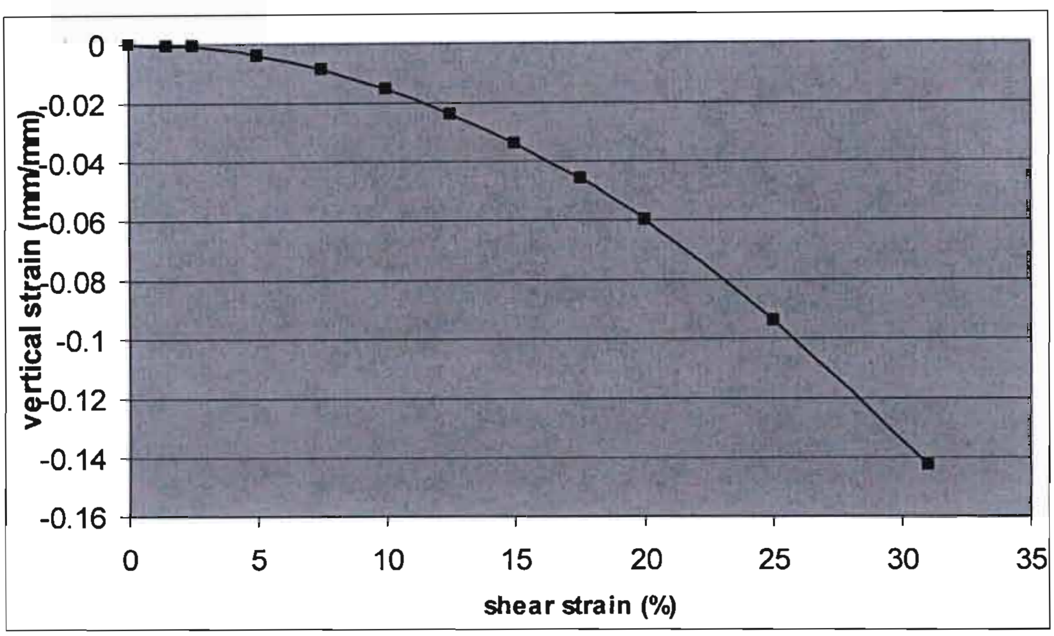


Fig. 7.2A: simple shear stress strain curves for different normal stresses applied on a sand specimen. Mode of failure is represented by transition from K_o to the asymptotic value of K_p due to increase in simple shear strain on the sample. The specimen indicated compressive vertical strain due to imposed simple shear strain.

The vertical strain represents the amount by which the sample must be extended during simple shear deformation to ensure constant volume of sample through out shearing duration. Only one such curve is presented as the vertical strain shown is due to simple shear strain imposed by the boundaries of the apparatus, and is independent of the applied normal stress.

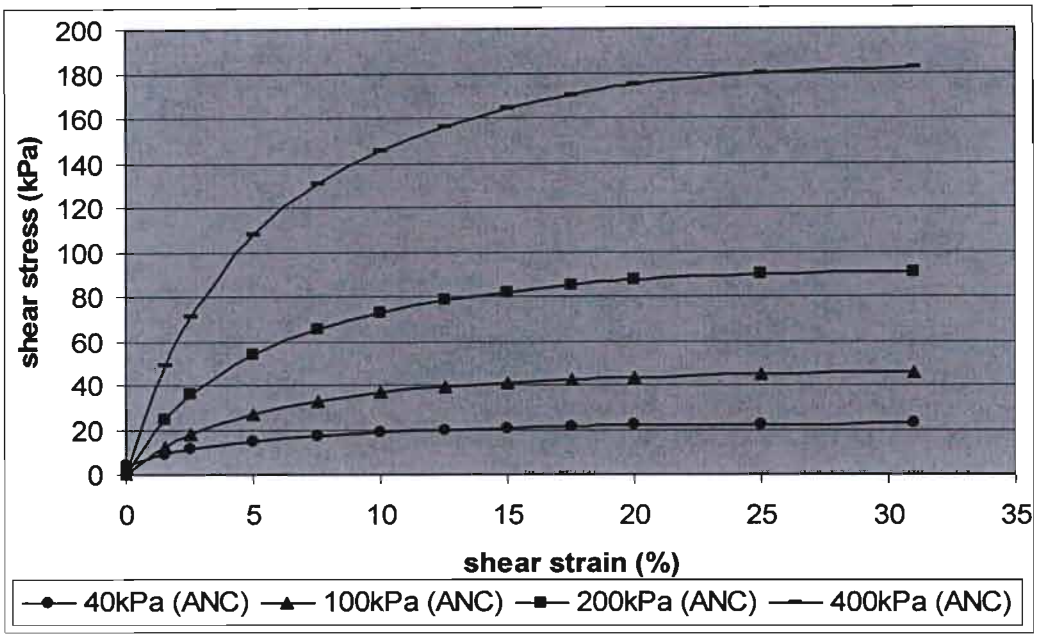


Fig. 7.2B: Corrected analytically predicted simple shear stress - strain curves (ANC) were determined by modifying stress – strain curves (AN) shown in fig 7.2A by replacing the sample height H with H/cos*a* to ensure constant volume deformation i.e. zero compression and zero dilation.

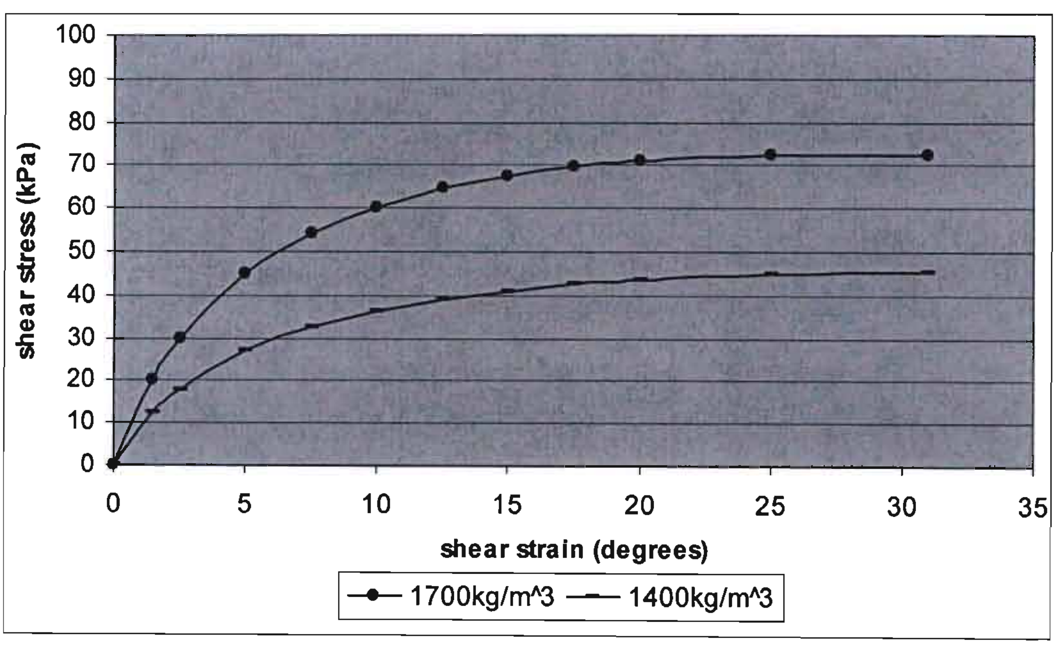


Fig. 7.3: Simple shear stresses for specimen of different dry densities subjected to the same normal stress of 100 kPa.

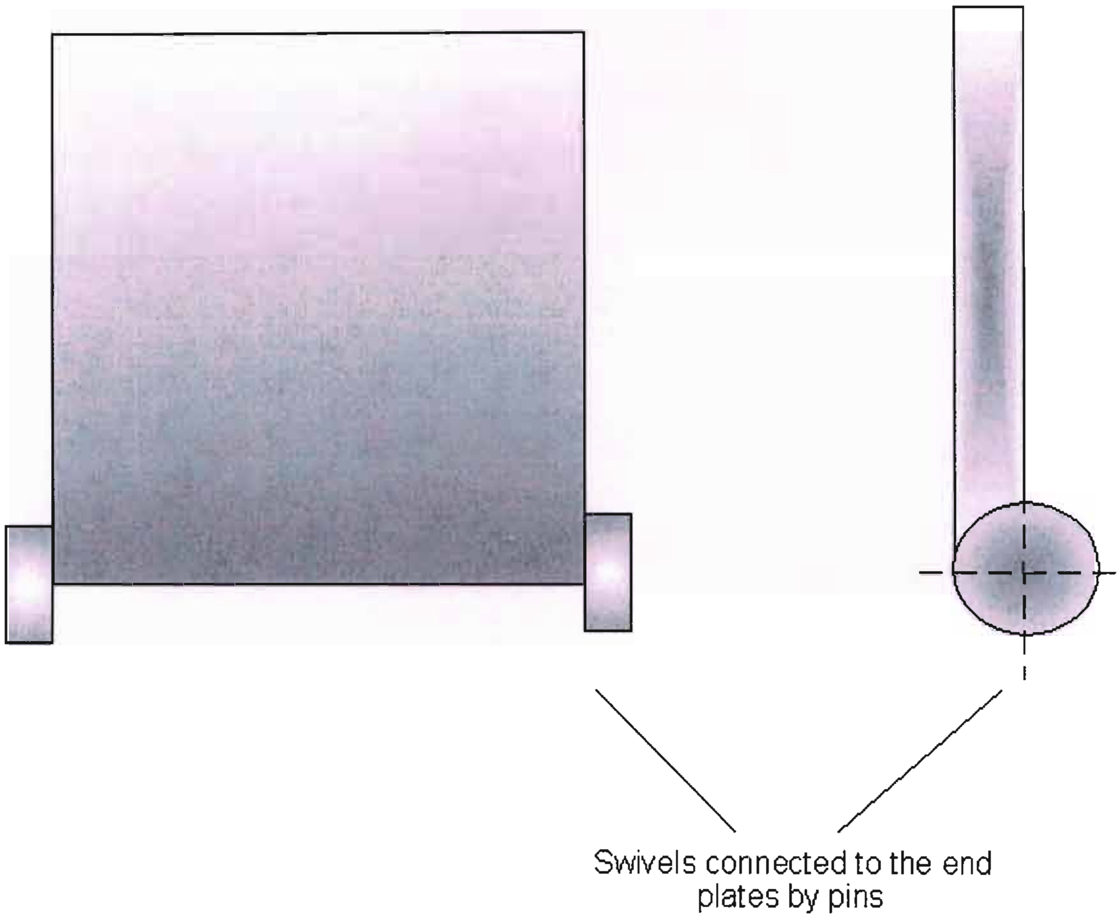


Fig 7.4a: Front Elevation End of plates

Fig 7.4b: Side Elevation of End Plates

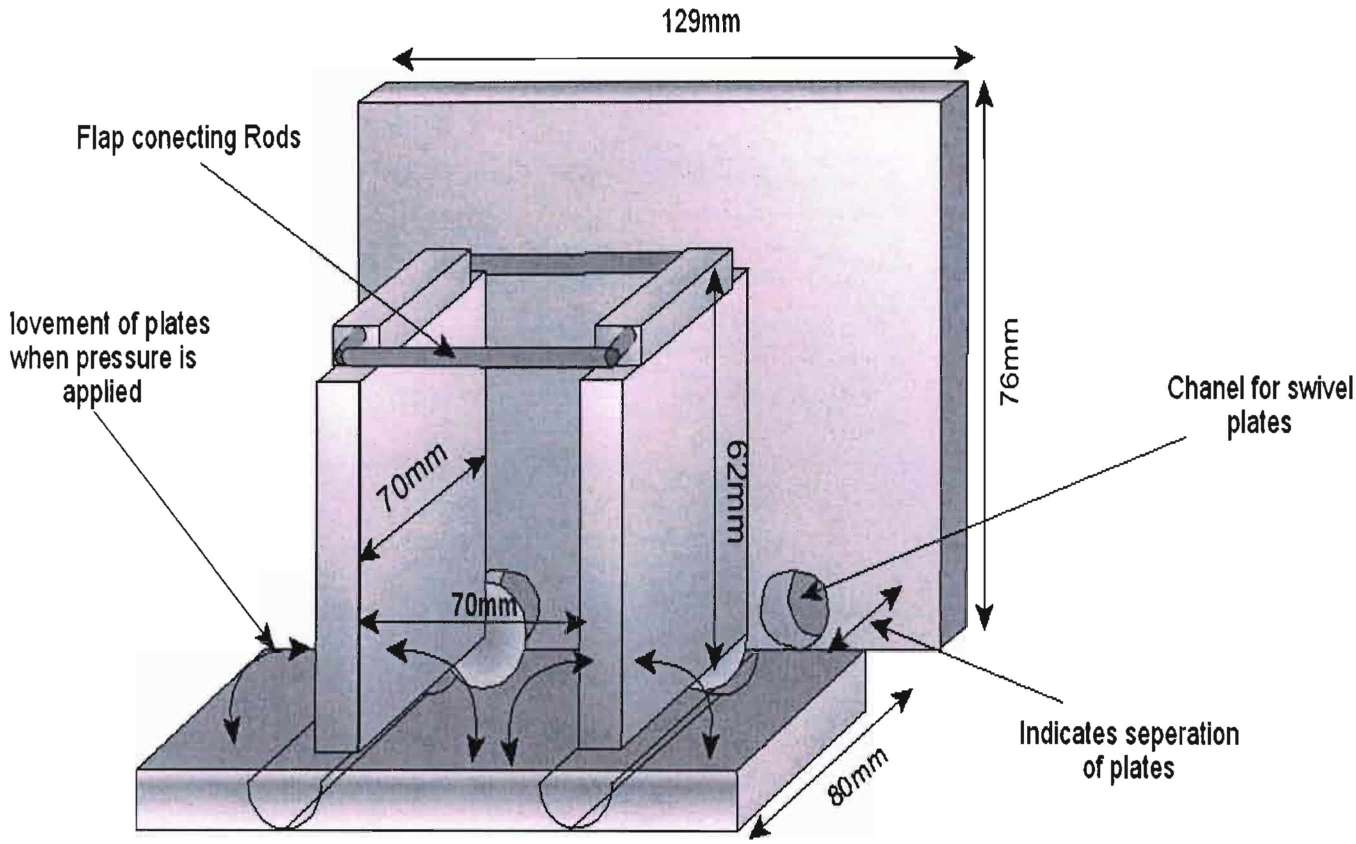


Fig 7.5: Dimension of the simple shear apparatus

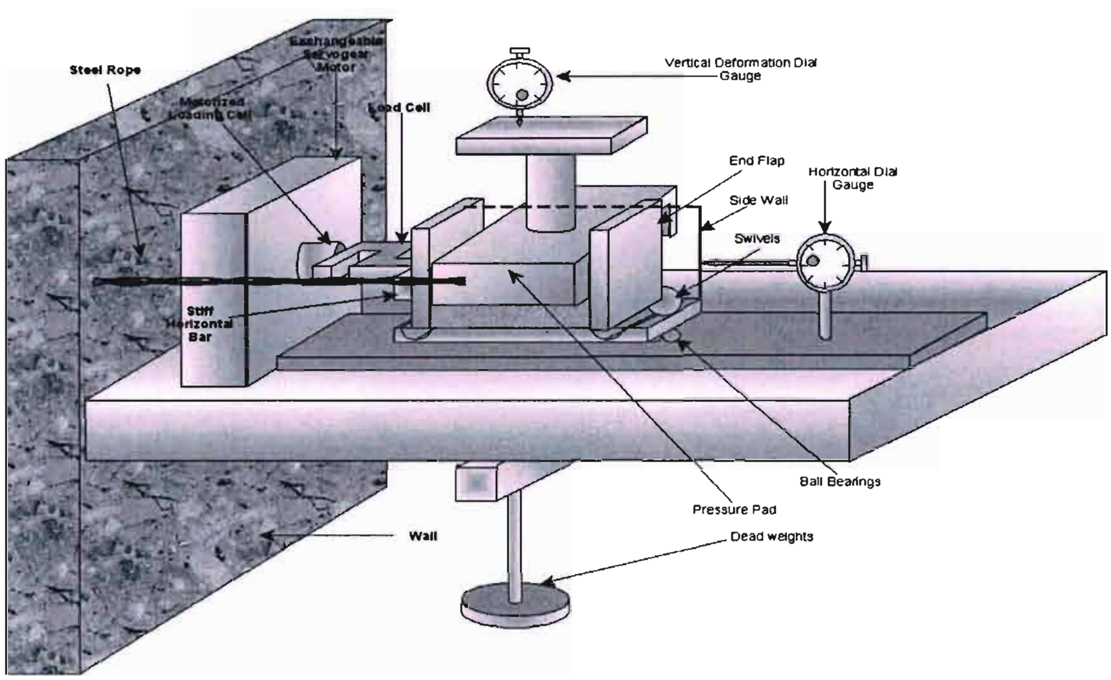
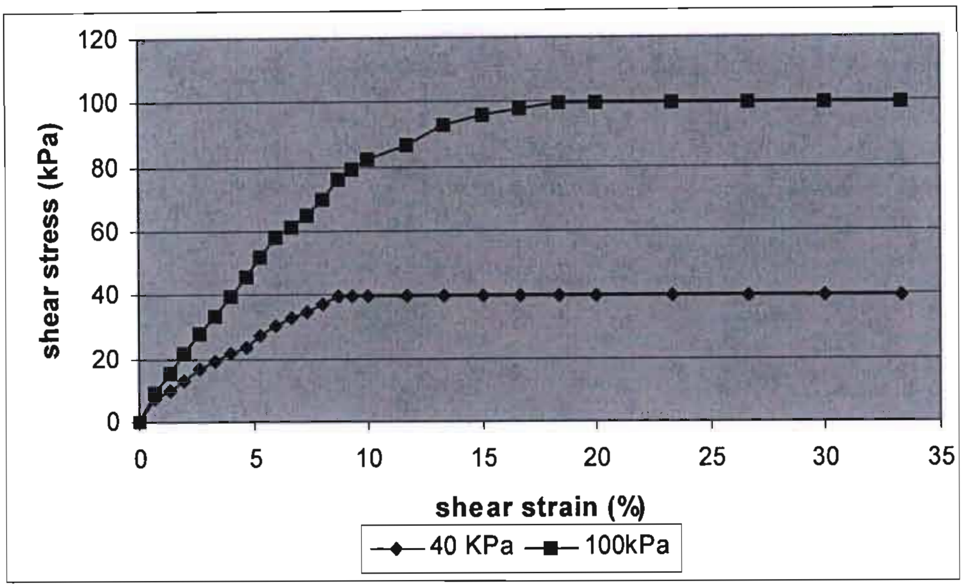


Fig. 7.6: The type A simple shear apparatus. Shear load is transferred through the top and base surface of the sample.



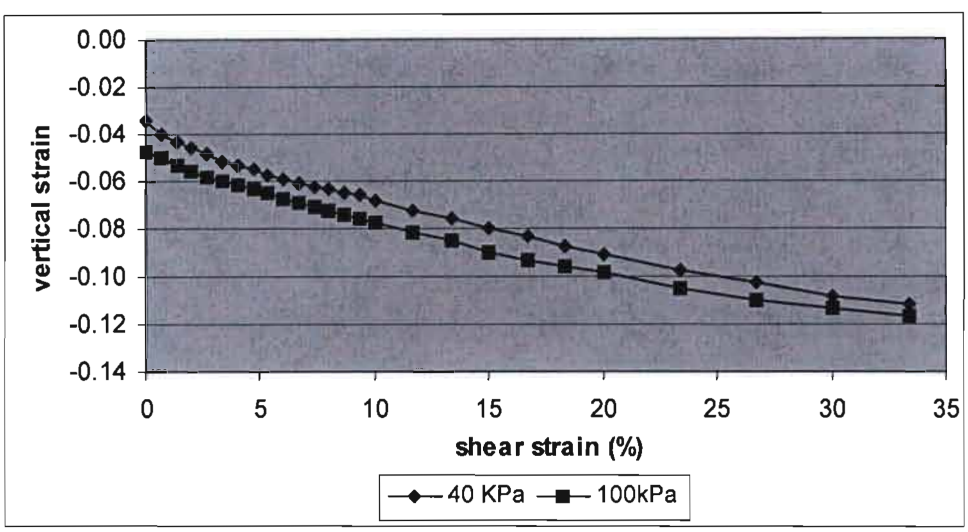


Fig 7.7; Stress – strain and vertical strain – shear strain curves of Berea Red Sands compacted to 30mm height, dry density of 1400 kg/m^3 and 7.5% moisture content and tested under constant normal stresses of 40kPa and 100kPa in the type A simple shear apparatus

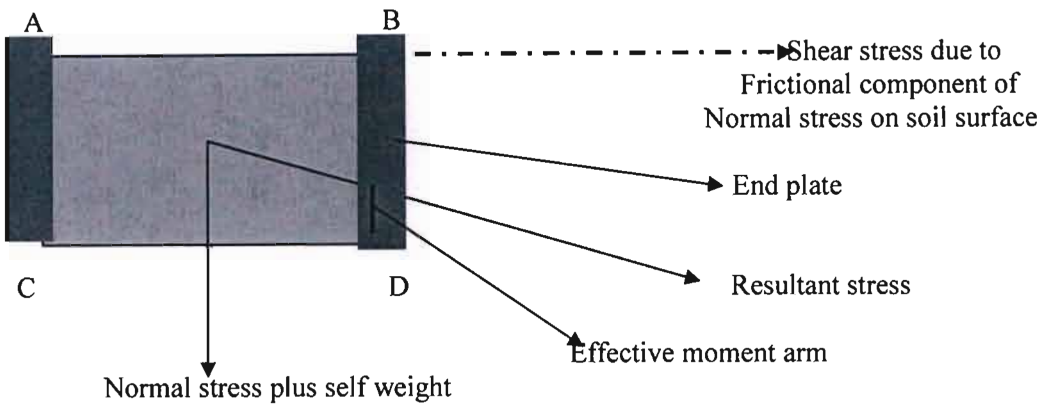


Fig 7.8: Loading configuration of the type A device

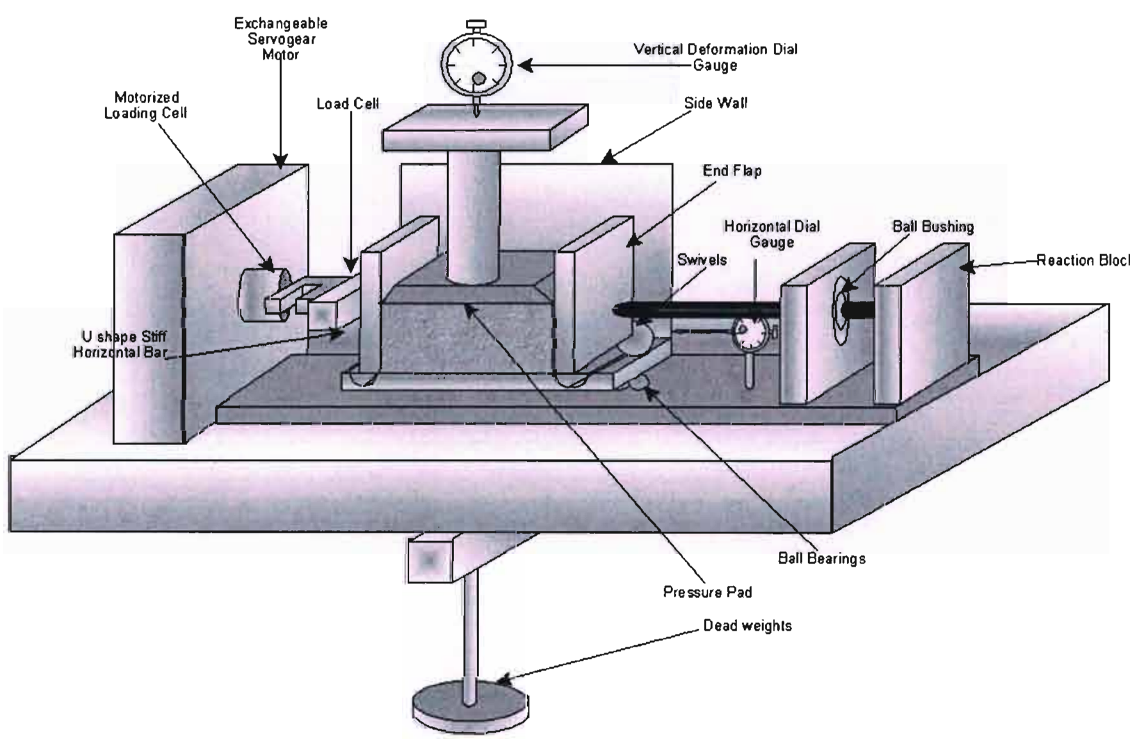


Fig 7.9: The type B simple shear apparatus

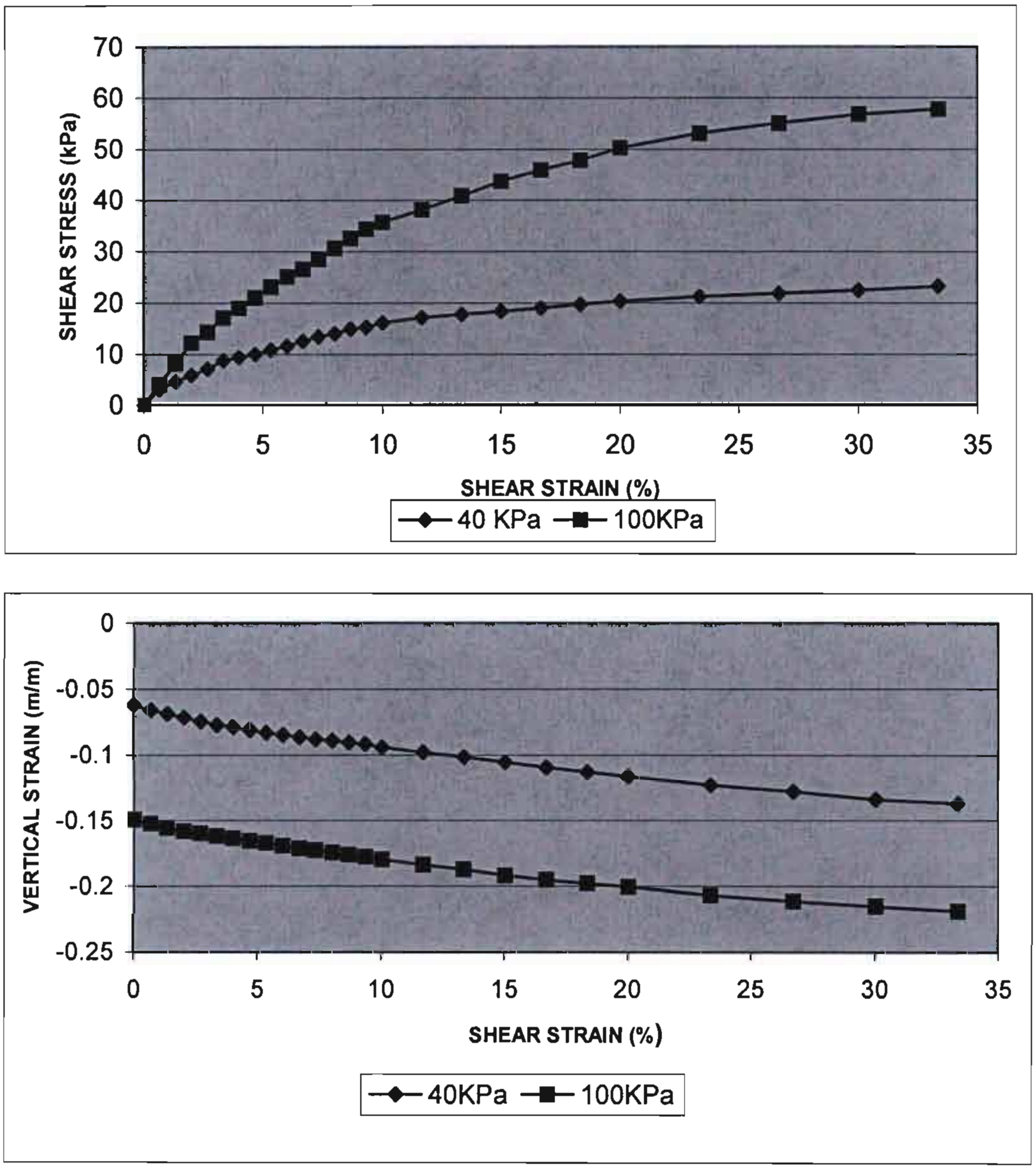


Fig. 7.10: Stress- strain curves of Berea Red Sands compacted to 30mm height, dry density of 1400 kg/m^3 and 7.5% moisture content and tested under constant normal stresses of 40 kPa and 100 kPa in the type B simple shear apparatus.

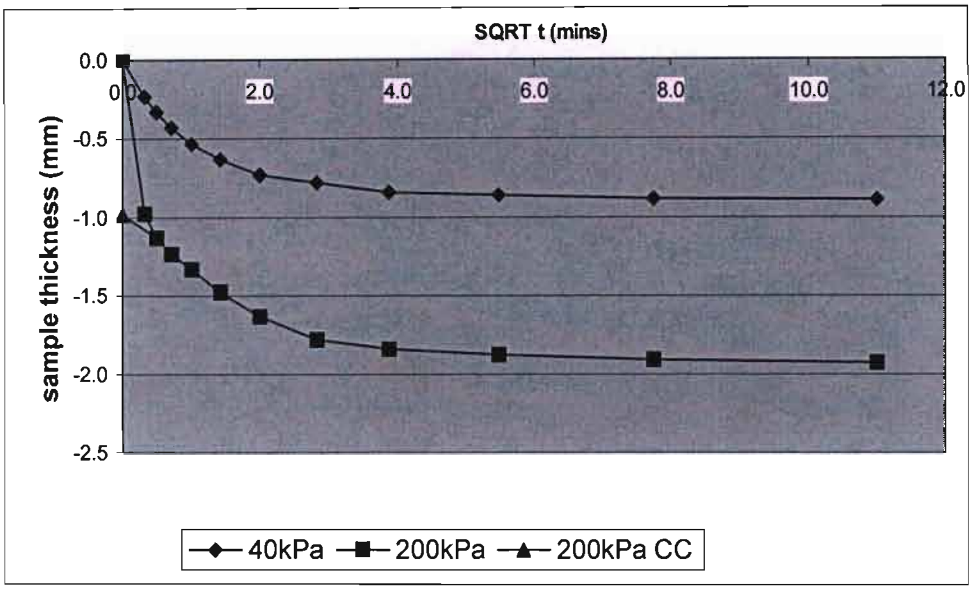
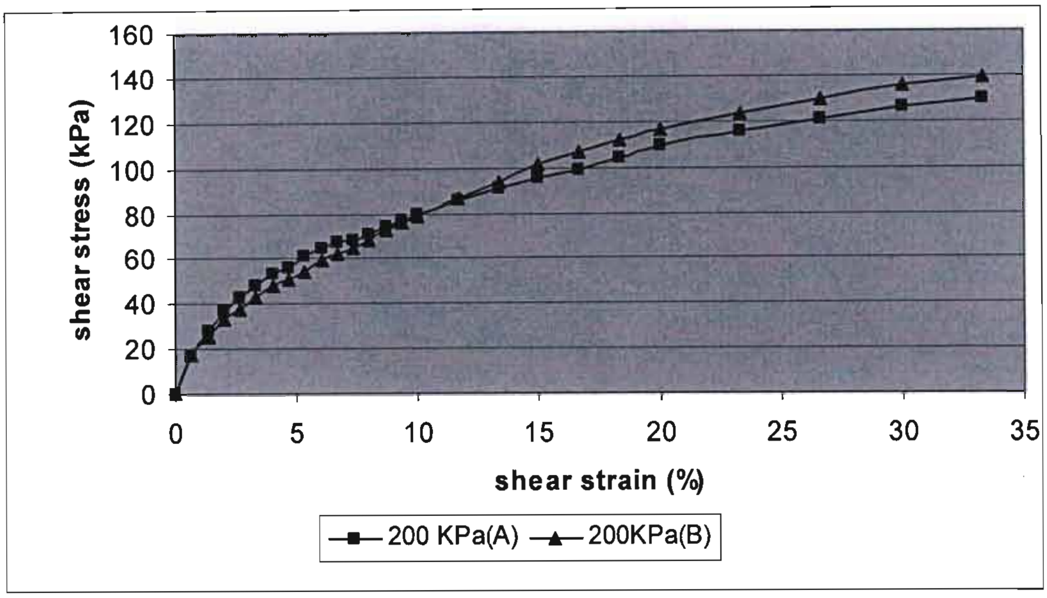


Fig 7.11: Compression tests of Berea Red Sands compacted to 1403 kg/m^3 at 15% moisture content. The samples were first inundated before compression to 40kPa and 200kPa respectively. The curve of the sample compressed to 200kPa was corrected and the final curve 200kPa CC, is similar to the 40kPa curve in size.



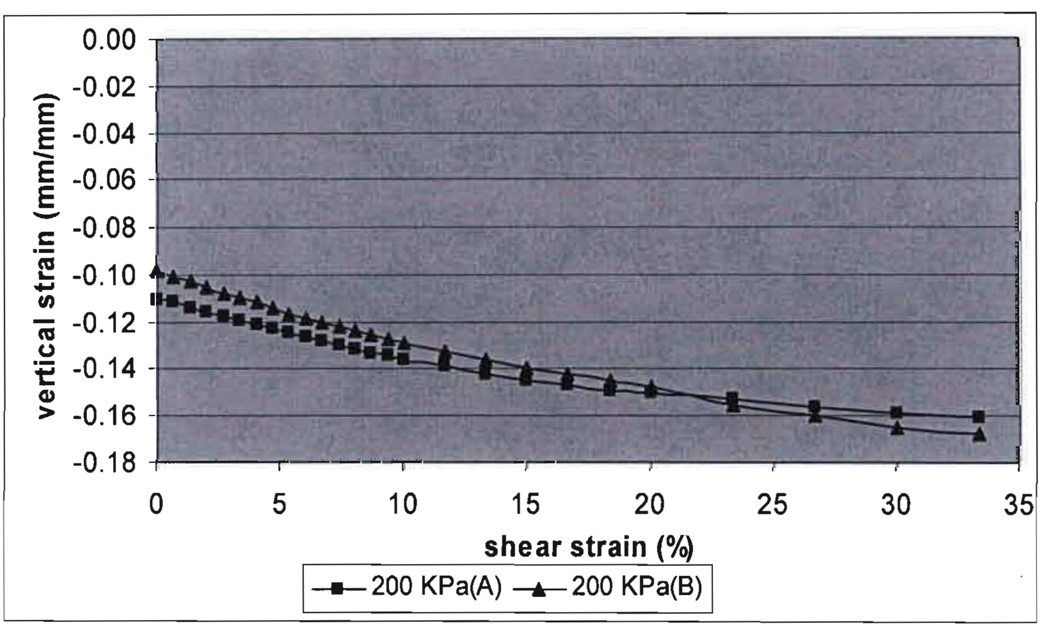


Fig 7.12 The effect of strain rate on stress – strain behavior in simple shear. The 200 KPa (A) represents the stress – strain curve for strain rate of 1mm/hr and 200KPa (B) represents the curve for strain rate of 0.1mm/hr

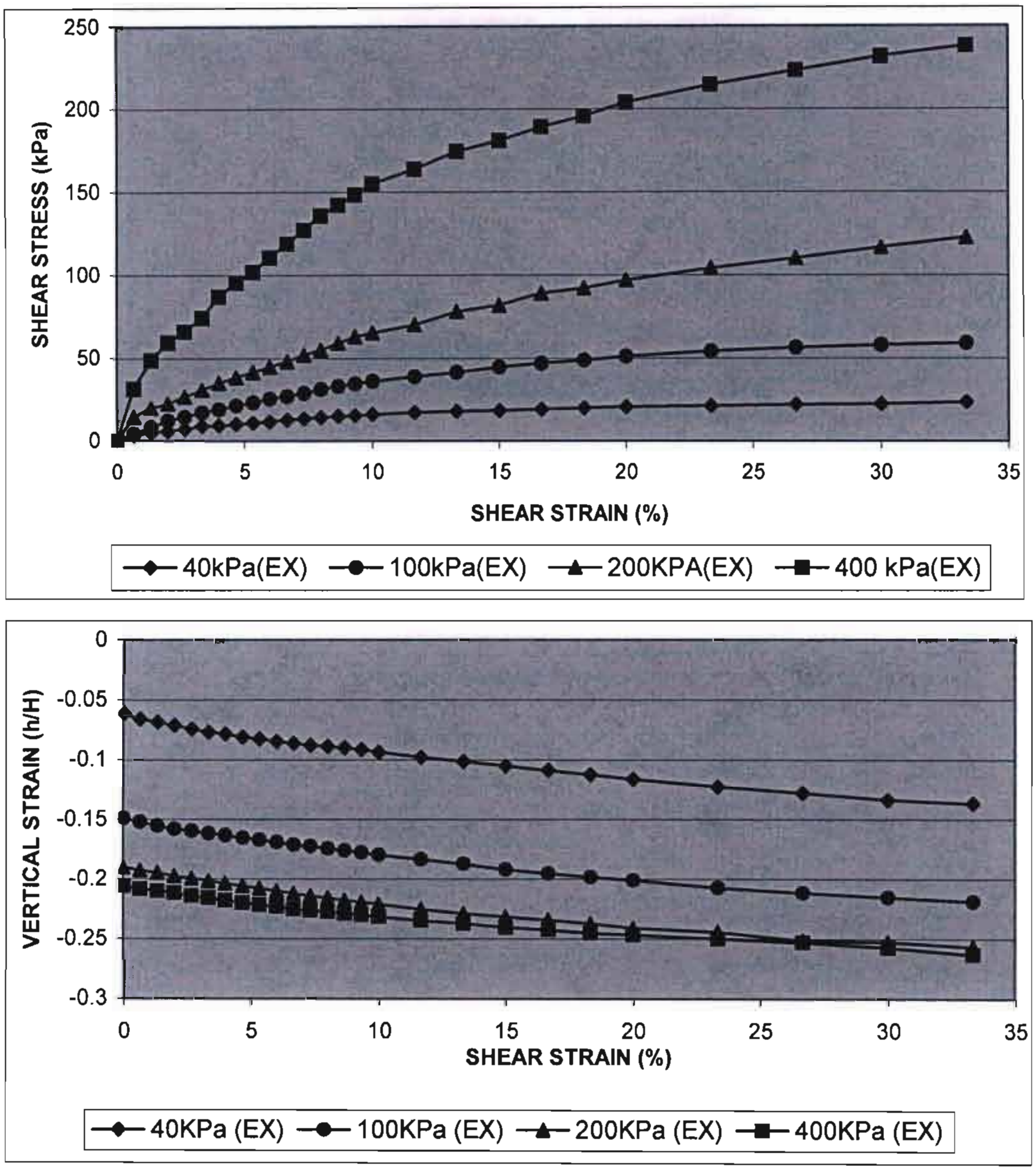
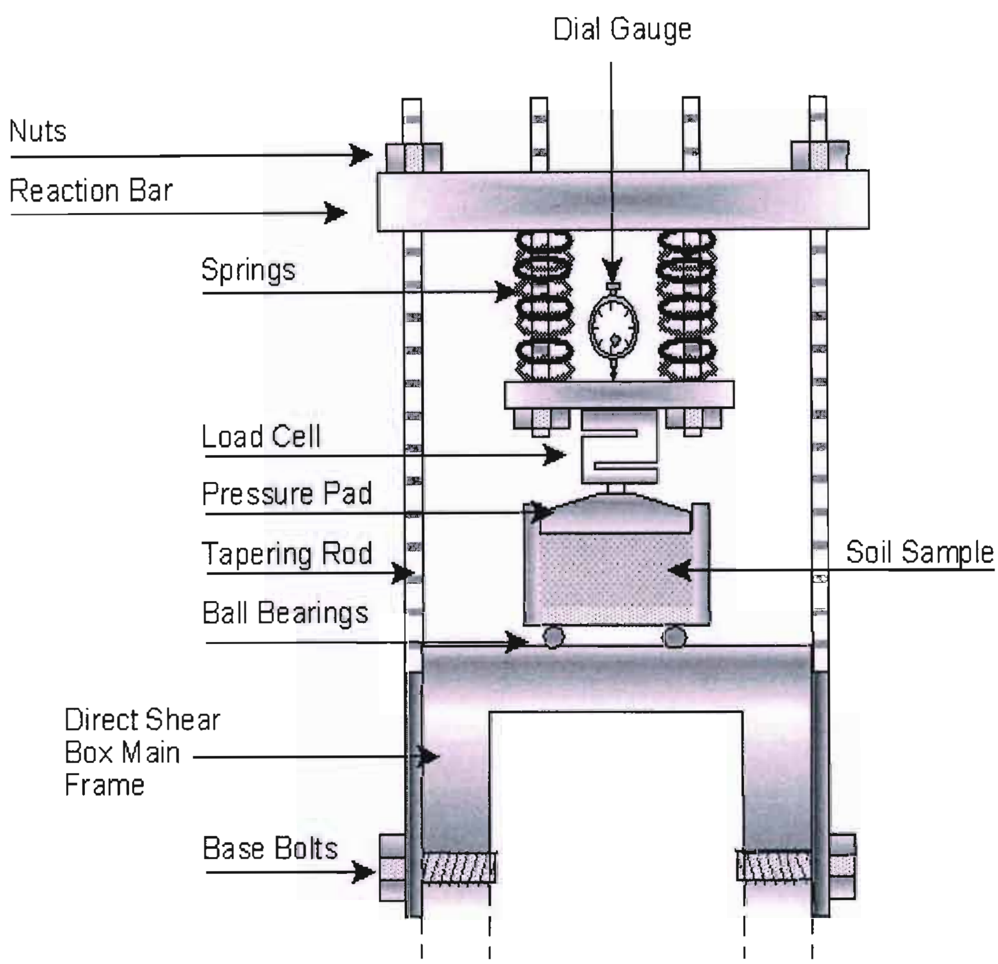


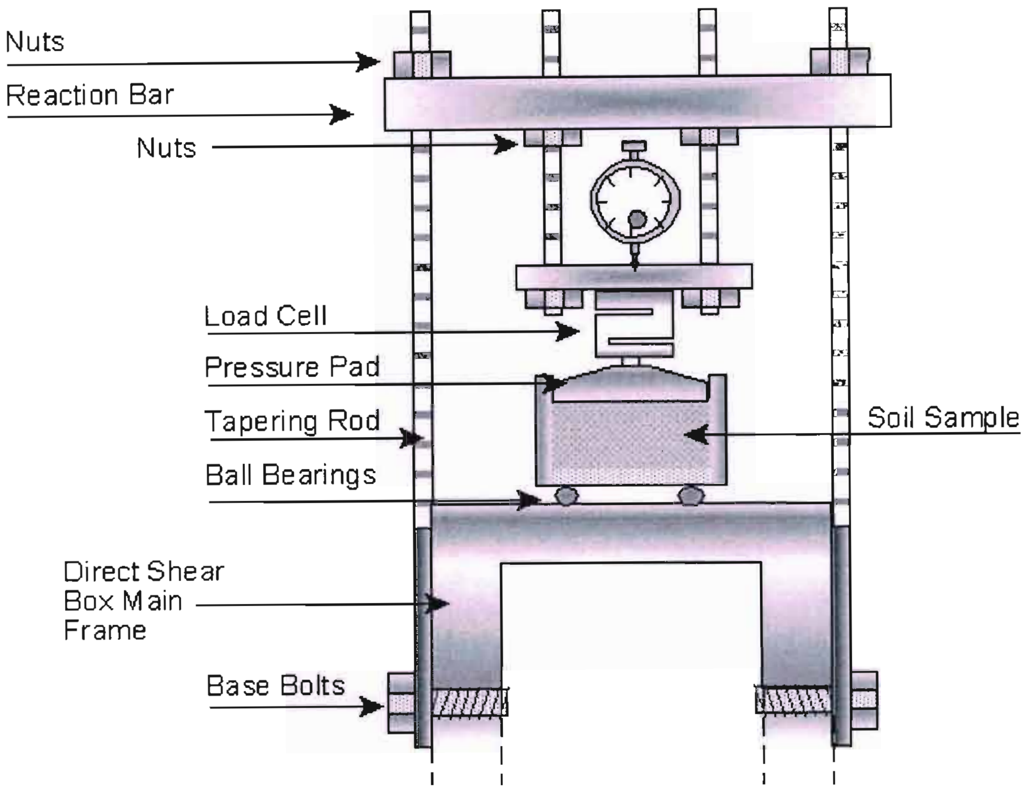
Fig. 7.13 Stress - strain curves of Berea Red Sands compacted to 30 mm height, dry density of 1401 kg/m^3 - 1405 kg/m^3 and 4.5 % - 4.8 % moisture content and tested under constant normal stresses of 40 kPa – 400 kPa in the type B simple shear apparatus.



For Constant Stiffness Tests

Fig 7.14 GENERAL ASSEMBLY FOR NORMAL STIFFNESS SIMPLE SHEAR TESTS

Laboratory tests at different normal stiffness conditions were conducted using springs of different stiffness constant. Different normal stresses were applied through the springs.



For Constant Volume tests

Fig 7.15 ASSEMBLY FOR CONSTANT VOLUME SIMPLE SHEAR TESTS

Constant volume during shear deformation was maintained by adjusting the nuts.

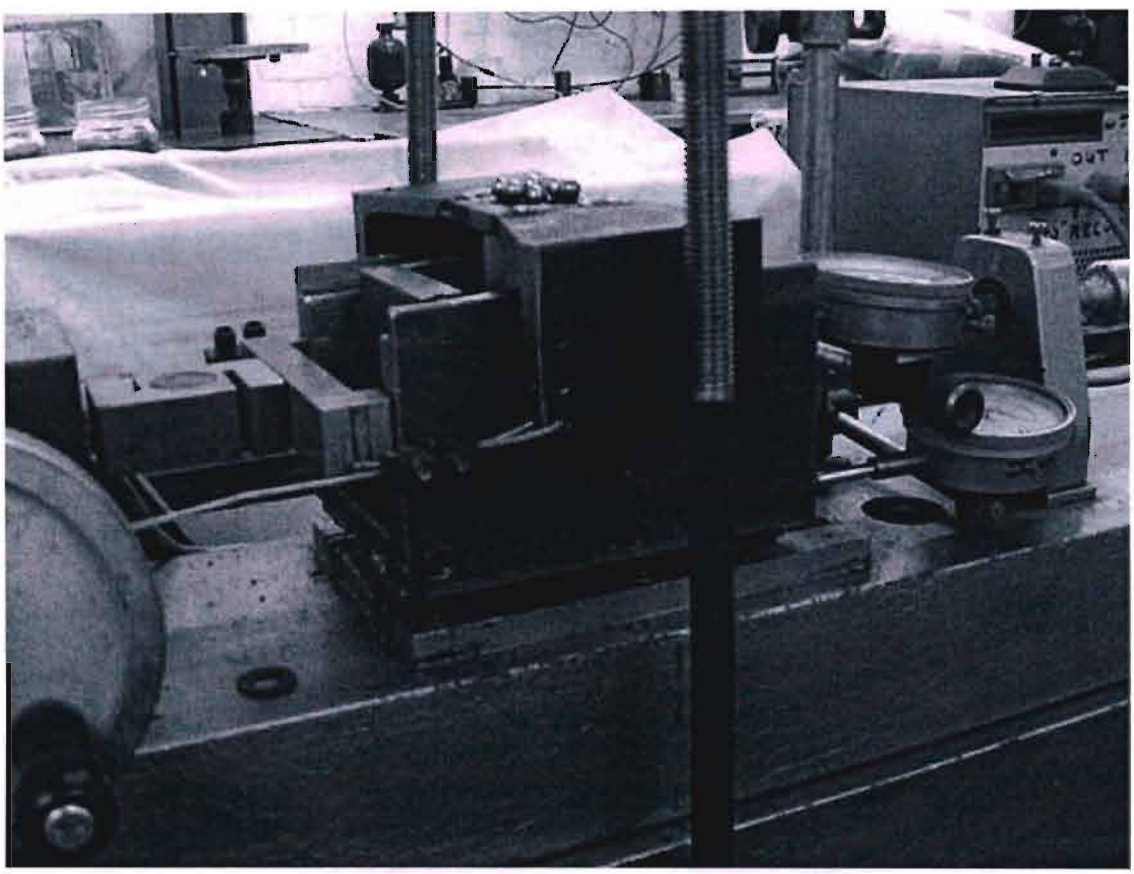
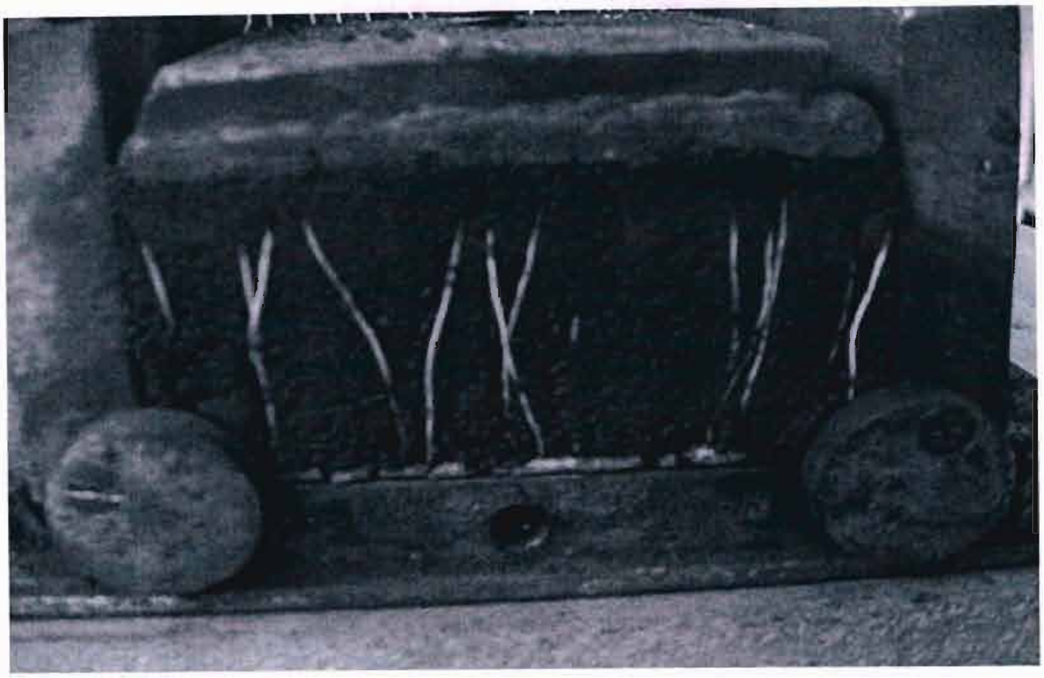


PLATE 7.1: TYPE – A SIMPLE SHEAR DEVICE (simple shear strain imposed on samples top surface).



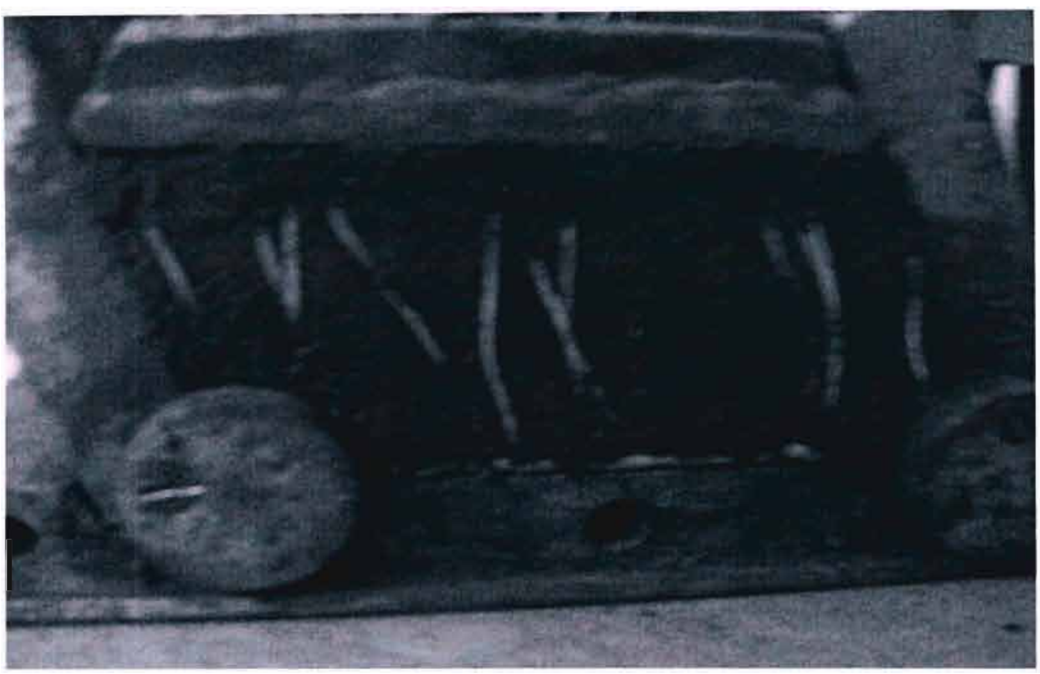


Plate 7.2 Sample of Berea Sands compacted to 30mm height, before and after testing under constant normal stress of 100 kPa in the type A simple shear apparatus. The end flaps move back to the initial position once the imposed lateral loads were removed. Thus this deformed was not sustained and was reexamined by the Type B loading conditions.with the same line grids.

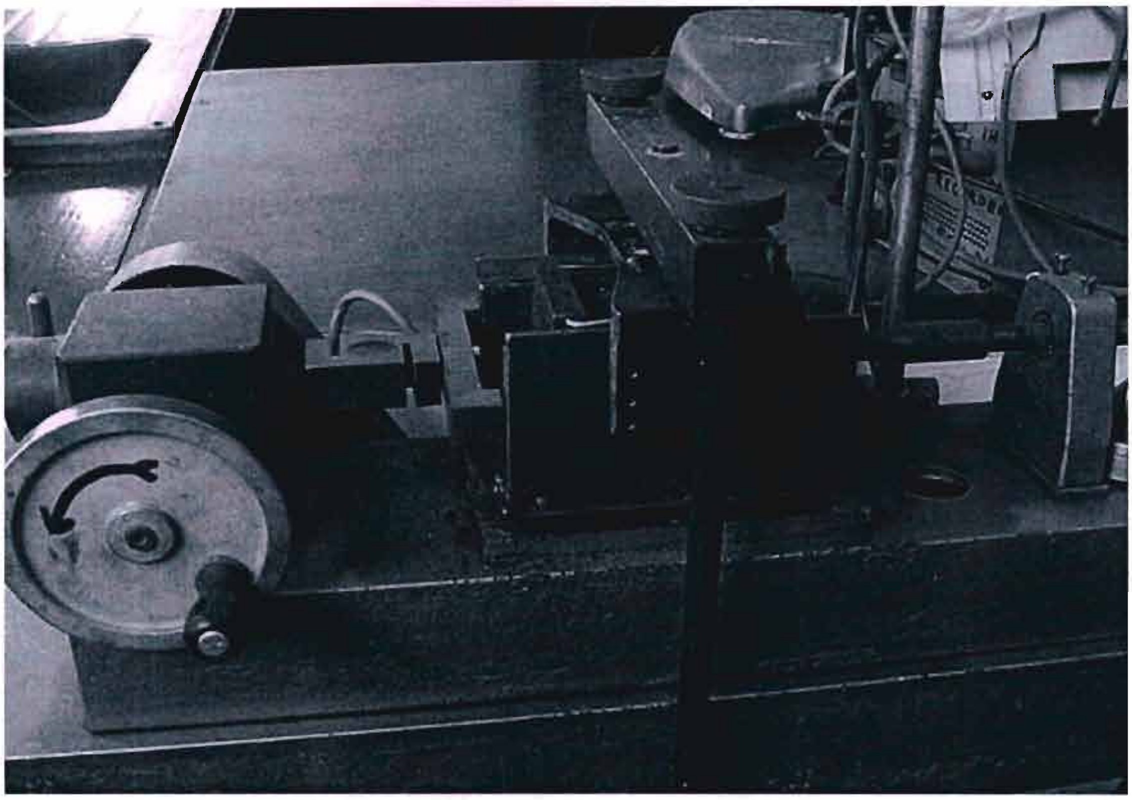


PLATE 7.3 Type – B simple shear device (shear strain imposed on flap)

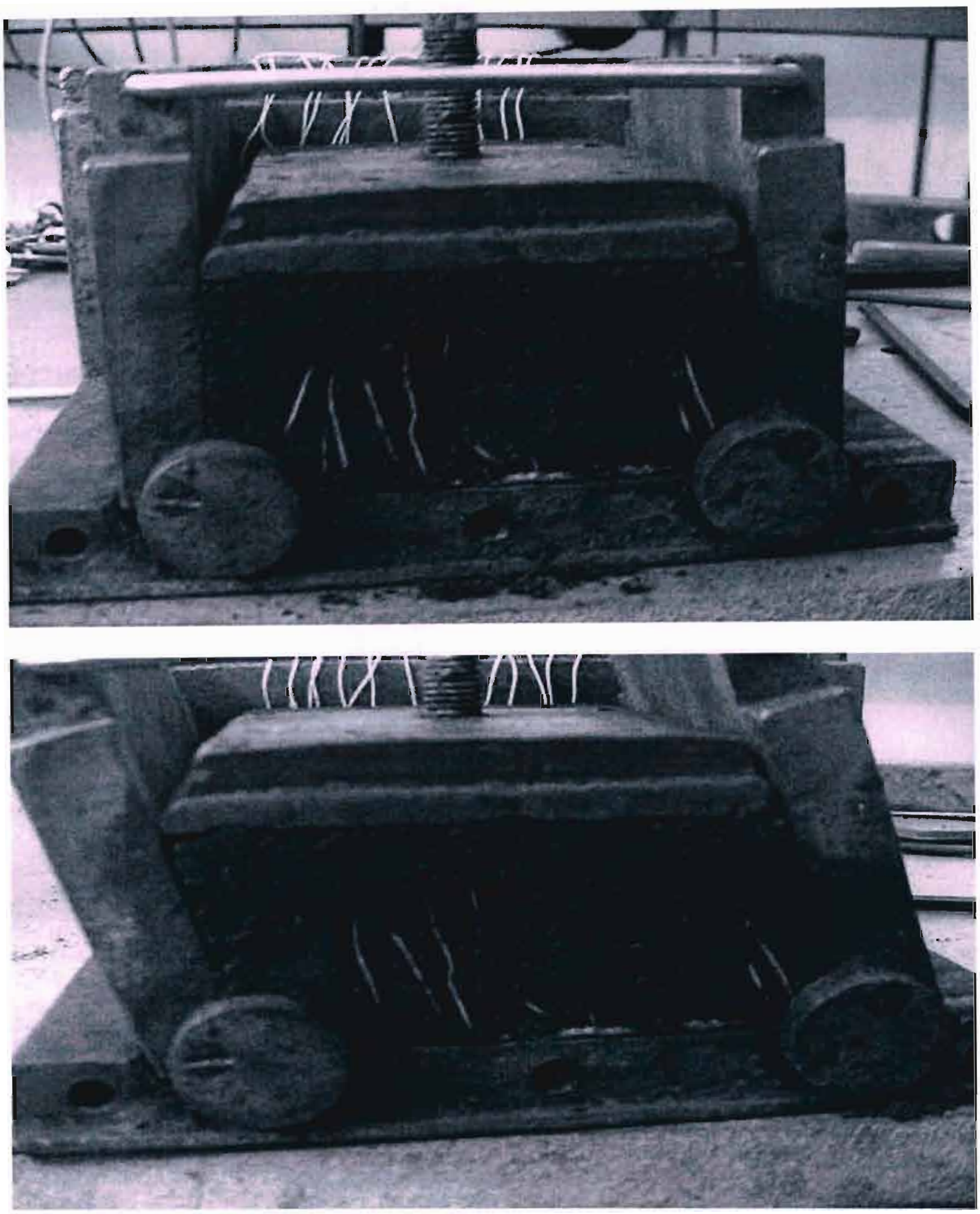


Plate 7.4 Sample of Berea Sands compacted to 30mm height, before and after testing under constant normal stresses of 40 kPa in the type B simple shear apparatus.

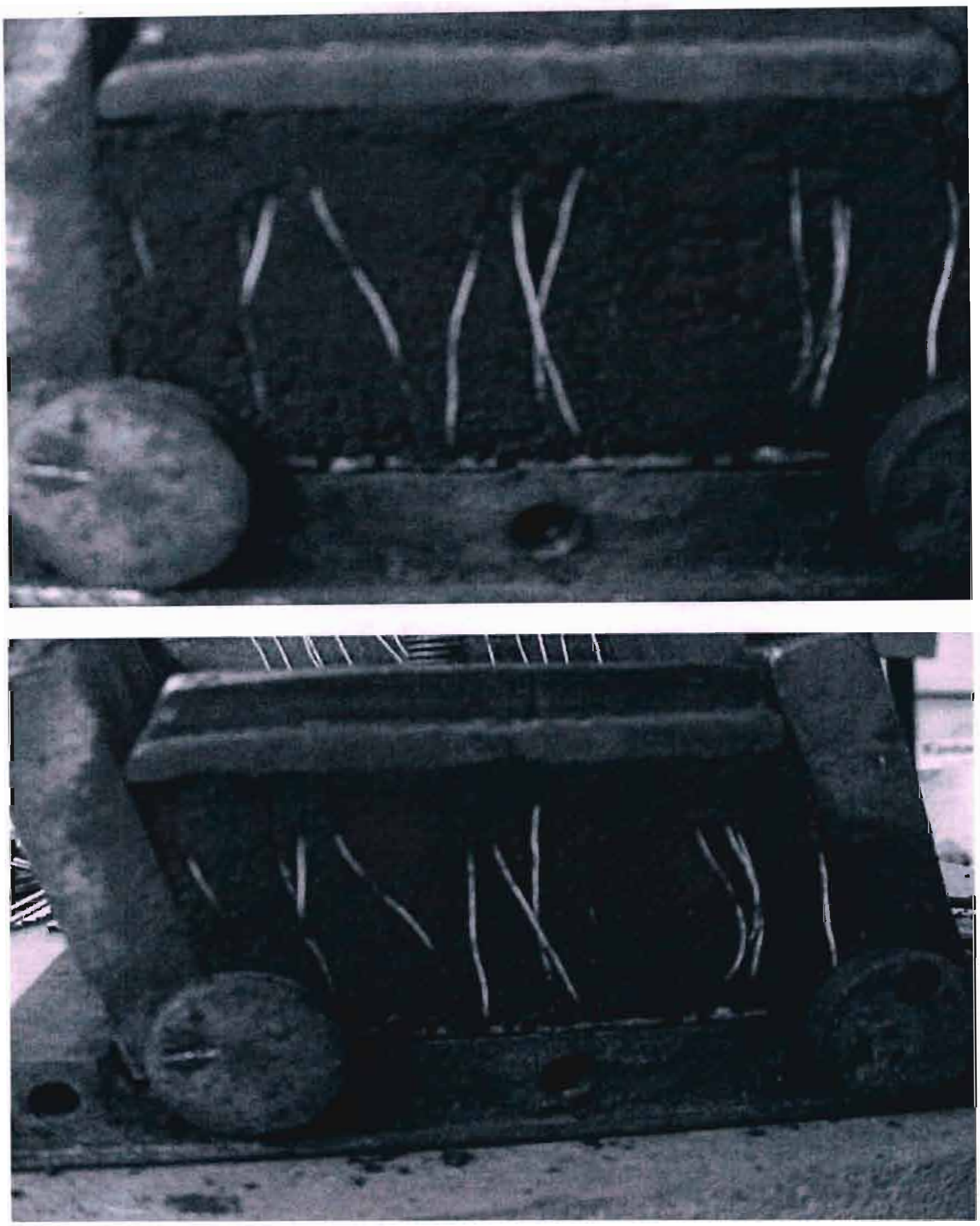


Plate 7.5: Sample of Berea Sands compacted to 30mm height, before and after testing under constant normal stresses of 100 kPa in the type B simple shear apparatus.

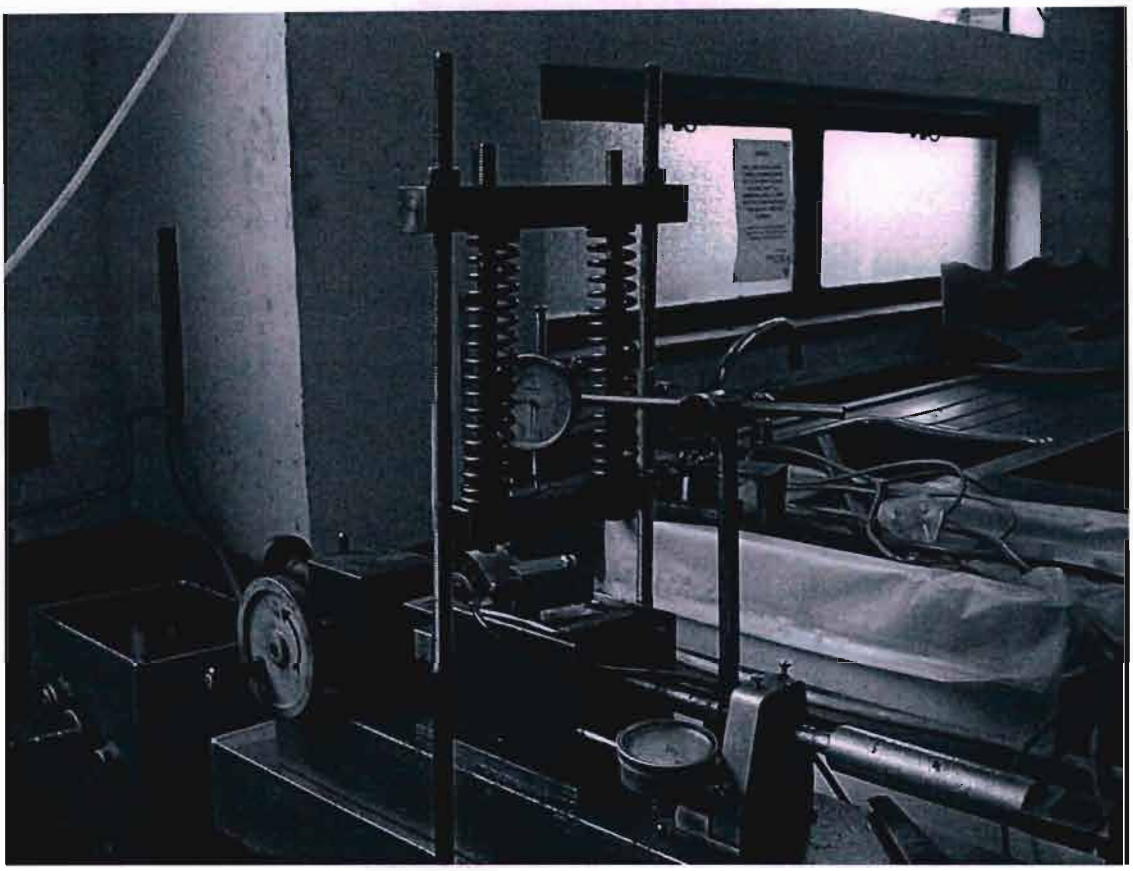


Plate 7.6 Constant normal stiffness test assembly.

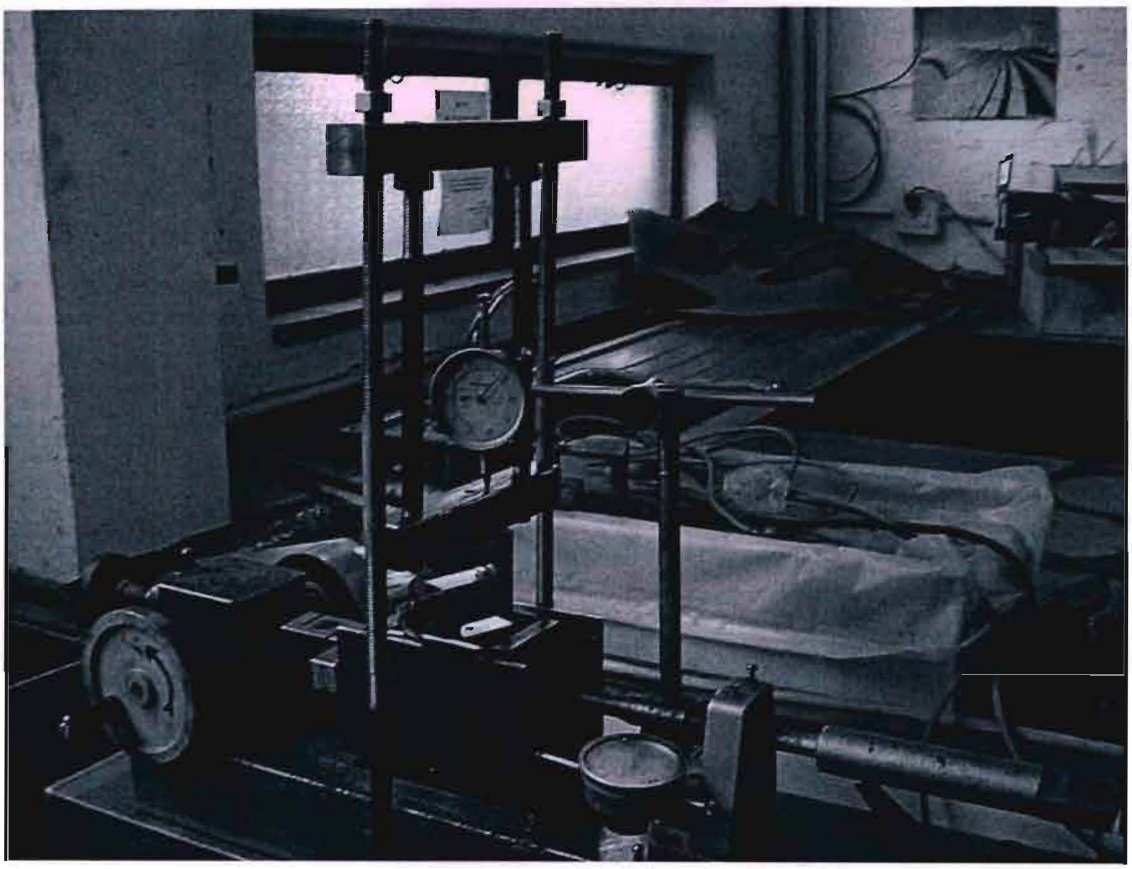


Plate 7.7 Constant volume test assembly

CHAPTER EIGHT

EFFECT OF TEST CONDITIONS ON SHEAR STRESS IN THE SIMPLE SHEAR APPARATUS

8.1 INTRODUCTION

Further study of the new device relating to important test conditions that may affect the results of tests conducted in the new apparatus were conducted and presented below. The aim of this chapter is the evaluation of the effect of the conditions of the boundaries of the soil specimen on the stress strain behavior of samples in simple shear. The main objectives are the experimental study of the effect of varying height to length ratio of a test sample on stress - strain behavior in simple shear as well as experimental investigation of the effect of covering soil samples with thin membranes on stress – strain behavior.

In addition, the predictive capabilities of the simple shear stress equation were studied by comparing the predicted stress – strain curves with the experimental curves. Stress – strain curves for different applied normal stresses were predicted based on numerical analysis of stresses at the boundaries of a soil specimen subject to simple shear strain. The deformation of samples measured in the new apparatus was used to modify the predicted simple shear stress – strain curves as the shear induced volume change could not be determined analytically without fore knowledge of the shear modulus and other elasto plastic properties of the soil.

8.2. EFFECT OF TEST BOUNDARY CONDITIONS ON SHEAR STRENGTH.

In order to evaluate the predictive capabilities of the simple shear equation, the predicted stress – strain curves were compared with the experimentally derived curves. Then experimental data on sample deformation were substituted into the analytical equation in order to generate the final stress – strain curves compatible with the deformation of the soil in simple shear.

Simple shear apparatus rely on the boundaries to impose simple shear strain. Thus the size of the sample in the device is thought to influence the magnitude of mobilized shear stress and stress and strain distribution within the sample.

The conventional laboratory practice is to test soil samples covered in thin membrane. This is non avoidable in triaxial tests and is also applicable in direct shear tests where conventional cyclic undrained shear strength is sought. The use of membranes for the reduction of end effects associated with excessive stress concentration in a test sample at the corners of simple shear device also made it attractive for drained testing of cohesive and cohesionless soils as it is thought that end effect has significant effect on the magnitude of mobilized shear stress. The effect of membrane on the shear strength mobilized by Berea Red Sands in simple shear was investigated and the findings are presented in the next section.

Since stress concentration at the corners of the simple shear apparatus is thought to affect mobilized shear stress, the use of samples of sizes that may minimized such effect on the mobilized shear stress and thus improve the average uniformity of stresses in the sample was examined below.

8.2.1 HEIGHT TO LENGTH RATIO.

Non-uniform stress- strain distributions may occur in a sample if there are significant stress and strain concentrations along the edges of the confined specimen. The test program presented here investigates whether changing the height to length ratio of the test specimen leads to changes in the stress- strain behavior of a soil sample subject to simple shear strain. The samples were compacted to heights of 15 mm, 30 mm and 41 mm, corresponding to height to length ratios of 0.21, 0.42 and 0.58 respectively. The constant normal stress tests were conducted on samples compacted to average dry density of 1401 kg/m^3 – 1408 kg/m^3 and tested at moisture content of 4.2 % - 4.8 % and applied normal stress of 400 kPa.

The curves shown in Fig. 8.1 for samples with height to length ratios of 0.21 are similar to the curves of Fig. 8.2 for samples with height to length ratios of 0.42 in terms of stiffness and mobilized shear stress for maximum strain allowable by the device. The curves shown in Fig. 8.3 for samples with height to length ratios of 0.58 indicated values of mobilized shear stress lower than the values mobilized by the samples with height to length ratios of 0.21 and 0.42

respectively, for the maximum strain imposed by the device. The general trend and deduction is that for a given length of sample, there is a limiting height to length ratio beyond which the mobilized shear stress begins to decrease. For compacted Berea Red Sands this ratio is between 0.42 and 0.58. The tests indicated that for samples with height to length ratio below 0.42, the same value of mobilized shear strength is to be expected.

The vertical strain plots for the 15 mm, 30 mm and 41 mm high samples indicated shear induced volume compression and decreasing rate of reduction in soil volume with increasing shear strain.

Vucetic and Lacasse (1982) reported close agreement between the stress-strain curves up to peak stress for undisturbed specimens of medium stiff naturally laminated Haga clays tested in the NGI simple shear apparatus. The samples diameter varied from 50 mm to 115 mm for the same sample thickness of 10 mm. The height to diameter ratios of all the Haga clay samples tested were below 0.5.

There is no significant strength – deformation behavioral difference between samples tested at height of 15mm and 30mm in the new simple shear apparatus. However the same cannot be said for 41mm thick samples as evident from the stress – strain curves shown in fig 8.3.

30mm height or height to length ratio of 0.42 was selected as the sample dimension for the investigation of the mechanical properties of compacted Berea Red Sands. Soil – pile material interface tests by Potyondy (1961) and Uesigi and Kishida (1986) indicated uniform stress concentration in sample thicknesses up to 35mm. Beyond 35mm it was difficult to ensure uniform stress and strain distribution within the soil mass in direct shear.

Conducting simple shear tests with 30mm thick samples rather than with 15mm thick samples ensures that a reasonable thickness of the soil mass becomes the shear zone, permits direct measurement of a wider range of data on sample deformation, allows for a more consistent measurement of strains and ensures that changes in the mechanical properties are direct reflections of changes in initial state i.e. moisture content and dry density at which the samples were prepared and sheared.

8.2.2 MEMBRANE EFFECTS

Kiekbusch and Schuppener (1977) and Lade and Hernandez (1977) have shown that for undrained triaxial tests on sands, the excess pore water pressures generated during the test will be reduced by the deformation of the rubber membrane. With increasing change in pore water pressure (Δu), the rubber membrane will be pushed away from the interstices of the sand particles at the lateral boundary of the soil specimen.

A small amount of water will thus migrate to the lateral boundary. Since the overall volume of the specimen is kept constant, migration of water results in the reduction of volume of the grain structure of the sand which has the same effect on Δu as do partial drainage.

Lubricated membranes 0.5 mm thick were used to reduce the end effects of the shear box in the Cambridge apparatus (Franke et al, 1979). Laboratory investigation on membrane effects by Franke et al, (1979) reveal that the thickness of membranes affected the measured pore pressure due to changes in shear stress and octahedral total normal stress in undrained static and cyclic loading in simple shear device where the zero lateral strain conditions are achieved by stiff metal walls.

Peacock and Seed (1968) used 0.3 mm thick membranes in the assessment of sand liquefaction, but did not investigate the effect of the presence of and thickness of the membrane. As no quantitative investigation that addressed the membrane effect on the Cambridge type simple shear apparatus especially with regards to drained testing are available, further investigation is necessary.

All the samples tested so far were placed in the simple shear apparatus without being enclosed in a membrane. It is generally believed that stress non-uniformity due to end effects and wall friction can be reduced significantly by enclosing the samples in a thin membrane. The effect of testing samples that were enclosed in membranes on the strength and deformation behavior in simple shear was investigated.

Thin membranes of 0.5mm thickness were used. Two types of membranes were used to test samples and were examined after tests. The first type of membrane was sealed at the corners and has internal area of 68mm X 68mm, slightly less than the area of the simple shear box, and the based was glued to the base of the box. The membrane stretches to the size of the box when the soil is compacted into it. The second type of membrane is similar to the first but slited at the corners, with the vertical sides covering the end flaps and slightly overlapping into the side walls.

Both types of membranes were lubricated to minimize friction between membrane and the walls of the apparatus. The top of the samples were not sealed by the membranes as the conventional procedure for undrained tests was not adopted in this research. Visual examination after preliminary tests indicated that stretching and squeezing of the membranes were strongly indicated in samples covered with slitted membranes. Finn et al, (1971) came to the same conclusion after series of tests on Ottawa Sands. Thus investigation of the effect of membranes on stress - strain behavior was conducted with the first type of membranes, i.e. membranes that were not slitted at the vertical corners.

Figs. 8.4, 8.5 and 8.6 show the stress- strain curves of Berea Red Sands compacted to 30 mm height, dry density of 1401 kg/m^3 - 1404 kg/m^3 and tested at 4.5 % - 4.8 % moisture content under constant normal stress of 400 kPa in the type B simple shear apparatus. Fig. 8.4 show the stress-strain curves of compacted Berea Red Sands samples tested without membranes, Fig. 8.5 show the curves for samples tested with membranes while Fig. 8.6 show the curves for samples tested with membranes on flaps only. The membranes used for these tests were of the same type used for conventional triaxial tests with thickness of 0.50 mm

The curves shown in Figs. 8.4, 8.5 and 8.6 indicate that the presence of membranes resulted in reduction in measured shear stress. However the reduced value may not be representative or intrinsic of the mechanical behavior of compacted Berea Sands as simple shear deformation is associated with cross diagonal contraction and extension of the membrane. The effect of cross diagonal contraction and extension of the membrane on the magnitude of mobilized shear stress is indeterminate. While series of tests reported by Vucetic and Lacasse (1982) have revealed that the use of a membrane that is two times thicker than the standard membrane (0.5mm) resulted in no change in measured strength and modulus of soil tested in the NGI apparatus, they however reported unreliable measurement, due to very large scatter of the vertical strain data when thick (2 mm) rubber membranes were used in the NGI device.

The contractile strain measured in soil samples enclosed in the membrane is unreliable as the membrane has to stretch and contract in response to shear induced volume change in the sample and where the two modes of deformation are not compatible, volume change of the sample may be restricted by the presence of the membrane especially when testing dry or unsaturated soils.

Since correct measurement of contractile strain that can be related to pile capacity is a primary objective of this research, it was decided to carry out the remaining investigation on 30mm thick samples tested without the membrane. This is in a bid to ensure reliable measurement of the compression of a reasonable thickness of the soil sample.

8.3 PREDICTED AND EXPERIMENTAL STRESS – STRAIN CURVES

The stress – strain curves of constant normal stress, normal stiffness and constant volume tests conducted on compacted samples of Berea Red Sands in the new simple shear apparatus were compared with the stress – strain curves predicted by the simple shear stress equation. The comparism was for samples of the same moisture contents and dry densities.

Since the simple shear equation cannot predict changes in mobilized shear stress due to changes in moisture content, the predicted curves were compared with stress – strain curves of samples that were compacted into the new apparatus at high moisture content and then dried down slowly to moisture contents close to zero percent. The details are presented below

8.3.1 CONSTANT NORMAL STRESS CURVES

Series of constant normal stress tests were conducted on Berea Red Sands samples compacted to dry density of 1404 kg/m^3 – 1424 kg/m^3 and moisture content of 0.4% - 0.8%. The samples were tested at constant normal stresses of 40 kPa, 100 kPa, 200 kPa and 400 kPa. The samples were dried down to moisture contents of 0.37% - 0.76% under laboratory room temperature. The stress – strain curve of dry Berea Red Sands can be compared with the stress – strain curves predicted by the use of simple shear stress equation derived in chapter six, since the simple shear stress equation cannot predict the effect of changes in specimen moisture content on mobilized shear strength.

The experimental stress – strain curves are shown in Fig. 8.7. The stress – strain curves indicate continuous increase in shear stress for the range of simple shear strain imposed on the compacted samples and peak value of shear stress cannot be established for all the normal stresses used for the tests.

The dry samples indicated significant (~ 10%) shear induced volume compression for samples tested at constant normal stress of 100 kPa and above. For samples tested at constant normal stress of 40 kPa, the shear induced volume compression of the soil is not significant (~ 3%) as the sample remain stiff for the range of imposed simple shear strain.

The predicted stress – strain curves were determined by substituting the vertical strain values of the experimental curves for the different constant normal stresses shown in Fig 8.7, into the simple shear stress equation. The predicted stress – strain curves shown in Fig 8.8 indicate peak values of shear stress for all values of constant normal stress used.

The predicted and the experimental curves were superimposed in Fig 8.9. Here the curves show that the shear stress determined by the analytical and the experimental methods are the same for imposed shear strain between 10% and 18%, beyond which the curves begin to diverge, with the experimental curves rising steadily above the predicted curves. The strain at which the experimental and the predicted curves begin to diverge decreases with increase in constant normal stress.

The general trend observed in Fig 8.9 is that reliable values of simple shear stress can be determined at small values of imposed simple shear strain since non uniformity of internal stresses and strains within the samples are not significant at small values of imposed shear strain.

Devices that impose simple shear strain on samples do not impose complimentary shear stresses on the vertical surfaces of the samples. Complimentary shear stresses on the vertical surfaces of the samples were incorporated in the formulation of the simple shear stress equation.

The effect of the complimentary stresses did not result in increase but a decrease in computed shear stress, and the magnitude of the complimentary stresses increases with increase in imposed simple shear strain. This is the reason for the divergence of the curves as the simple shear strain increases.

The primary deduction is that reliable values of shear modulus can be determined from the stress – strain curves generated from constant normal stress simple shear tests data using the new apparatus.

8.3.2 NORMAL STIFFNESS CURVES

Series of normal stiffness tests were conducted on Berea Red Sands samples compacted to dry density of 1404 kg/m^3 – 1408 kg/m^3 and moisture content of 0.3% - 0.6%. The samples were tested at normal stiffness conditions of 140 N/mm, 250 N/mm and 1600 N/mm and initial normal stresses of 40 kPa to 400 kPa.

The normal stiffness tests conducted on samples with springs of stiffness constant of 140 N/mm are given below. The constant normal stress tests were conducted on 30mm thick samples compacted into the simple shear box to dry density of 1411 kg/m^3-1433 kg/m^3 and moisture content of 15.58 % - 16.44 % and tested at moisture content of 0.3% - 0.8%.

The experimental stress - strain curves are presented in Fig 8.10 while the analytically predicted curves are shown in Fig 8.11. The predicted curves were modified by substituting the vertical strain values indicated in the tests reported in Fig 8.10, in the simple shear equation.

The experimental and the analytically predicted curves are superimposed in Fig 8.12. The analytical curves predict accurately the experimental stress – strain curves for imposed simple shear strain of 10%. For imposed strains higher than 10%, the experimental curves diverged from the predicted curves. The difference in the values of shear stress at strains beyond 10% may be due to the fact that internal stress and strain non uniformity in sands increases with increase in simple shear strain. In addition, simple shear apparatus does not impose complimentary shear stresses on the vertical surfaces of the sample.

The effect of these non complimentary stresses on mobilized shear stress increases with increase in imposed simple shear strain.

The mobilized shear stress at normal stiffness of 140N/mm is marginally smaller than the mobilized shear stress at constant normal stress, within the range of imposed simple shear strain used.

The normal stiffness tests were conducted on 30 mm thick samples with springs of stiffness constant of 250 N/ mm. The tests were conducted on 30mm thick samples compacted into the simple shear box to dry density of 1411 kg/m^3-1424 kg/m^3 and moisture content of 16.77 % - 17.55 %, the samples were then slowly dried down and tested at moisture content of 0.24% - 0.57%.

Fig. 8.13 shows the stress – strain curves for dry Berea Red Sands. Fig. 8.14 shows the analytical normal stiffness curves. The analytical simple shear stress – strain curves were modified by substituting the decreasing normal stress values indicated in the tests shown in Fig. 8.13, in the constant normal stress equation. The Analytical and experimental normal stiffness simple shear stress – strain curves are superimposed in Fig 8.15. Here the analytical curves overestimates the experimental results at imposed strains up to 15 %-20% and underestimates the experimental at strains higher beyond this range.

The experimental normal stiffness tests conducted on 30 mm thick samples with springs of stiffness constant of 1660 N/ mm are given below. The normal stiffness tests were conducted on 30mm thick samples compacted into the simple shear box to dry density of 1415 kg/m^3-1427 kg/m^3 and moisture content of 15.55 % - 16.55 % and tested at moisture content of 0.22% - 0.60%.

Fig. 8.15 shows the stress – strain curves for dry Berea Red Sands. Fig. 8.16 shows the predicted normal stiffness curves. The analytical simple shear stress – strain curves were modified by substituting the decreasing normal stress values indicated in the tests shown in Fig. 8.15, in the constant normal stress equation. The predicted and experimental normal stiffness simple shear stress – strain curves were superimposed in Fig 8.17. Here the predicted curves indicated stiffer stress - strain response to imposed simple shear strain and intersects the experimental curves at imposed strains within the range of 15 %-20%.

The experimental values of simple shear stresses at which the experimental and analytical curves started diverging i.e. for samples tested under 140N/mm or intersect ie for samples tested under 250N/mm and 1660N/mm are slightly less than the peak shear stress indicated by the analytical curves. These shear stress values may be taken as the peak values since the experimental curves continue to increase with increase in imposed shear strain.

The experimental stress – strain curves for samples tested under normal stiffness of 140 N/mm, 250 N/mm, and 1660 N/mm indicate that the stiffness of the sample increases with increase in the imposed normal stiffness.

8.3.3 CONSTANT VOLUME CURVES

Fig 8.18 shows the experimental constant volume simple shear stress – strain for compacted dry Berea Red Sands The samples were tested at the following initial state; sample height is 30mm , moisture constant range is 0.3% -0.7%, dry density range is 1402 kg/m^3 – 1407 kg/m^3. For the initial normal stresses of 40 kPa – 400 kPa, all the samples tested indicate peak values of shear stress.

The analytical constant volume simple shear stress – strain curves are shown in Fig 8.19. The analytical and experimental constant volume simple shear stress – strain curves are superimposed in Fig 8.20. Fig 8.20 indicates large difference between the experimental and the predicted stress – strain curves. The experimental curves indicated higher shear stress values than the predicted curves.

While the experimental curves are generally bilinear, the analytical curves indicated strain softening behavior for all initial normal stresses, implying that compacted Berea Red Sands may exhibit brittle behavior under constant volume shear condition. The significant difference between experimental and analytical response may be due to internal non uniform stresses and strains within the soil mass which is unavoidable in test conditions in which volume change are prevented manually.

While large difference between the experimental and the predicted stress – strain curves may tend to imply that the simple shear equation cannot predict the constant volume response of compacted Berea Red Sands, the predicted curves are similar in shape to the curves presented by Bouy and Carter (1988) and Jewell (1988) from series of tests conducted with the University of Sydney constant normal stiffness direct shear device on samples of artificially cemented sands (cement content of 5 – 15%).

Thus the predicted curves represent the correct constant volume behavior of compacted Berea Red Sands.

TABLE 8.1

TEST SERIES 8.1:Constant normal stress tests; The experimental results of constant normal stress tests conducted on samples compacted in the simple shear box to 30mm thick, dry density of 1404kg/m^3 – 1424kg/m^3 at moisture content of 16.44% - 18.49% and tested at moisture range of 0.37% - 0.76% are given below.

test no	normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test Mg/m^3	experimental shear stress (kPa)	total vertical strain (mm/mm)	shear induced vertical strain (mm/mm)
8.1	40	17.66	0.42	1.424	20.82	0.06	0.04
8.2	100	17.22	0.76	1.420	54.00	0.08	0.06
8.3	200	16.77	0.47	1.404	111.66	0.08	0.06
8.4	400	17.33	0.37	1.411	205.33	0.09	0.07

TABLE 8.2

TEST SERIES 8.2 (NORMAL STIFFNESS TEST): The normal stiffness tests conducted on samples with springs of stiffness constant of 140 N/mm are given below. The constant normal stress tests were conducted on 30mm thick samples compacted into the simple shear box to dry density of 1411 kg/m^3-1433 kg/m^3 dry density and moisture content of 15.58 % - 16.44 % and tested at moisture content of 0.3% - 0.8%.

test no	Initial normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	experimental shear stress (kPa)	final normal stress (kPa)	total vertical strain (mm/mm)	shear induced vertical strain (mm/mm)
8.21	40	15.77	0.30	1.433	15	20	0.02	0
8.22	100	15.58	0.56	1.422	43	35	0.03	0.01
8.23	200	16.44	0.40	1.411	95	50	0.04	0.02
8.24	400	16.33	0.80	1.417	190	90	0.04	0.02

TABLE 8.3

TEST SERIES 8.3 (NORMAL STIFFNESS TEST): The experimental normal stiffness tests conducted on 30 mm thick samples with springs of stiffness constant of 250 N/ mm are given below. The constant normal stress tests were conducted on 30mm thick samples compacted into the simple shear box to dry density of 1411 kg/m^3-1424 kg/m^3 dry density and moisture content of 16.77 % - 17.55 % and tested at moisture content of 0.24% - 0.57%.

test no	Initial normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	experimental shear stress (kPa)	final normal stress (kPa)	total vertical strain (mm/mm)	shear induced vertical strain (mm/mm)
8.31	40	17.33	0.24	1.412	14	20	0.07	0.05
8.32	100	17.55	0.46	1.411	38	25	0.05	0.03
8.33	200	16.77	0.33	1.407	77	25	0.03	0.01
8.34	400	17.11	0.57	1.424	160	50	0.02	0.00

TABLE 8.4

TEST SERIES 8.4 (NORMAL STIFFNESS TEST): The experimental normal stiffness tests conducted on 30 mm thick samples with springs of stiffness constant of 1660 N/mm are given below. The constant normal stress tests were conducted on 30mm thick samples compacted into the simple shear box to dry density of 1415 kg/m^3-1427 kg/m^3 dry density and moisture content of 15.55 % - 16.55 % and tested at moisture content of 0.22% - 0.60%.

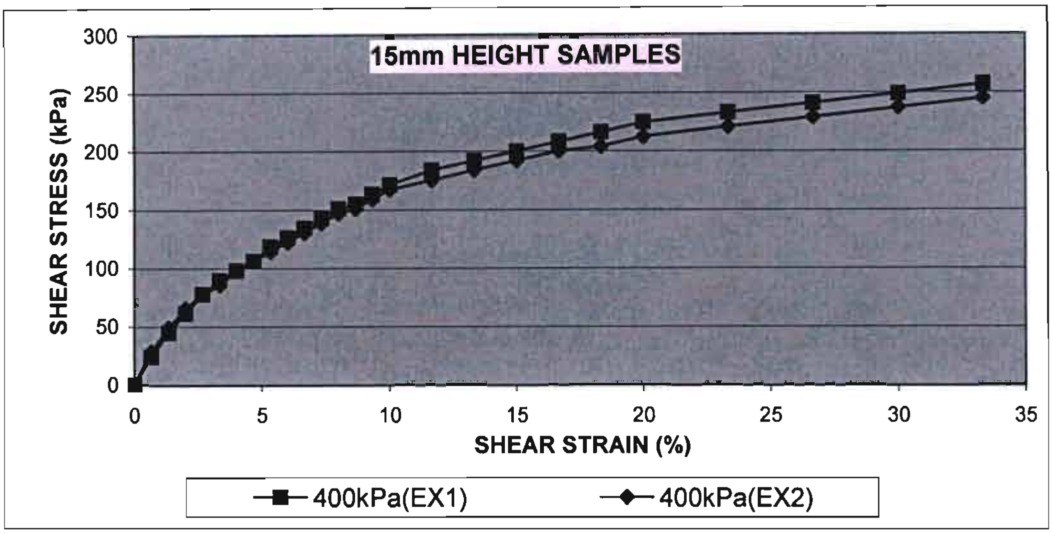
test no	Initial normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	experimental shear stress (kPa)	final normal stress (kPa)	total vertical strain (mm/mm)	shear induced vertical strain (mm/mm)
8.41	40	15.88	0.40	1.415	12	20	0.05	0.03
8.42	100	15.55	0.22	1.427	29	60	0.04	0.021
8.43	200	16.22	0.55	1.426	62	160	0.03	0.01

8.44	400	16.55	0.60	1.420	136	335	0.03	0.01

TABLE 8.5

TEST SERIES 8.5 (CONSTANT VOLUME TEST): The experimental results of constant volume tests conducted on 30 mm thick samples compacted into the simple shear box to 1434 kg/m^3 - 1442 kg/m^3 dry density and moisture content of 15.04% - 15.63% and tested at moisture content of 0.44 % - 0.77 %.

test no	Initial normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	experimental shear stress (kPa)	Final. normal stress (kPa)
8.51	40	15.44	0.66	1.4427	23.61	18.10
8.52	100	15.63	0.77	1.4341	43.55	33.84
8.53	200	15.34	0.55	1.4352	78.44	65.45
8.54	400	15.04	0.44	1.4416	110.44	174.49



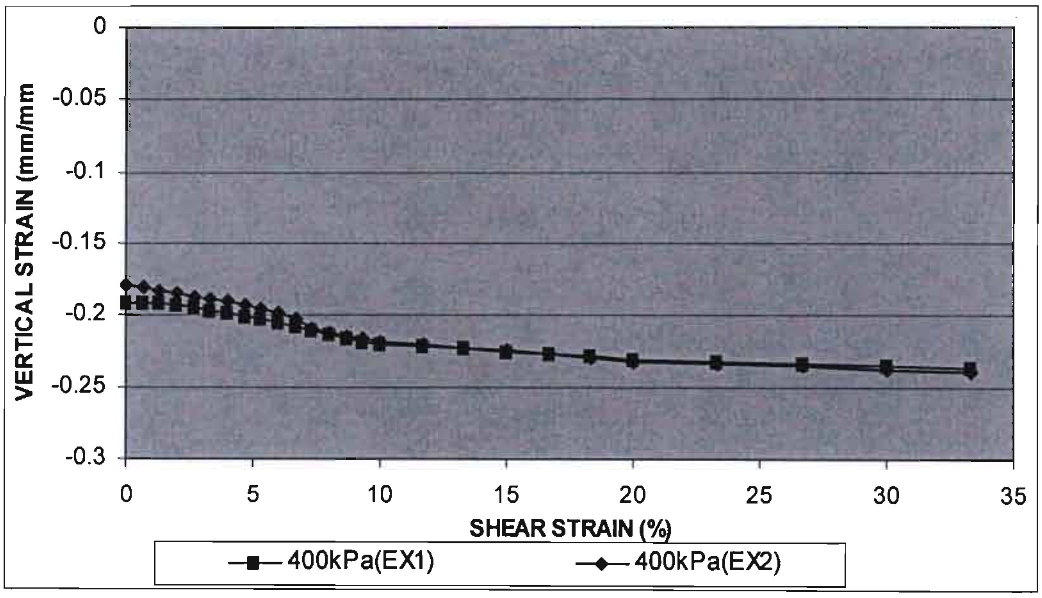
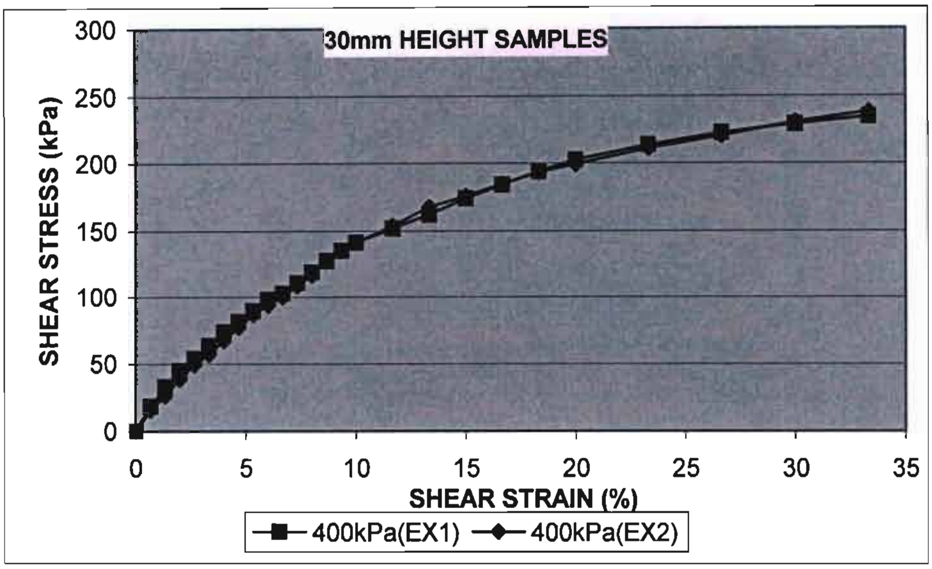


Fig. 8.1 Stress- strain curves of Berea Sands compacted to 15 mm height, dry density of 1401 kg/m^3 - 1405 kg/m^3 and 4.1% - 4.5 % moisture content and tested under constant normal stresses of 400 kPa in the type B simple shear apparatus.



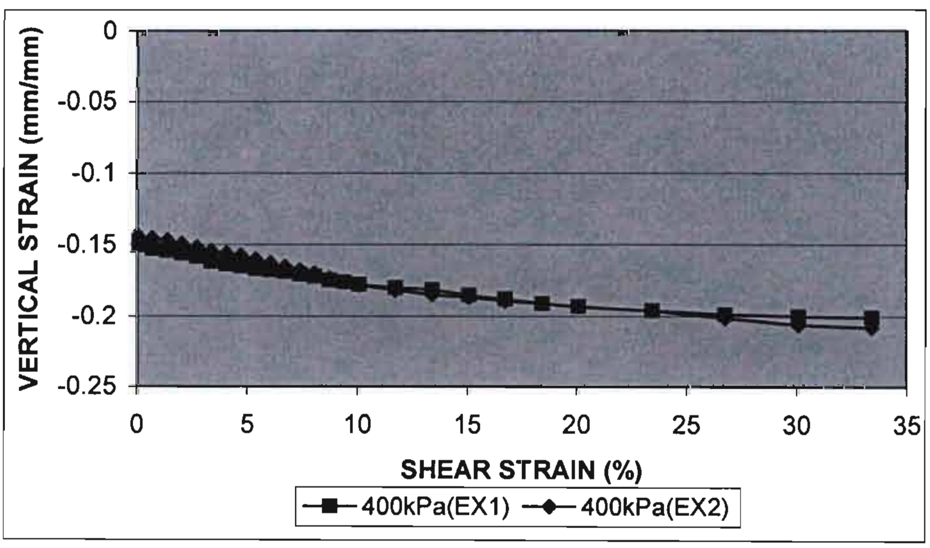


Fig. 8.2 Stress- strain curves of Berea Sands compacted to 30 mm height, dry density of 1400 kg/m^3 - 1408 kg/m^3 and 4.2 % - 4.7 % moisture content and tested under constant normal stresses of 400 kPa in the type B simple shear apparatus.

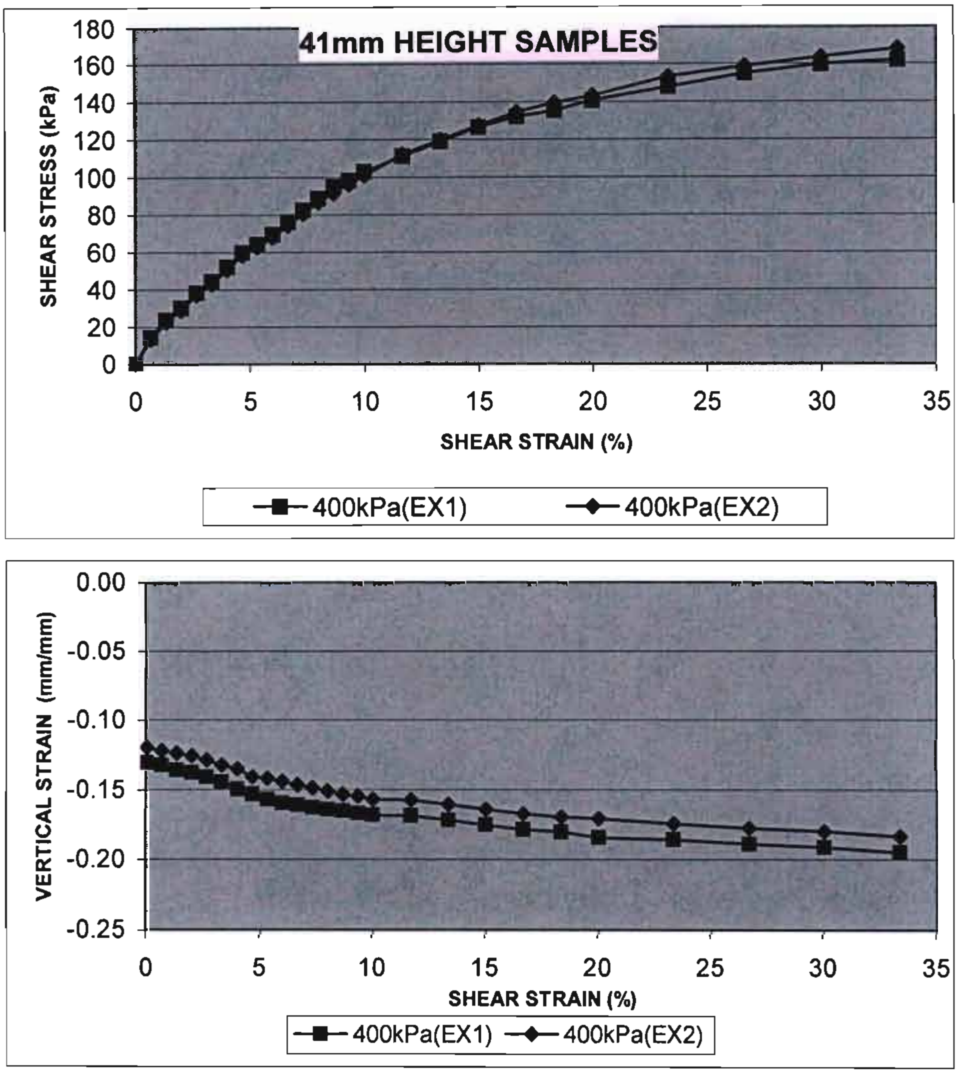


Fig. 8.3 Stress- strain curves of Berea Sands compacted to 30 mm height, dry density of 1401 kg/m^3 - 1405 kg/m^3 and 4.2 % - 4.6 % moisture content and tested under constant normal stresses of 400 kPa in the type B simple shear apparatus.

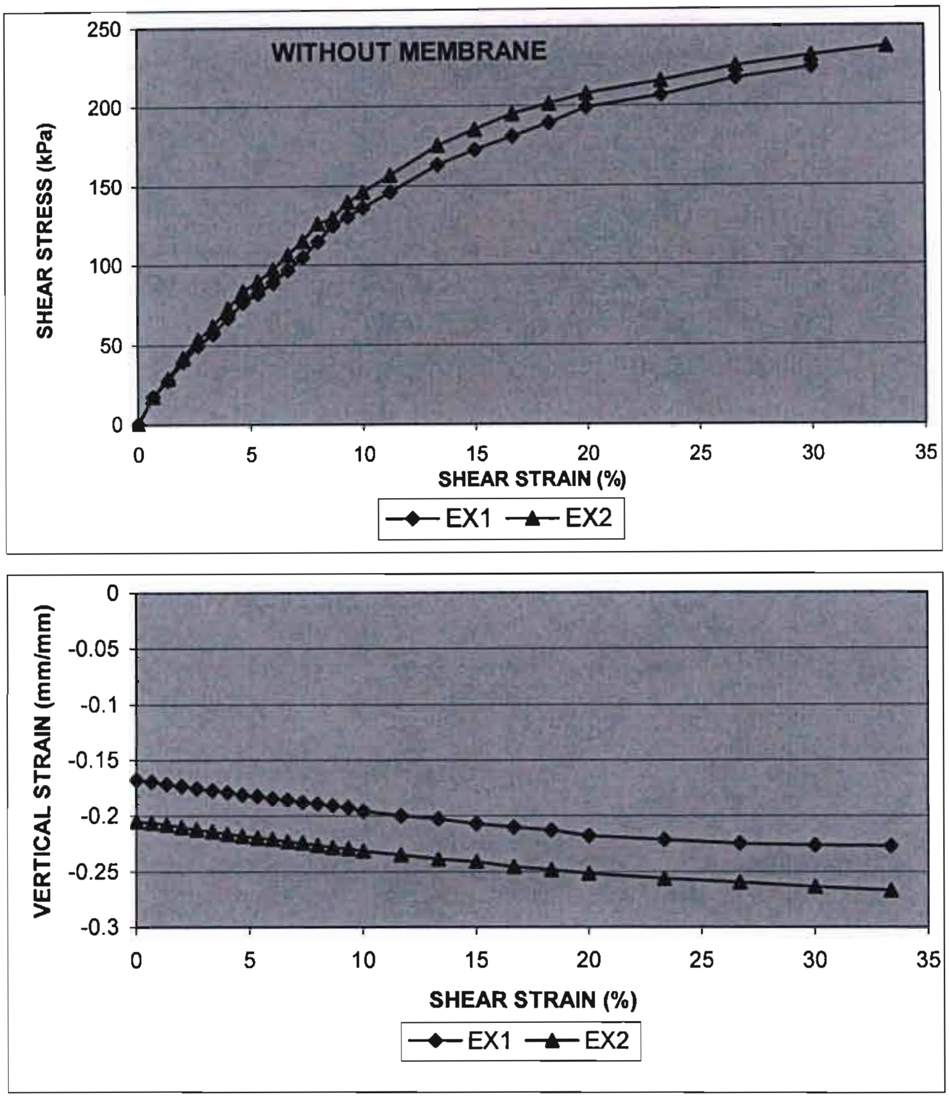
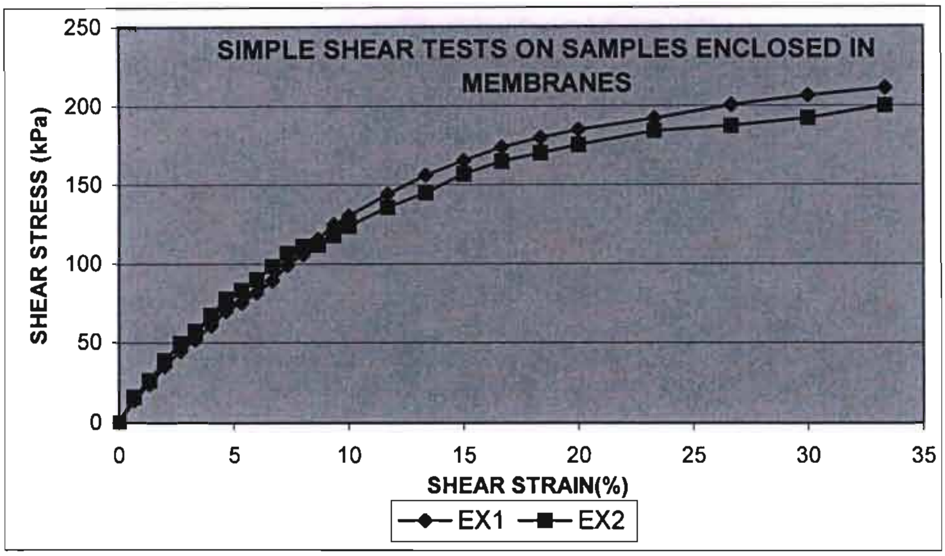


Fig 8.4 Stress- strain curves of Berea Sands compacted to 30 mm height, dry density of 1401 kg/m^3 - 1404 kg/m^3 and 4.5 % - 4.8 % moisture content and tested under constant normal stress of 400 kPa in the type B simple shear apparatus without membranes.



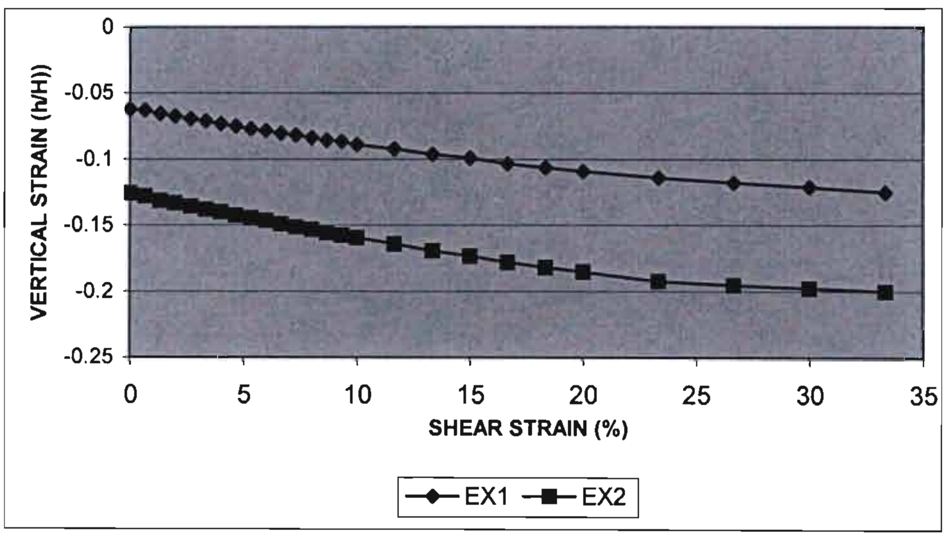
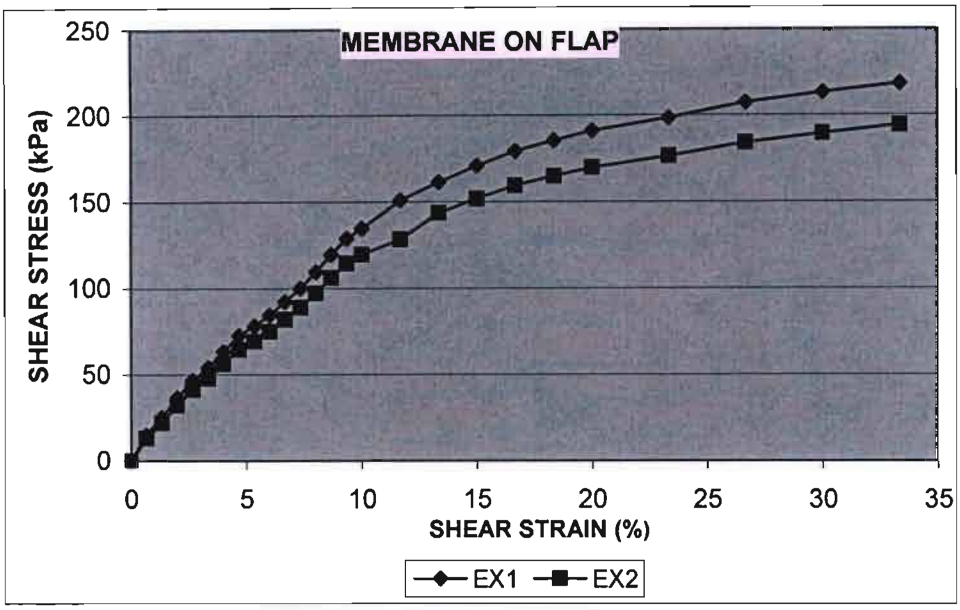


Fig 8.5 Stress- strain curves of Berea Sands compacted to 30 mm height, dry density of 1401 kg/m^3 - 1405 kg/m^3 and 4.1 % - 4.2 % moisture content and tested under constant normal stress 400 kPa in the type B simple shear apparatus with membranes.



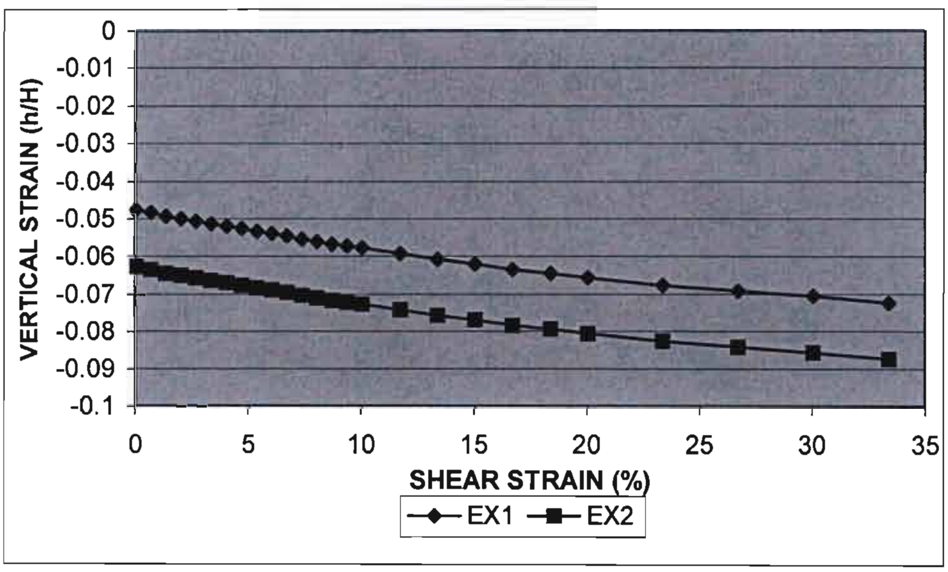
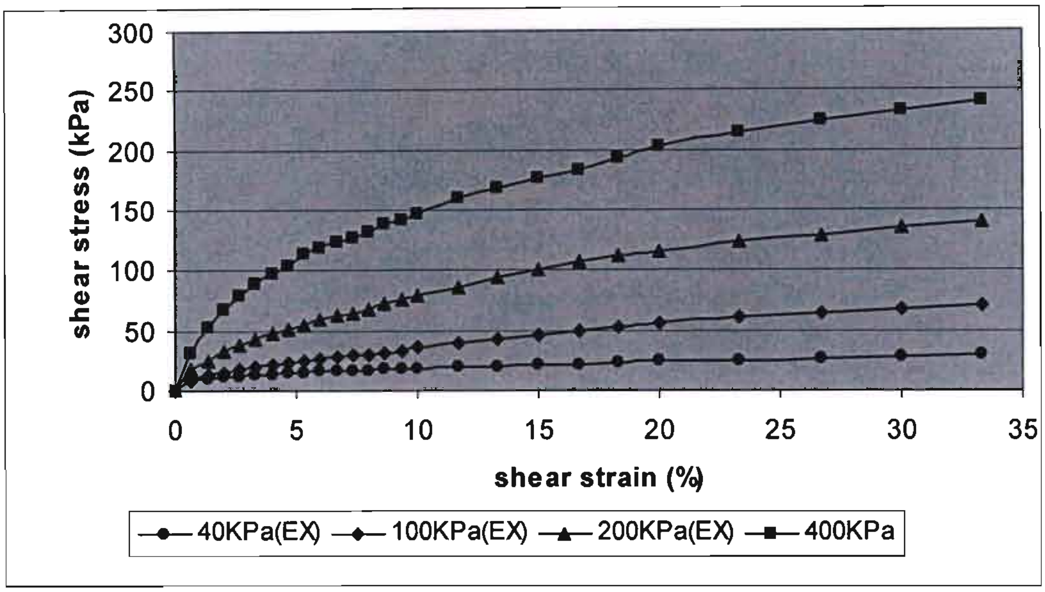


Fig 8.6 Stress- strain curves of Berea Red Sands compacted to 30 mm height, dry density of 1400 kg/m^3 - 1406 kg/m^3 and 4.2 % - 4.4 % moisture content and tested under constant normal stress of 400 kPa in the type B simple shear apparatus with membranes on flaps only.



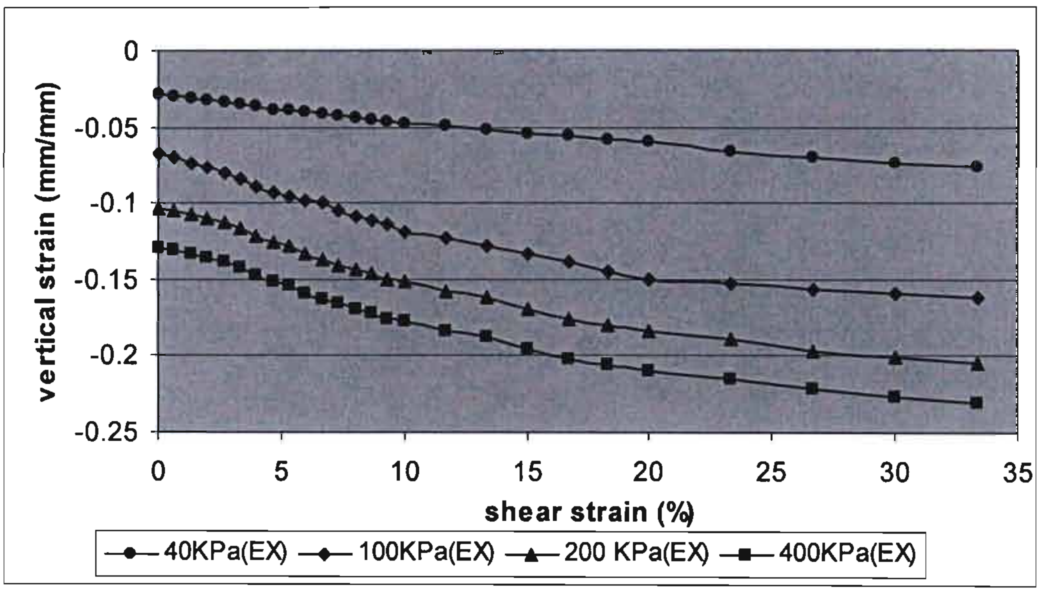


Fig 8.7 Experimental (EX) constant normal stress simple shear stress – strain curves of compacted Berea Red Sands tested at moisture content of 0.37% - 0.76%

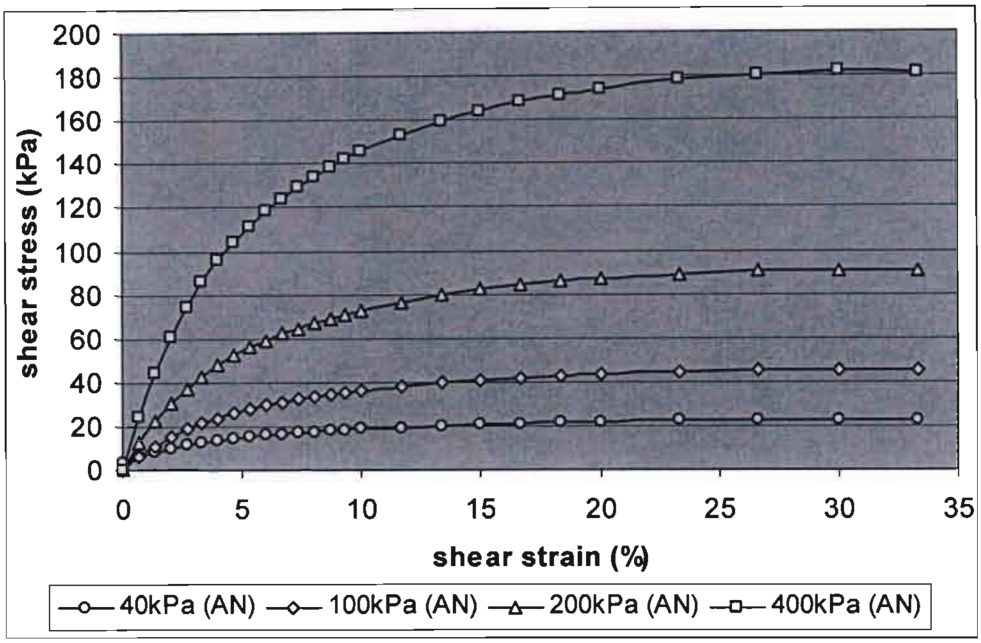


Fig. 8.8 Analytically predicted (AN) simple shear stress strain curve for conditions of constant normal stress on the specimen

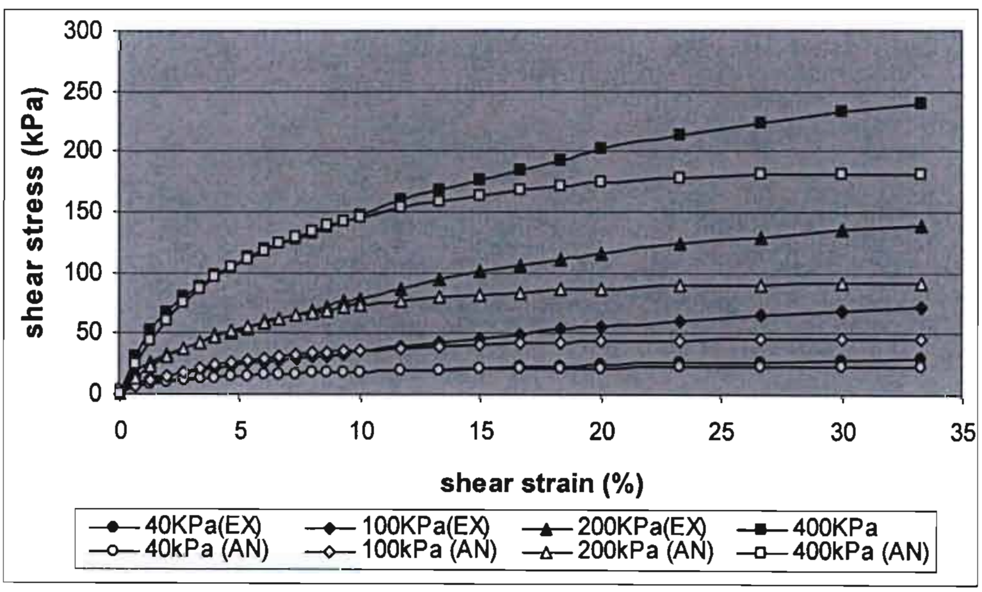
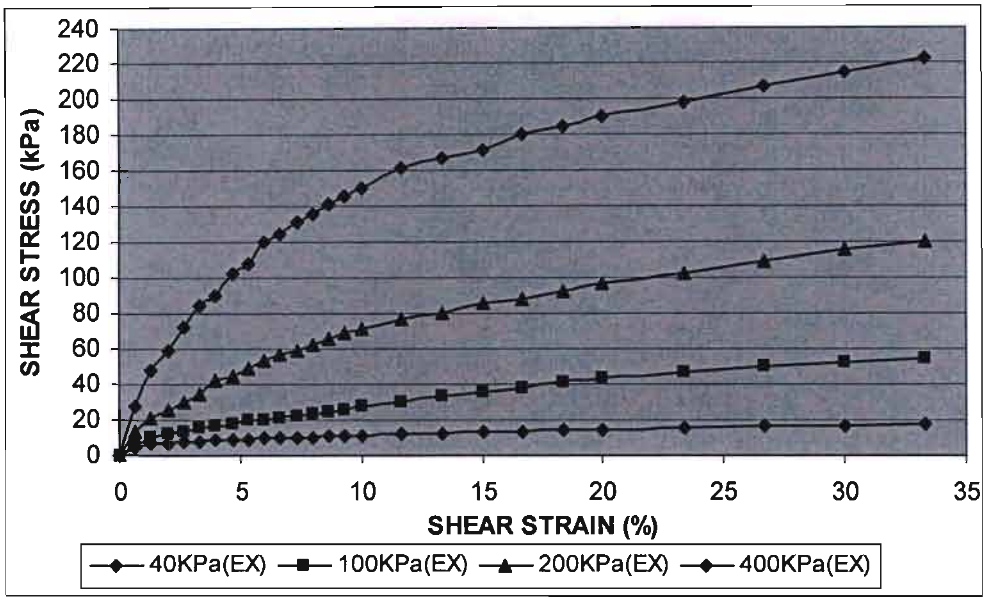


Fig. 8.9 Analytically predicted (AN) and Experimentally determined (EX) simple shear stress strain curve for specimen subjected to constant normal stress.



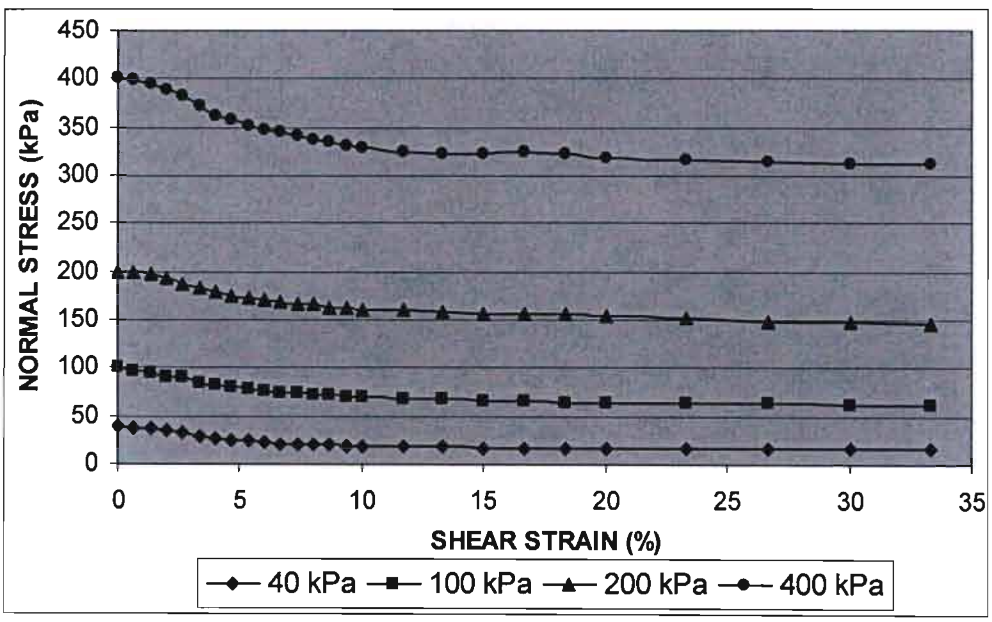


Fig. 8.10 Constant stiffness simple shear stress – strain curves for dry Berea Red Sands. The samples were tested at normal stiffness of 140N/mm

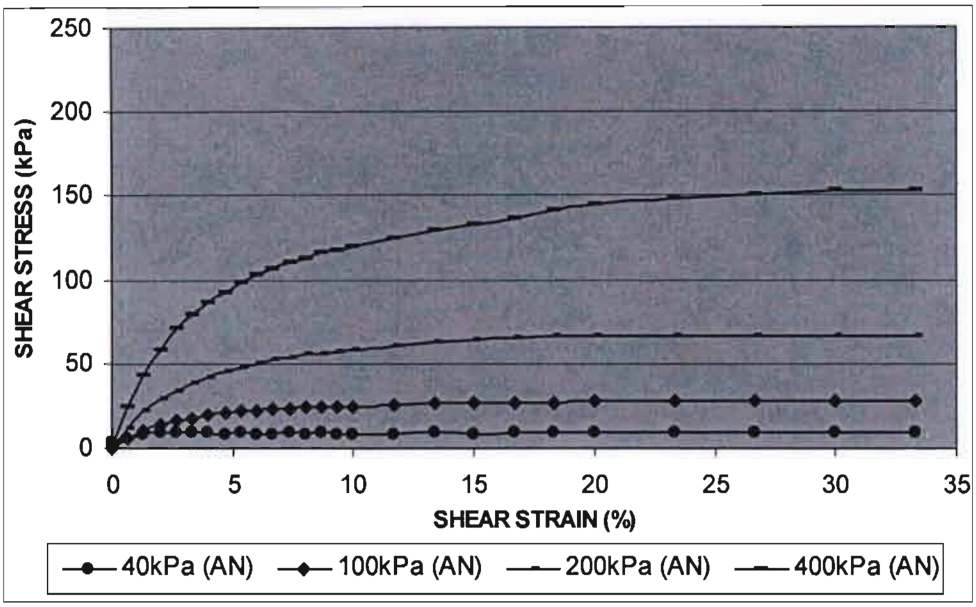


Fig. 8.11 Analytical normal stiffness curves of Berea Red Sands. The simple shear stress – strain curves were generated by substituting the decreasing normal stress values shown in Fig. 8.10 in the constant normal stress equation.

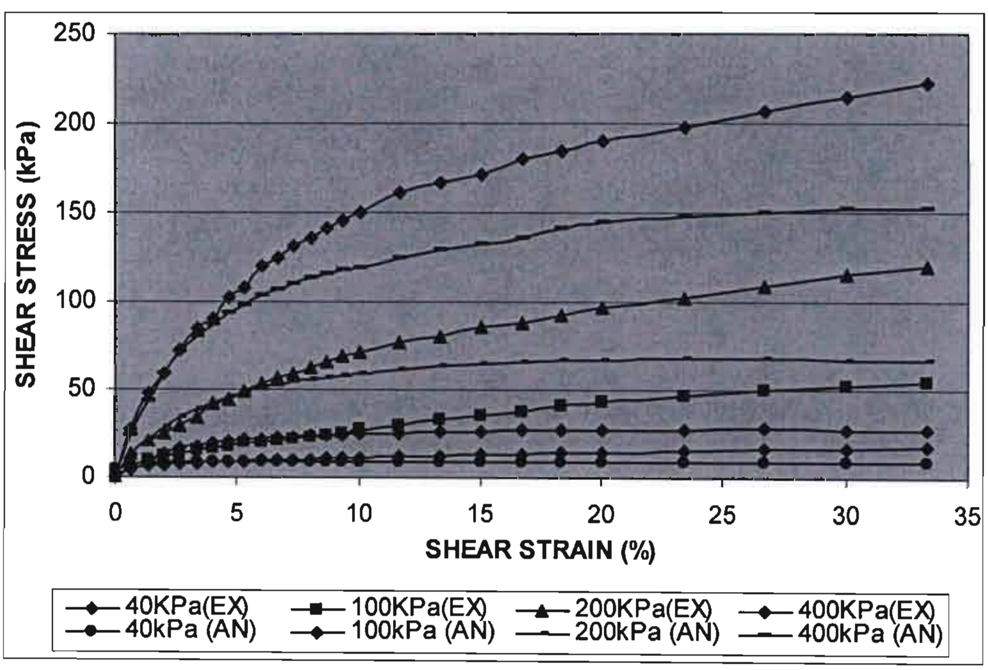
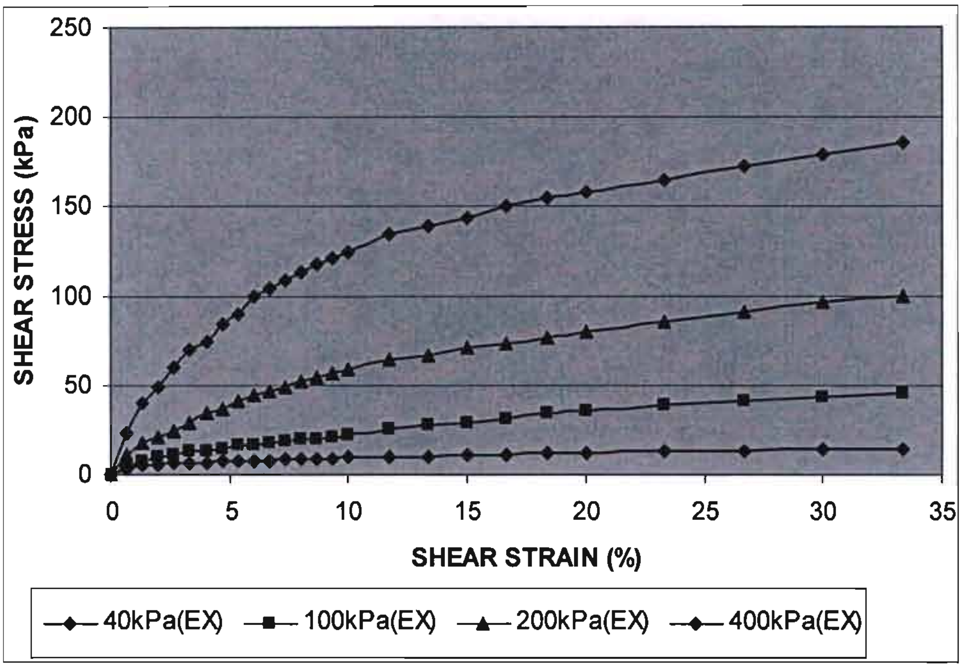


Fig 8.12 Analytical and experimental normal stiffness simple shear stress – strain curves. The normal stiffness of the spring is 140 N/mm



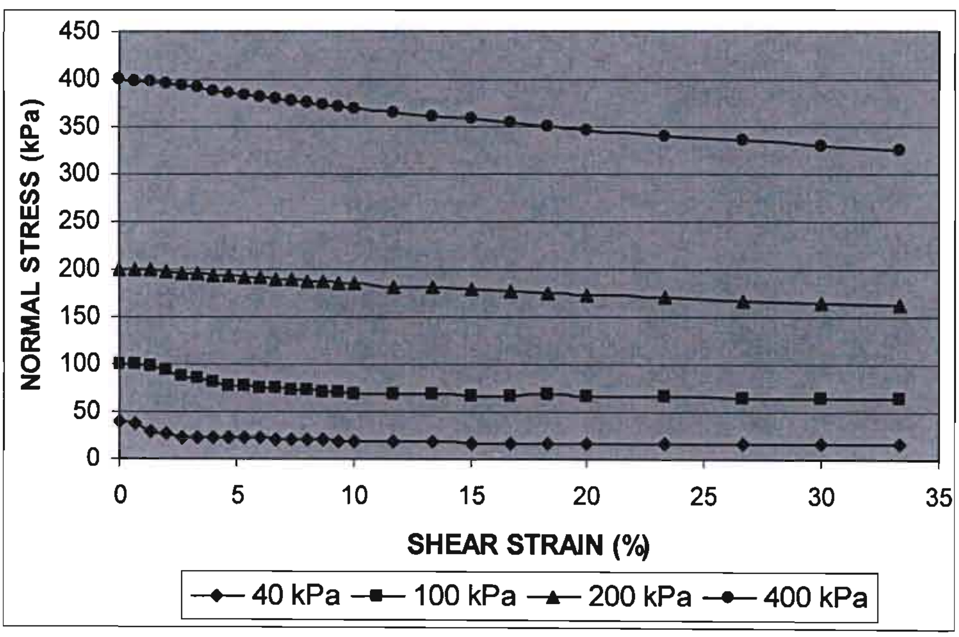


Fig. 8.13 Normal stiffness simple shear stress – strain curves for dry Berea Red Sands. The samples were tested at normal stiffness of 250N/mm.

The experimental normal stiffness simple shear curves for compacted Berea Red sands tested at moisture content of 0.2% - 0.6%.

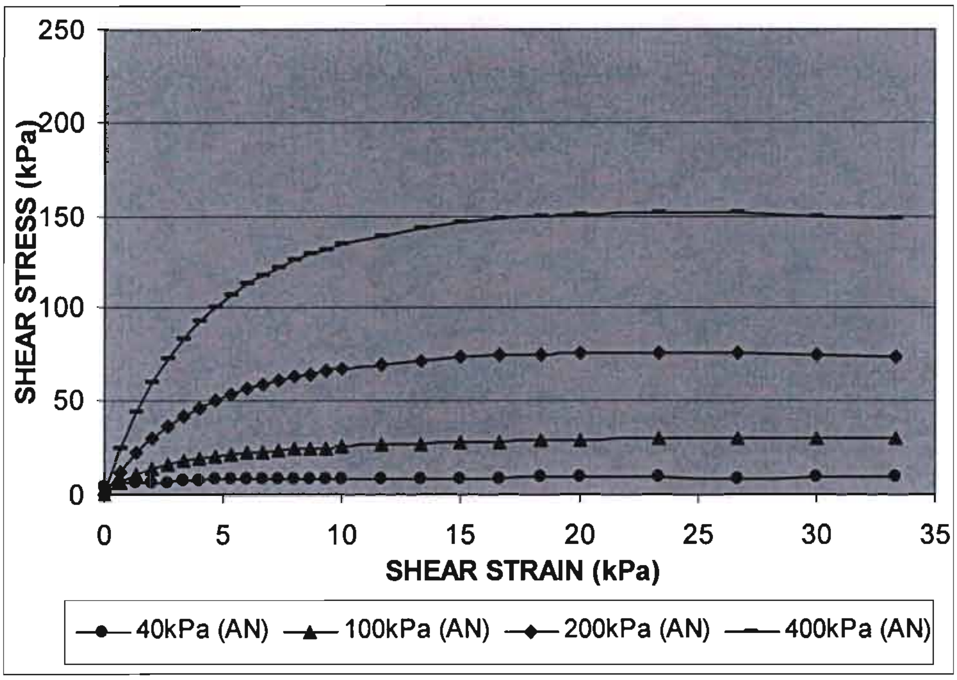


Fig. 8.14 Analytical normal stiffness curves of Berea Red Sands. The simple shear stress – strain curves were generated by substituting the decreasing normal stress values shown in Fig. 8.13 in the constant normal stress equation.

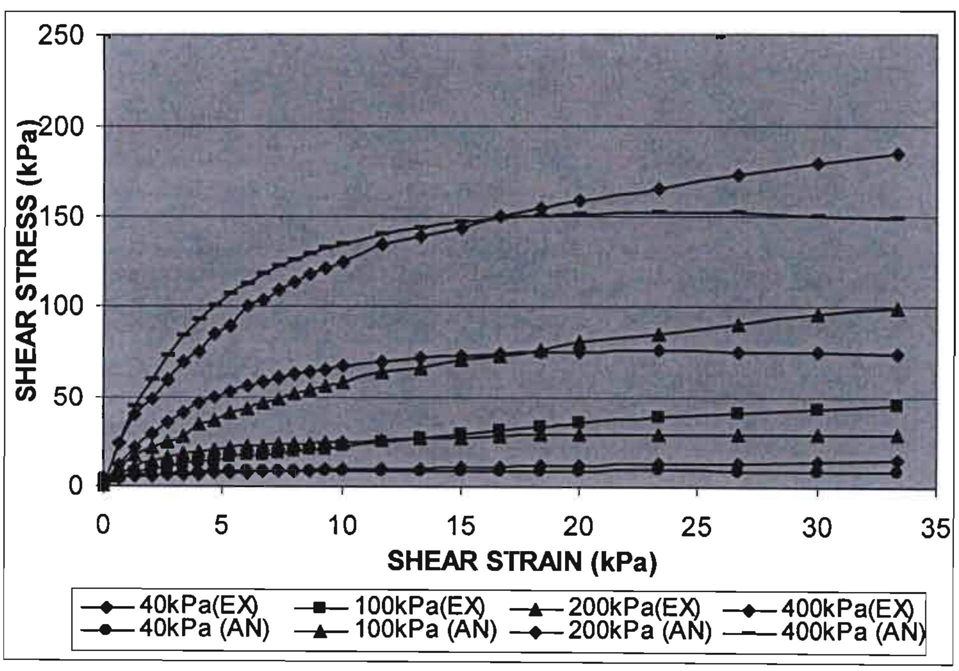
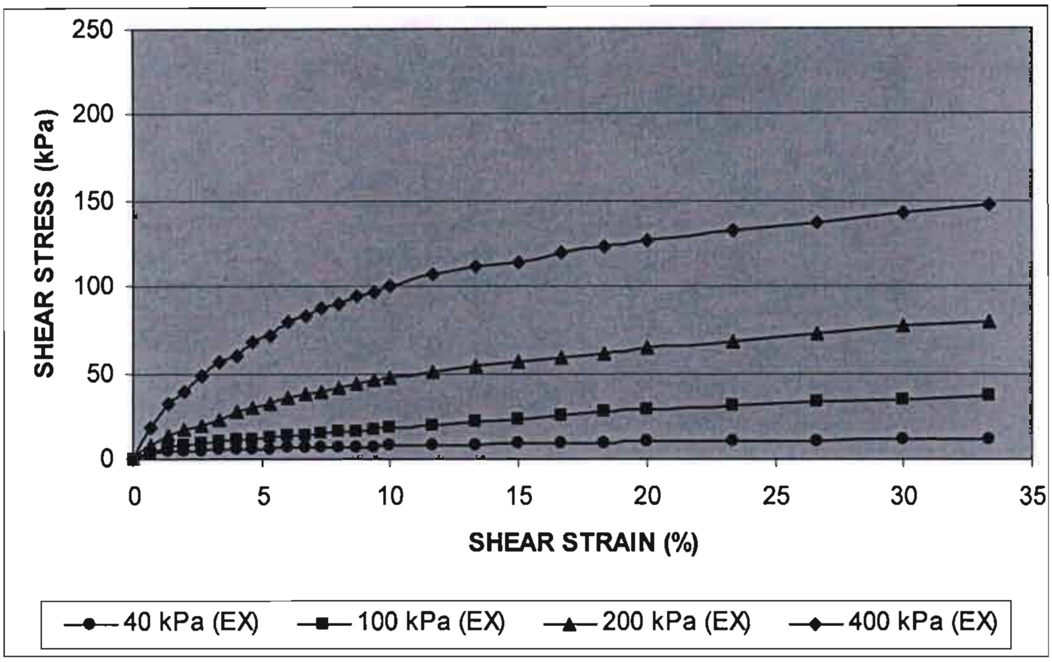


Fig 8.15 Analytically predicted and experimental normal stiffness simple shear stress – strain curves. The normal stiffness of the spring is 250 N/mm.



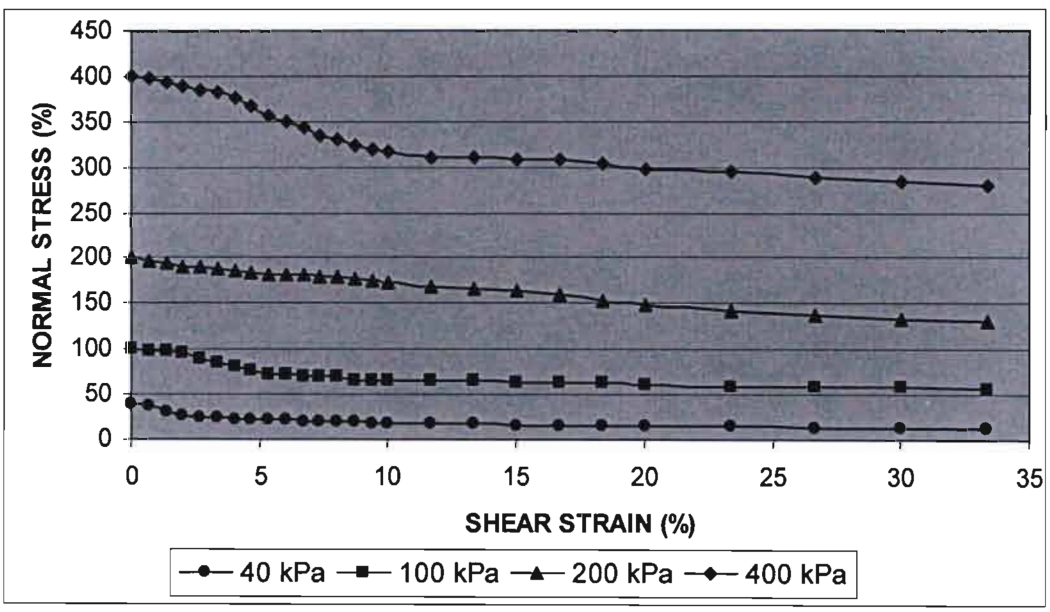


Fig. 8.16 Normal stiffness simple shear stress – strain curves for dry Berea Red Sands. The samples were tested at normal stiffness of 1660N/mm.

The experimental normal stiffness simple shear curves for compacted Berea Red Sands tested at moisture content of 0.2% - 0.6%.

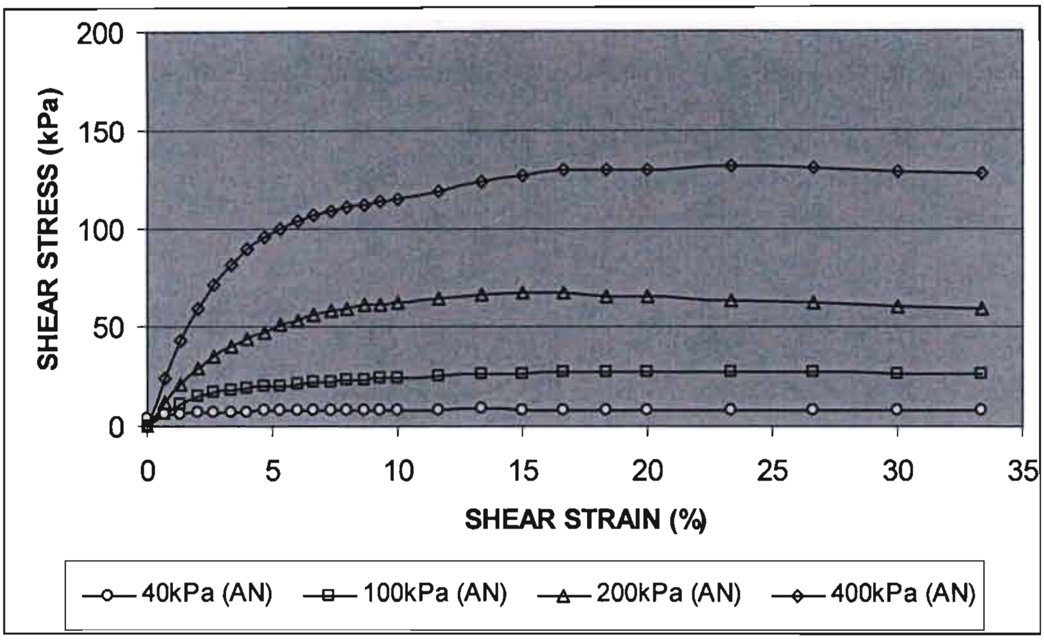


Fig. 8.17 Analytical normal stiffness curves of Berea Red Sands. The simple shear stress – strain curves were generated by substituting the decreasing normal stress values shown in Fig. 8.16 in the constant normal stress equation. The normal stiffness of the spring is 1660 N/mm

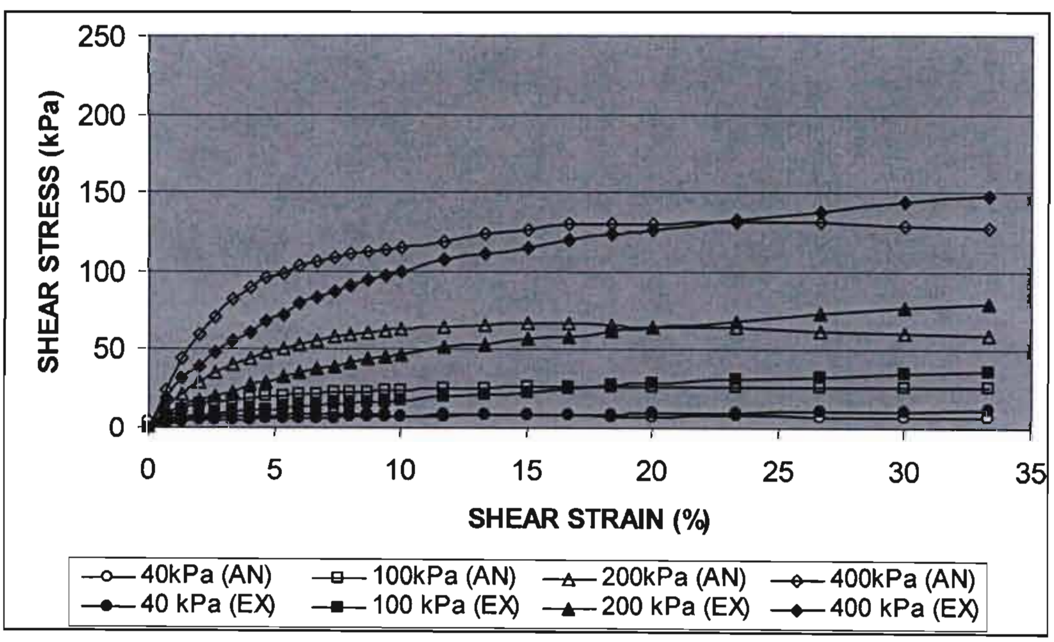


Fig 8.18 Analytical and experimental normal stiffness simple shear stress – strain curves. The normal stiffness of the spring is 1660 N/mm.

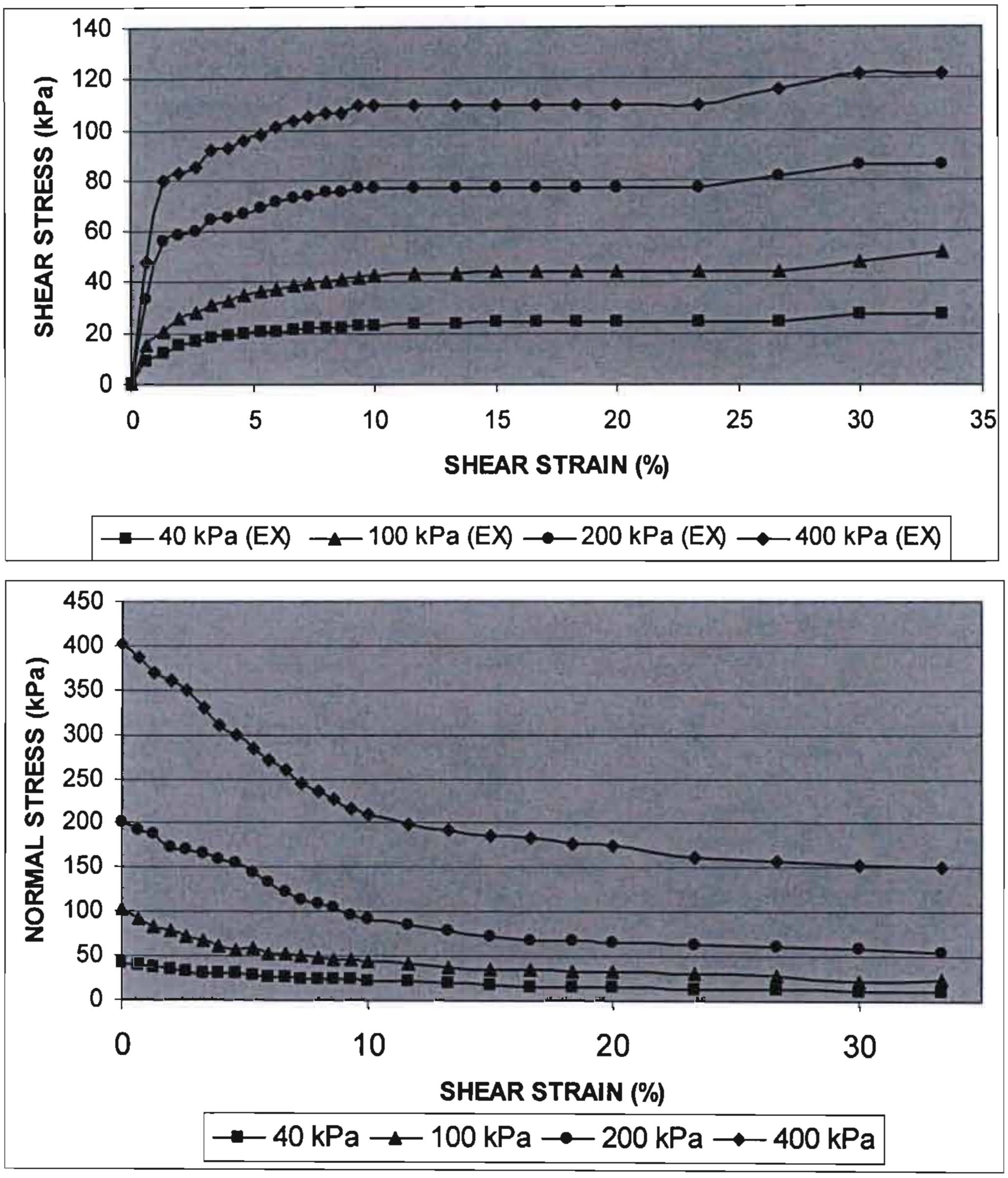


Fig 8.19 Experimental constant volume simple shear stress – strain for compacted dry Berea Red Sands.

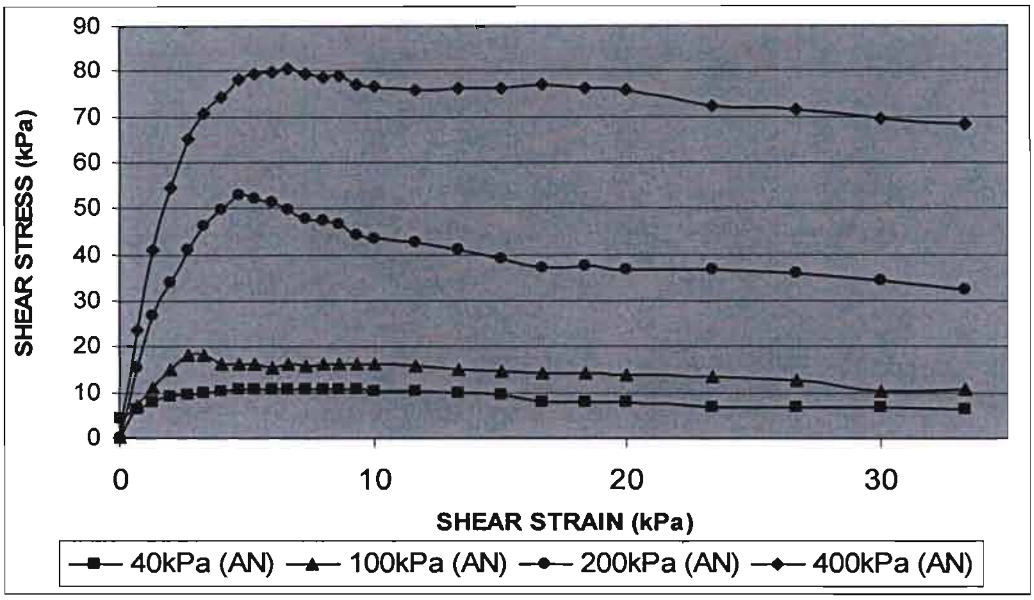


Fig 8.20 Analytical constant volume simple shear stress – strain curves. The simple shear stress – strain curves were modified by substituting the decreasing normal stress values shown in Fig. 8.19 in the constant normal stress equation.

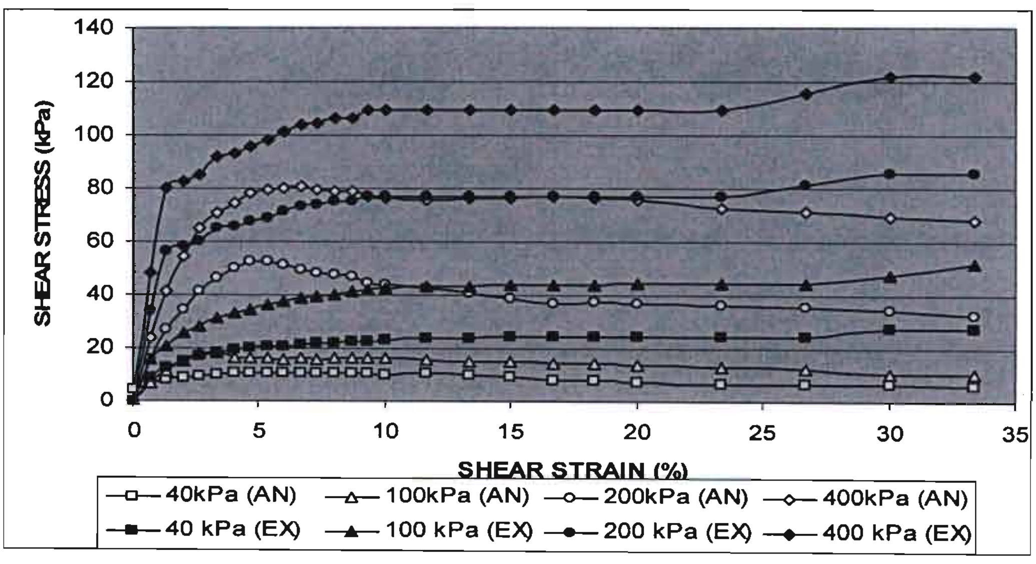


Fig 8.21 Analytical and experimental constant volume simple shear stress – strain curves for Berea Red Sands.

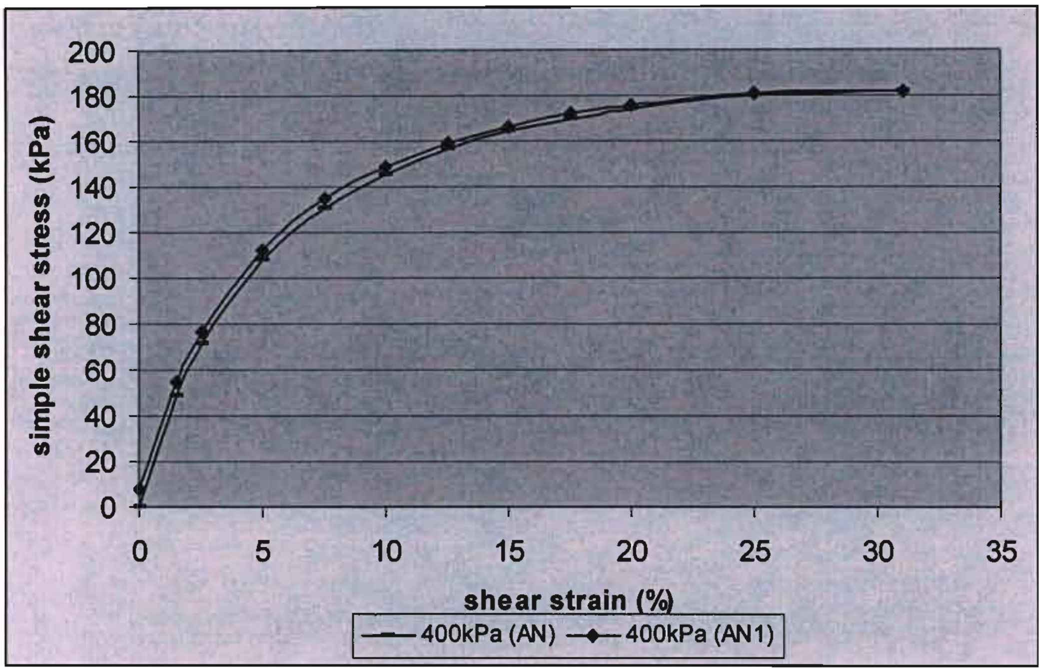


Fig 8.22 Predicted stress stain curves for normal stresses of 400kPa (AN) related to Ko and for normal stress related to 400kPa (AN1) related to 1.2 Ko.

Fig 8.22 demonstrate that the initial stress state of a sample compacted into the simple shear box defined by different values of Ko have no effect on the mobilized shear strength for constant normal stress test condition The curves however indicate that very stiff elastic response is expected at small imposed strain as Ko increases.

CHAPTER NINE

SIMPLE SHEAR BEHAVIOR OF COMPACTED BEREA SANDS

9.1 INTRODUCTION

The effect of changes in moisture content on stress - deformation behavior of compacted Berea Red Sands tested in the conventional direct shear box apparatus has been presented in chapter four. In this chapter the effect of changes in moisture content on the stress deformation behavior of compacted samples of Berea Red Sands that were prepared at the same initial states of water contents and dry densities as the samples tested in the conventional direct shear box apparatus is investigated and the results are compared with the results of the conventional direct shear box tests.

Both the direct shear and simple shear tests were conducted with applied normal stresses within the range consistent with field conditions. The effect of changes in moisture content on normal stiffness simple shear and constant volume simple shear behavior of compacted Berea Red Sands was also investigated. The main types of simple shear tests conducted are relevant to the study of stress state of the soil around a vertically loaded pile.

The filter paper method of matric suction tests were carried out on the soil samples before and after constant normal stress simple shear tests in the new simple shear apparatus. Test procedures, tables of results and stress – strain curves are presented.

The method of sample preparation used for the shear box tests was adopted in the simple shear apparatus. The samples were compacted into the simple shear device to 30mm height and low dry density at relatively high moisture content.

The samples were then dried down to selected lower moisture contents at laboratory temperature and sheared. Series of simple shear tests were carried out using the new apparatus at selected moisture content. All the samples used for the constant normal stress tests, constant volume tests and the normal stiffness tests were prepared by the same method.

9.2 CONSTANT NORMAL STRESS CURVES

Constant normal stress simple shear tests were conducted on a set of samples at constant normal stress of 40 kPa, 100 kPa, 200 kPa and 400 kPa. All the samples are 30mm thick and are not enclosed in a membrane. The samples were compacted into simple shear box to30mm thickness, dry density range of 1404 kg/m^3 – 1424 kg/m^3 and at moisture content range of 16.44% - 18.49%. The samples were then gradually dried down to and tested at the following range of moisture content, 4.2% - 5.08%, 7.07% - 8.25%, 9.47% - 10.82% and 13.35% - 14.24%.

Before and after the shear tests, tests to determine the matric suction of each sample was conducted using Whatmann No 42 filter paper following the method of Chandler and Gutierrez (1988) and Chandler et al, (1992).

The results of the matric suction tests are presented in Fig. 9.14 – Fig. 9.18. The constant normal stress results are presented in Tables 9.2 – 9.5 and Figures 9.2 to 9.5. ; while the tests conducted on very dry samples at moisture content of 0.37% - 0.76%, already reported in chapter eight are again presented in Table 9.1 and Figure 9.1 to facilitate interpretation of the effect of moisture content on the behavior of compacted Berea Sand in simple shear. The shear stress corresponding to 20 % shear strain was taken as the shear strength for all tests reported in this work

No defined peak can be observed from the stress strain curves at the normal stress range and moisture contents used for these tests. The shear stress continued to increase beyond the maximum imposed strain of 33 % used for the tests. The compacted samples mobilized shear resistance due to imposed simple shear strain. The increasing shear stress induces a gradual breakdown of the compacted structure accompanied by volume contraction and subsequent gain in strength.

This behavior is neither unique to sands compacted to low density nor tests conducted in rigid boundary simple shear apparatus. Bjerrum and Landva (1963) reported similar behavior for drained tests on specimen of Norwegian Quick clay in a reinforced membrane simple shear device. The Norwegian Quick clay exhibited bulk volume compression when subject to simple shear strain. A general feature of all tests conducted by Bjerrum and Landva (1963) for samples consolidated at normal stress range of 20 kPa – 400 kPa, was that the shear stresses continued to rise through out the test and consequently a failure value could not be determined.

Dutt et al, (1986) performed drained tests on reconstituted fine bioclastic sand at constant normal stress condition in an NGI type simple shear device. They reported that no clear peak value of shear stress could be identified as the shear stress continued to rise under imposed shear strain.

Other mechanical features revealed by Fig. 9.1 to Fig. 9.5 are that the stiffness and shear stress of compacted Berea Red Sands increases with increase in normal stress and also decrease with increase in moisture content at the same constant normal stress.

The samples also indicated bulk volume compression that varies with the magnitude of applied constant normal stress. The bulk volume changes are defined in terms of vertical strain since the new device allows no change in sample area. Both the total vertical strain and the shear induced vertical strain decreases with increase in normal stress and for samples tested at the same constant normal stress and different moisture content, the total vertical strain and the shear induced vertical strain decreases with increase in test moisture content.

The samples that were tested relatively dry ie at 0.37% - 0.76%, 4.2% - 5.08%, 7.07% - 8.25%, exhibited more compression than samples tested at higher moisture content i.e. 9.47% - 0.82% and 13.35% - 14.24%. The behavior may be associated with the availability of air voids in the drier samples. The findings reveal a dependency of shear induced compression on the magnitude of normal stress and the trend is opposite to that indicated by the samples tested in the direct shear box apparatus. The simple shear deformation behavior is different from the result of the shear box tests where the contractile vertical strain increases with increase in normal stress.

The average simple shear frictional angle of 15 degrees is less than the values given by the direct shear box apparatus.

9.3 NORMAL STIFFNESS CURVES

Series of constant stiffness tests were conducted on samples compacted into the simple shear box to 30mm height, dry density of 1425 kg/m^3 – 1465 kg/m^3 and tested at two moisture content range of 0.2% - 0.8% and 7.55% - 8.45%. For compacted samples tested at the respective moisture content range, initial normal stresses were applied by springs of 140 N/mm, 250 N/mm and 1666 N/mm stiffness

The results of samples tested at moisture content range of 0.2% - 0.8% are presented in Tables 9.6 – 9.9 and Fig 9.6 – 9.9, while the results of samples tested at moisture content range of 7.55% - 8.45% are presented in Tables 9.10 -9.12 and Fig 9.10 – 9.12.

The stress strain curves shown in Fig 9.6 – 9.9 are similar to the constant normal stress curves, with shear stress increasing continuously for applied initial normal stress range and different moisture contents at which the samples were tested. However for the initial stress range used, the measured shear stresses are lower than the constant normal stress plot due to the gradual decrease in the applied initial normal stresses with increase in imposed shear strain.

The mobilized shear stress decreases with increase in imposed normal stiffness for all the samples tested at the two moisture contents Also the measured shear induced vertical strain is lower than that of the constant normal stress tests for corresponding values of normal stresses.

For samples tested at the dry moisture content of 0.2% - 0.8%, the ratio of the normal stress at 20% simple shear strain to the applied initial normal stress, increases with increase in applied initial normal stress. The same trend is observed from curves of samples tested at moisture content of 7.55% - 8.45%. For samples tested at the different moisture contents and the same normal stiffness, the above ratio decreases with increase in moisture content. This implies that shear induced volume compression of compacted Berea Sands increase with increase in moisture content for samples subjected to normal stiffness conditions. The dry samples exhibited very stiff response to imposed simple shear strain.

In relation to the stress condition of the soil close to the pile surface, the constant normal stress condition represents the softest response of the outer elastic soil region. The normal stiffness stress condition represents a relatively stiffer response of the outer elastic soil region. With respect to the pile interface stress conditions, springs stiffer than 140 N/mm will simulate conditions closer to constant volume behavior while springs softer than 140 N/mm will simulate conditions closer to the constant normal stress condition This is clearly indicated by the slope of normal stress curves of samples tested with springs of 1666 N/mm. For the same applied initial normal stress, increase in spring stiffness decreases the mobilized shear stress and increases the reduction in the initial applied normal stress.

The vertical strains and mobilized shear strength of compacted Berea Sand measured in the constant stiffness stress tests using spring of 140 N/mm stiffness are lower than the values recorded for constant normal stress tests for corresponding values of initial normal stress.

9.4 CONSTANT VOLUME CURVES

Owing to the difficulty in the prevention of drainage in direct shear tests, the undrained shear tests were carried out as constant volume tests (Taylor 1953, Bjerrum 1954). During the shear phase of the constant volume tests, the specimens were drained. At the rate of simple shear strain used for the tests, pore pressure build up was prevented through out the shear phase by regulating the applied vertical load in such a way that the vertical height of the sample is maintained constant. Thus the constant volume test is equivalent to an undrained test and the change in applied stress on the specimen is equal to the change in pore pressure that would have occurred in the specimen if the specimen had been prevented from draining for a condition of constant applied vertical stress. That this is the case has already been demonstrated by Taylor (1953) and Bjerrum (1954) in truly undrained tests in which the pore pressure was measured at the base of the specimen.

Since pore pressure buildup is prevented by the combination of slow strain rate and reduction of applied vertical stress, the effective stress in the sample is equal to the applied total stress.

Constant volume simple shear tests were conducted on a set of samples with applied initial normal stress of 40 kPa, 100 kPa, 200 kPa and 400 kPa. All the samples are 30mm thick and are not enclosed in a membrane. The samples were compacted into simple shear box to30mm thickness, dry density range of 1422 kg/m^3 – 1448 kg/m^3 and at moisture content range of 15.00% - 18.03%. The samples were then gradually dried down to and tested at the following range of moisture content, 44% - 0.77%, 4.29% - 4.5% and 7.5% - 8.63%.

The constant volume curves shown in Figs 9.11 - Figs 9.13 are generally bilinear. They all indicated well defined peak values of shear stress at imposed strains of between 8% and 12%. The samples however did not exhibit the post peak drop in shear stress typical of brittle materials; the fairly constant post peak shear stress indicated may be associated with decreasing applied normal stress. The curves also indicated slight increase in shear stress at imposed simple shear strain beyond 25%. This increase in shear stress at strains greater than 25% may not be characteristic of the compacted samples but rather due to the friction or contact pressure associated with the movement of the end plates and the pressure pad in their respective directions.

The above problem would not have any effect on the interpretation of constant volume behavior of compacted Berea Sands as the maximum shear stress was mobilized at strains of between 8% and 12%. It can also be seen from the curves of Figs 9.11 - Figs 9.13 that irrespective of the moisture content at which the samples were tested, the ratio of the maximum shear stress to the initial normal stress indicated by compacted samples of Berea Red Sands decreased with increasing initial normal stress. This implies that the structural rigidity of compacted Berea Sands is significantly destroyed at high initial normal stress.

The changes in effective normal stress acting on the compacted samples of Berea Sands are also shown in Figs 9.11 - Figs 9.13. The change in the effective normal stresses is equivalent to the pore pressure change in any conventional undrained tests. The samples exhibited a general tendency to decrease in volume when sheared and thus the normal stress decreases during the test. For all the samples tested, the change in normal stress observed is in two stages. During the early stage of the tests, the rate of decrease in normal stresses increased rapidly and reaches a maximum at imposed strain of 7%- 10% and this is the strain range at which all the samples tested mobilize most of the shear strength.

Further strain reduces the normal stress continuously but at a steadily decreasing rate. At imposed shear strain of 20%, the normal stress curves of Fig 9.11 - Fig 9.13 indicated that the ratio of the remaining normal stress to the initial normal stress decreased with increase in initial normal stress. This finding again indicate that significant part of the structural rigidity of compacted Berea Sand indicated at early stages of imposed strain was broken down before failure at a rate that increases with increase in initial normal stress applied on the compacted samples.

The reduction in normal stress is observed to vary with respect to moisture content at which compacted Berea Sands is tested and for samples tested at average moisture content of 44% - 0.77%, 4.29% - 4.5% and 7.5% - 8.63%, stiffer response is indicated with increasing moisture content.

Low density Berea sand exhibited mechanical behavior, in constant volume stress condition, similar to that of the behavior of over consolidated soil. The largest stress ratio is indicated by sample tested under the lowest initial normal stress of 40 kPa and decreases with increase initial normal stress. The trend is a clear indication of the effect of magnitude of normal stress on the structural rigidity of compacted Berea sands. At low normal stresses, the structure of compacted sands is not significantly destroyed and so the initial structural rigidity makes a significant contribution to mobilized shear strength. At increased value of applied normal stress, significant part of the compacted structure is destroyed so reduced strength is mobilized. This trend is typical of overconsolidated samples. Thus it can be rightly concluded that at low moisture content, compacted Berea Sand exhibit mechanical behavior typical of over consolidated soils.

Triaxial tests on carbonate sands reported by Poulos and Lee (1988) revealed that peak frictional angle decreases with increase in effective confining pressure. They concluded that the trend is closely associated with measured low poisson value of 0.15 which are typical of naturally cemented soils, dry granular soft soils and dry and soft mud stones. Constant volume tests of Berea sand compacted to low density indicated similar behavior as evident from Fig 9.11 - Fig 9.13.

9.5 MATRIC SUCTION CURVES

The suction tests were performed on samples compacted into the oedometer and shear box to investigate the effect of suction on shear induced volume change in Berea Sand. As the matrix suction is known to influence soil mechanical behavior, only matric suction was measured using the laboratory filter paper techniques of Chandler and Gutierrez (1986) and Chandler et al (1992).

The Chandler et al (1992) filter paper method entails exposing a relatively small piece of Whatman No 42 filter paper to the soil in which the suction is to be measured. Both the soil and the filter paper are placed in an air tight chamber consisting of four layers of plastic bags that were sealed with wax so that stable moisture equilibrium can be established. The tests were conducted in a temperature controlled room where temperature is maintained at 20°. The equilibrium times has been found to be dependent on the mass of filter paper and the temperature (Schreiner and Gourley 1993).

For matric suction measurement, the filter paper and the soil are placed in direct contact. This permits the dissolved salts in the soil pore water to be transferred to the filtered paper along with the soil water. The effect of the salt on the suction at equilibration time is therefore the same in the filter paper as in the soil. Thus only the matrix suction is measured (Schreiner and Gourley 1993).

Following equilibration the filter paper is removed from the sealed chamber and weighed wet inside a small plastic bag, and then oven dried and finally reweighed in the dry condition. The moisture content of the filter paper is then used in the calibration equation of Chandler et al, (1992) to determine the value of the suction. Changes in moisture content of the filter paper disc are measured using a 0.0005g precision Denver balance.

For matric suction measurement, where the paper is in contact with the soil, care must be taken not to apply too great a contact stress. Schreiner and Gourley (1993) indicated that a water content – stress curve for filter paper will show significant reduction in the saturated zero suction water content with increasing contact stress, while Chandler and Gutierrez (1986) claimed that error of up to 50% of the value of suction can be caused due to excessive contact stress. As there is little guidance on the minimum contact stress that will ensure effective contact without permitting squeezing out of the water from the filter paper, a preliminary investigation was deemed necessary.

The mass of filter paper relative to the mass of soil sample affects the value of the measured sample moisture content. Schreiner and Gourley (1993) suggested that to reduce the changes in suction resulting from the moisture removal by the filter paper, it would be necessary to use larger masses of soil or smaller masses of filter paper. The approach of using smaller masses of filter paper was adopted and necessitated the use of a Denver scientific balance with 0.0005g precision.

In the simple shear tests, matric suction tests were conducted on the compacted samples before and at the end of constant normal stress simple shear tests to assess the effect of changes in matrix suction due to shear induced sample compression at constant applied normal stress.
The precision of the filter paper method is dependent on the precision with which the filter paper moisture content can be determined. For a balance with a precision of 0.0005g and small filter

paper dry mass of about 0.35g, error in the opposite direction of the dry mass and wet mass gives an error in the moisture content of $\pm$ 0.14 percentage points (Schreiner and Gourley 1993).

The aim of the matric suction tests is to provide a guide to the range of matric suction values attainable in the field resulting from the fluctuation in in-situ moisture content as well as to assess the contribution of matric suction to the shear strength of compacted Berea Sands deformed in simple shear. In the evaluation of any measurement system, it is desirable to include some means of assessing the accuracy of the techniques. The initial problem associated with the use of the Chandler et al, (1992) filter paper method is the choice of a suitable contact stress as excessive contact stress can result in reduction in filter paper moisture content. Preliminary tests to measure the matric suction of saturated samples under different values of contact stresses were conducted.

The result indicated a decreasing value of moisture content with increasing contact stress for contact stresses higher than 120g. For values less than 120g filter paper moisture content was constant. 100g thin plate of Perspex was adopted as the standard contact stress for the suction studied.

Having established the contact stress, tests to assess whether the filter paper method can measure changes in matric suction resulting from small changes in the volume or height of a soil sample under constant moisture content were conducted. In order to investigate the above problem, matric suction tests were conducted on samples of Berea Sands which were subjected to incremental stages of vertical compression stress. The matric suction tests were conducted on the samples at the end of each compression load stage.

The results are presented in Fig 9.14 - Fig 9.15, and the two figures indicate that the filter paper method can reproduce results when the test procedures are carefully adhered to. The figures show that changes in matric suction of compacted Berea Red Sands at moisture content of above 13% with respect to normal stresses and vertical strain are not significant. The slope of the curves indicate that for the relatively dry samples tested at 7.25% and 7.35% moisture content, the matric suction is extremely sensitive to small changes in saturation.

It was then necessary to carry out preliminary tests to ascertain whether significant changes in matric suction is to be expected from shear induced volume changes in compacted Berea Red

Sands that were tested in direct shear. Matric suction tests were conducted on samples compacted to average moisture content of 7.6% in direct shear box, before and after the direct shear tests.

Fig 9.16 show the change in matric suction of samples following volume compression. The semi log plot indicates that changes in matric suction are independent of the normal stress. The samples were tested at low moisture content permitting very little pore drainage, thus the resistance to deformation offered by matric suction will involve the rearrangement of menisci (Maswoswe 1985). The contractile strain increases with increase in normal stress, thus change in matric suction increases with normal stress since it is controlled by the degree of saturation, that trend was not observed because the volume compression were generally of small magnitude.

At low normal stress range, the response of matric suction to increasing normal stress is not easily predictable, this is probably due to the effect of the relatively small total volume compression of the samples (>2mm) being significantly influenced by the small drainage from the pores. At relatively higher values of normal stresses, the larger total deformation result in significant net increase in degree of saturation and a fairly constant arrangement of menisci. Thus significant change in matric suction is expected from increasing normal stresses yet a constant value was indicated for normal stresses higher than 100 kPa.

Matric suction tests were conducted on samples at different moisture content subjected to constant normal stress conditions in simple shear. The stress - strain curves were presented in figs 9.1 – 9.5. The changes in matric suction due to shear induced volume change for different applied normal stresses and moisture content are shown in Fig 9.17 .Similar trend to that already reported for samples tested in direct shear box was indicated. At normal stresses less than 100 kPa changes in matric suction resulting from shear induced volume compression of samples can not be easily determined as the graphs within this region of normal stress indicated very wide scatter.

The effect of changes in soil matric suction on samples tested at various moisture contents in the new simple shear device was presented in Fig 9.18. In these plots only the change in matrix suction associated to the fairly predictable higher normal stresses are presented.

The general observation from the plots is that matric suction indicated by samples sheared at all moisture content due to soil volume compression in shear decreases with increase in sample moisture content and may be insignificant (>> 10%) for samples with moisture content above 12 %.

9.6 SUMMARY OF MAJOR FINDINGS

The effect of changes in moisture content on constant normal stress simple shear, normal stiffness simple shear and constant volume simple shear behavior of compacted Berea Red Sands was also investigated. The matric suction of each sample was tested using Whatmann No 42 filter paper

For the constant normal stress tests, no defined peak was observed from the stress - strain curves within the range of normal stress and moisture contents used for these tests. The shear stress continued to increase beyond the maximum imposed strain of 33 % used for the series of tests.

The samples also indicated shear induced bulk volume compression that decreases with increase in the magnitude of applied constant normal stress. The shear induced compression behavior in simple shear is thus different from shear induced compression behavior of samples tested in conventional shear box.

Series of constant stiffness tests were conducted on samples compacted into the simple shear box at moisture content of 0.2% - 0.8% and 7.55% - 8.45% and initial normal stiffness of (8.4 MPa) 140 N/mm, (15 MPa) 250 N/mm and (50 MPa) 1666 N/mm.

The stress strain curves are similar to the constant normal stress curves, with shear stress increasing continuously for normal stress range and different moisture contents at which the samples were tested. However for all the stress range used, the measured shear stresses are lower than the constant normal stress plot due to the gradual decrease in the applied initial normal stresses with increase in imposed shear strain.

The vertical strains and mobilized strengths of compacted Berea Sand measured in the constant stiffness stress tests using spring of 140 N/mm stiffness are lower than the values recorded for constant normal stress tests for corresponding values of initial normal stress. It can also be seen from the constant stiffness stress – strain curves that irrespective of the moisture content at which the samples were tested, the ratio of the maximum shear stress to the initial normal stress indicated by compacted samples of Berea Red Sands decreases with increasing initial normal stress. This implies that the structural rigidity of compacted Berea Sands is significantly destroyed at high initial normal stress.

The constant volume curves are generally bilinear. They all indicated well defined peak values of shear stress at imposed strains of between 8% and 12%. The samples however did not exhibit the post peak drop in shear stress typical of brittle materials. The largest stress ratio is indicated by sample tested under the lowest initial normal stress of 40 kPa and decreases with increase initial normal stress. The trend is a clear indication of the effect of magnitude of normal stress on the structural rigidity of compacted Berea sands. At low normal stresses, the structure of compacted sands is not significant destroyed and so the initial structural rigidity makes a significant contribution to mobilized shear strength and stress ratio.

The matric suction tests results indicate that the filter paper method can reproduce results when the test procedure is carefully adhered to. The results show that the change in matric suction of Berea Red Sands at moisture content of above 13% with respect to normal stresses and vertical strain are not significant. The slope of the 7.3% samples indicates that for the relatively dry samples, the matric suction is extremely sensitive to small changes in saturation. Thus it is expected that contribution of matric suction to the mobilized strength of compacted Berea Sands beyond 14% moisture content is insignificant.

Suction tests on the relationship between effective shear stress and matric suction at constant moisture content carried out on 4.2 % moisture content samples in a direct shear box indicates that changes in matric suction is independent of the effective normal stress. At relatively higher values of normal stresses, the larger total deformation result in significant net increase in degree of saturation and a fairly constant arrangement of menisci, thus significant change in matric suction is expected from increasing normal stresses yet a constant value was indicated for normal stresses higher than 100kPa.. Similar trend are also indicated for samples tested at different moisture content in the new simple shear device.

TABLE 9.1

TEST SERIES 9.1: Constant normal stress tests; The experimental results of constant normal stress tests conducted on samples compacted in the simple shear box to 30mm thick, dry density of 1404 kg/m^3 – 1424 kg/m^3 at moisture content of 16.44% - 18.49 % and tested at moisture range of 0.37 % - 0.76 % are given below.

test no	normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test Mg/m^3	experimental shear stress (kPa)	total vertical strain (m/m)	shear induced vertical strain (m/m)
9.1	40	17.66	0.42	1.424	20.82	0.06	0.04
9.2	100	17.22	0.76	1.420	54.00	0.08	0.06
9.3	200	16.77	0.47	1.404	111.66	0.08	0.06
9.4	400	17.33	0.37	1.411	205.33	0.09	0.07

TABLE 9.2

TEST SERIES 9.2: Constant normal stress tests; below The experimental results of constant normal stress tests conducted on 30mm thick samples compacted into the simple shear box at 1401 kg/m^3 – 1454 kg/m^3 dry density and moisture content of 4.20% -5.08% are given.

test no	normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test Mg/m^3	experimental shear stress (kPa)	total vertical strain (m/m)	shear induced vertical strain (m/m)
9.21	40	18.49	4.20	1.4384	20.82	0.06	0.04
9.22	100	17.16	5.08	1.4064	51.26	0.05	0.03
9.23	200	16.44	4.40	1.4540	95.78	0.05	0.03
9.24	400	17.28	4.37	1.4016	203.88	0.04	0.02

TABLE 9.3

TEST SERIES 9.3: Constant normal stress tests; The experimental results of constant normal stress tests conducted on 30 mm thick samples compacted into the simple shear box at 1411 - 1421 kg/m^3 dry density and moisture content of 7.07 %-8.25 % are given below.

test no	normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test Mg/m^3	experimental shear stress (kPa)	total vertical strain (mm/mm)	shear induced vertical strain (mm/mm)
9.31	40	21.42	7.07	1.4112	20.82	0.0633	0.0433
9.32	100	18.32	7.77	1.4195	37.04	0.0500	0.0300
9.33	200	20.48	7.23	1.4210	87.76	0.0500	0.0300
9.34	400	19.86	8.25	1.4195	155.10	0.0400	0.0200

TABLE 9.4

TEST SERIES 9.4: The experimental results of constant normal stress tests conducted on samples compacted into the simple shear box to 30mm height, dry density of 1414 kg/m^3 – 1433 kg/ m^3 and moisture content of 9.47% - 10.82 % are given below.

test no	normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	experimental shear stress (kPa)	total vertical strain (mm/mm)	shear induced vertical strain (mm/mm)
9.41	40	16.31	9.47	1.4285	15.51	0.05	0.03
9.42	100	15.56	9.76	1.4331	40.20	0.05	0.03
9.43	200	16.78	10.82	1.4141	78.98	0.05	0.03
9.44	400	17.20	10.52	1.4285	139.59	0.04	0.02

TABLE 9.5

TEST SERIES 9.5: The experimental results of constant normal stress tests conducted on 30mm thick samples compacted into the simple shear box to dry density of 1419 kg/m^3-1407 kg/m^3 dry density and moisture content of 14.18 % - 17.45% and tested at moisture content of 13.17% - 14.50%.

test no	normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	experimental shear stress (kPa)	total vertical strain (mm/mm)	shear induced vertical strain (mm/mm)
9.51	40	17.45	14.50	1.4542	14.69	0.05	0.03
9.52	100	14.18	13.98	1.4875	37.16	0.04	0.02
9.53	200	16.34	13.17	1.4346	79.38	0.05	0.03
9.54	400	16.56	13.53	1.4195	130.65	0.04	0.02

TABLE 9.6

TEST SERIES 9.6 (NORMAL STIFFNESS TEST): The normal stiffness tests conducted on samples with springs of stiffness constant of 140 N/ mm are given below. The constant normal stress tests were conducted on 30mm thick samples compacted into the simple shear box to dry density of 1411 kg/m^3-1433 kg/m^3 dry density and moisture content of 15.58 % - 16.44 % and tested at moisture content of 0.3% - 0.8%.

test no	Initial normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	experimental shear stress (kPa)	final normal stress (kPa)	total vertical strain (mm/mm)	shear induced vertical strain (mm/mm)
9.51	40	15.77	0.30	1.433	15	20	0.02	0
9.52	100	15.58	0.56	1.422	43	35	0.03	0.01
9.53	200	16.44	0.40	1.411	95	50	0.04	0.02
9.54	400	16.33	0.80	1.417	190	90	0.04	0.02

TABLE 9.6

TEST SERIES 9.6 (NORMAL STIFFNESS TEST): The experimental normal stiffness tests conducted on 30 mm thick samples with springs of stiffness constant of 250 N/ mm are given below. The constant normal stress tests were conducted on 30mm thick samples compacted into the simple shear box to dry density of 1411 kg/m^3-1424 kg/m^3 dry density and moisture content of 16.77 % - 17.55 % and tested at moisture content of 0.24% - 0.57%.

test no	Initial normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	experimental shear stress (kPa)	final normal stress (kPa)	total vertical strain (mm/mm)	shear induced vertical strain (mm/mm)
9.61	40	17.33	0.24	1.412	14	20	0.07	0.05
9.62	100	17.55	0.46	1.411	38	25	0.05	0.03
9.63	200	16.77	0.33	1.407	77	25	0.03	0.01
9.64	400	17.11	0.57	1.424	160	50	0.02	0.00

TABLE 9.7

TEST SERIES 9.7 (NORMAL STIFFNESS TEST): The experimental normal stiffness tests conducted on 30 mm thick samples with springs of stiffness constant of 1660 N/ mm are given below. The constant normal stress tests were conducted on 30mm thick samples compacted into the simple shear box to dry density of 1415 kg/m^3-1427 kg/m^3 dry density and moisture content of 15.55 % - 16.55 % and tested at moisture content of 0.22% - 0.60%.

test no	Initial normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	experimental shear stress (kPa)	final normal stress (kPa)	total vertical strain (mm/mm)	shear induced vertical strain (mm/mm)
9.71	40	15.88	0.40	1.415	12	20	0.05	0.03
9.72	100	15.55	0.22	1.427	29	60	0.04	0.021
9.73	200	16.22	0.55	1.426	62	160	0.03	0.01
9.74	400	16.55	0.60	1.420	136	335	0.03	0.01

TABLE 9.8

TEST SERIES 9.8 (NORMAL STIFFNESS): The experimental results of constant normal stiffness tests conducted on samples compacted into the simple shear box to 30 mm thickness, 1411 kg/m^3 – 1433 kg/ m^3 dry density and moisture content of 15.52% - 16.27 % and tested at moisture content of 7.55% - 8.67% with springs of normal stiffness of 140 N/mm are given below.

test no	initial normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	experimental shear stress (kPa)	final normal stress (kPa)	total vertical strain (mm/mm)	shear induced vertical strain (mm/mm)
9.81	40	16.27	8.67	1.422	16.33	14.71	0.03	0.01
9.82	100	15.52	7.55	1.433	33.67	71.18	0.03	0.01
9.83	200	15.70	8.37	1.422	67.35	151.68	0.04	0.02
9.84	400	15.77	7.88	1.411	139.98	340	0.04	0.02

TABLE 9.10

TEST SERIES 9.10 (NORMAL STIFFNESS): The experimental results of constant normal stiffness tests conducted on samples compacted into the simple shear box to 30 mm thickness, 1405 kg/m^3 – 1466 kg/m^3 dry density and moisture content of 15.61% - 16.04 % and tested at moisture content of 7.51% - 8.59% with springs of normal stiffness of 250 N/mm are given below.

test no	normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	experimental shear stress (kPa)	decrease in normal stress (kPa)	total vertical strain (mm/mm)	shear induced vertical strain (mm/mm)
9.101	40	16.04	8.59	1.4054	15.18	13.98	0.02	0
9.102	100	15.61	7.51	1.4610	24.29	53.57	0.03	0.01
9.103	200	15.68	8.30	1.4559	47.96	140.82	0.04	0.02
9.104	400	15.69	7.81	1.4669	132.98	277.63	0.04	0.02

TABLE 9.11

TEST SERIES 9.11 (NORMAL STIFFNESS): The experimental results of constant normal stiffness tests conducted on samples compacted into the simple shear box to 30 mm thickness, 1426 kg/m^3 – 1475 kg/ m^3 dry density and moisture content of 15.52% - 16.27 % and tested at moisture content of 7.51% - 8.65% with springs of normal stiffness of 1660 N/mm are given below.

test no	Initial normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	experimental shear stress (kPa)	final normal stress (kPa)	total vertical strain (mm/mm)	shear induced vertical strain (mm/mm)
9.111	40	16.27	8.67	1.4757	8.78	6.2	0.02	0
9.112	100	15.52	7.55	1.4605	22.65	30	0.03	0.01
9.113	200	15.70	8.37	1.4379	40.20	87.8	0.04	0.02
9.114	400	15.77	7.88	1.4267	117.40	177	0.04	0.02

TABLE 9.12

TEST SERIES 9.12 (CONSTANT VOLUME TEST): The experimental results of constant volume tests conducted on 30 mm thick samples compacted into the simple shear box to 1434 kg/m^3 - 1442 kg/m^3 dry density and moisture content of 15.04% - 15.63% and tested at moisture content of 0.44 % - 0.77%

test no	Initial normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	experimental shear stress (kPa)	Final. normal stress (kPa)
9.121	40	15.44	0.66	1.4427	23.61	18.10
9.122	100	15.63	0.77	1.4341	43.55	33.84
9.123	200	15.34	0.55	1.4352	78.44	65.45
9.124	400	15.04	0.44	1.4416	110.44	174.49

TABLE 9.13

TEST SERIES 9.13 (CONSTANT VOLUME TEST): The experimental results of constant volume tests conducted on samples compacted into the simple shear box to 30 mm height at 1422 kg/m^3 - 1433 kg/m^3 dry density at moisture content of 15.55% - 16.99% and tested at moisture content of 4.29% - 4.50%

test no	Initial normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	experimental shear stress (kPa)	final normal stress (kPa)
9.131	40	16.55	4.38	1.433	18.61	15.10
9.132	100	16.77	4.29	1.422	32.14	25.84
9.133	200	15.880	4.33	1.422	53.88	53.45
9.134	400	16.99	4.50	1.431	90.90	104.49

TABLE 9.14

TEST SERIES 9.14 (CONSTANT VOLUME TEST): The experimental results of constant volume tests conducted on samples compacted into the simple shear box to 30 mm thickness at 1431 kg/m^3 - 1464 kg/m^3 dry density and moisture content of 15.58% - 16.83% and tested at moisture content 7.50% - 8.63%

test no	Initial normal stress (kPa)	moisture content after compaction (%)	moisture content after test (%)	dry density before test (Mg/m^3)	experimental shear stress (kPa)	final normal stress (kPa)
9.141	40	15.15	7.88	1.4822	18.33	15.10
9.142	100	16.83	8.21	1.4641	28.14	31.84
9.143	200	15.85	8.63	1.4452	45.88	62.45
9.144	400	15.58	7.50	1.4316	82.90	104.49

TABLE 9.15

TEST SERIES 9.15 (MATRIC SUCTION TEST): The experimental results of matric suction tests of samples compacted into the direct shear box at 1401 kg/m^3 dry density and subjected to different vertical compression stresses and moisture contents of 7.25 % and 7.35 %

test no.	normal stress (kPa)	vertical strain at 7.25 % moisture content (mm)	matric suction (kPa)	vertical strain at 7.35 % moisture content (mm)	matric suction (kPa)
9.151	0	0	402.2	0	484.94
9.152	40	4.73	373.52	3.16	396.83
9.153	100	7.66	331.19	9.33	297.21
9.154	400	12.53	234.33	13.7	237.84

TABLE 9.16

TEST SERIES 9.16 (MATRIC SUCTION TEST): The experimental results matric suctions of samples compacted into the simple shear box at 1403 kg/m^3 dry density and subjected to different vertical compression stresses and moisture contents of 13.5 % and 13.35 %

test no.	normal stress (kPa)	vertical strain at 13.5 % moisture content (mm)	matric suction (kPa)	vertical strain at 13.5 % moisture content (mm)	matric suction (kPa)
9.163	0	0	132.47	0	149.3
9.163	40	5.2	96.73	5.03	114.83
9.163	100	8.13	66	9.80	60.25
9.164	400	16.80	51.13	15.40	50.84

TABLE 9.17

TEST SERIES 9.17 (MATRIC SUCTION TEST): The experimental results matric suctions of samples compacted into the simple shear box at 1403 kg/m^3 dry density and subjected to different vertical compression stresses and moisture contents of 13.5 % and 13.35 %

test no.	normal stress (kPa)	vertical strain at 13.5 % moisture content (mm)	matric suction (kPa)	vertical strain at 13.5 % moisture content (mm)	matric suction (kPa)
9.171	0	0	132.47	0	149.3
9.172	40	5.2	96.73	5.03	114.83
9.173	100	8.13	66	9.80	60.25
9.174	400	16.80	51.13	15.40	50.84

TABLE 9.18

TEST SERIES 9.18 (MATRIC SUCTION TEST): The experimental results of matric suctions of samples compacted into the direct shear box at 1403 kg/m^3 dry density and 4.3% moisture content and sheared under different normal stresses

test no.	normal stress (kPa)	matric suction before shear box test (kPa)	matric suction after shear box test (kPa)	direct shear stress at 8mm displacement (kPa)	moisture content after shear test (%)
9.181	20	1516	804.8	25	3.94
9.182	40	917	731	41	3.97
9.183	60	704	627	57	4.01
9.184	100	718	538	84	4.20
9.185	200	910	701	165	4.26
9.186	400	635	567	321	4.21

TABLE 9.19

TEST SERIES 9.19 (MATRIC SUCTION TEST): The experimental results of matric suctions of samples compacted into the simple shear box at 1403 kg/m^3 dry density and 7.3 % moisture content and sheared under different normal stresses

Test No	Normal stress (kPa)	Matric suction Before shear box test (kPa)	Matric suction after shear box test (kPa)	Simple shear stress at 22% Strain (kPa)	Moisture content after shear test (%)
9.191	40	171.25	189.45	39	7.21
9.192	100	460.79	386.53	88	7.20
9.193	200	290.45	243.31	168	7.26
9.194	400	281.88	130.43	290	7.21

TABLE 9.20

TEST SERIES 7.24 (MATRIC SUCTION TEST): The experimental results of matric suctions of samples compacted into the simple shear box at 1403 kg/m^3 dry density and 10.35 % moisture content and sheared under different normal stresses

Test No	Normal stress (kPa)	Matric suction Before shear box test (kPa)	Matric suction after shear box test (kPa)	Simple shear stress at 22% Strain (kPa)	Moisture content after shear test (%)
9.201	40	24.26	28.04	36.94	10.21
9.202	100	15.13	10.38	76.5	10.20
9.203	200	17.35	7.29	158.00	10.26
9.204	400	42.48	9.94	251.02	10.21

TABLE 9.21

TEST SERIES 9.21 (MATRIC SUCTION TEST): The experimental results of matric suctions of samples compacted into the direct shear box at 1403 kg/m^3 dry density and 13.35% moisture content and sheared under different normal stresses

Test No	Normal stress (kPa)	Matric suction Before shear box test (kPa)	Matric suction after shear box test (kPa)	Simple shear stress at 22% Strain (kPa)	Moisture content after shear test (%)
9.211	40	6.23	5.37	27.36	13.21
9.212	100	5.22	5.64	72.2	13.20
9.213	200	6.03	5.58	144.42	13.26
9.214	400	6.54	5.62	237.44	13.21

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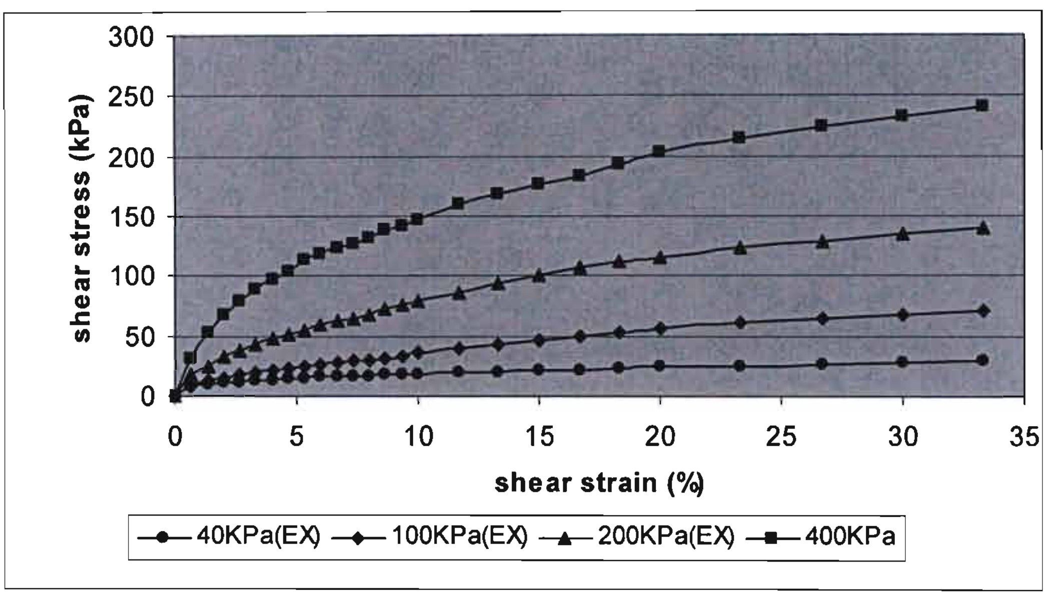


300
250
200
150
100
50
0
shear stress (kPa)
0
5
10
15
20
25
30
35
shear strain (%)
40KPa(EX)
100KPa(EX)
200KPa(EX)
400KPa

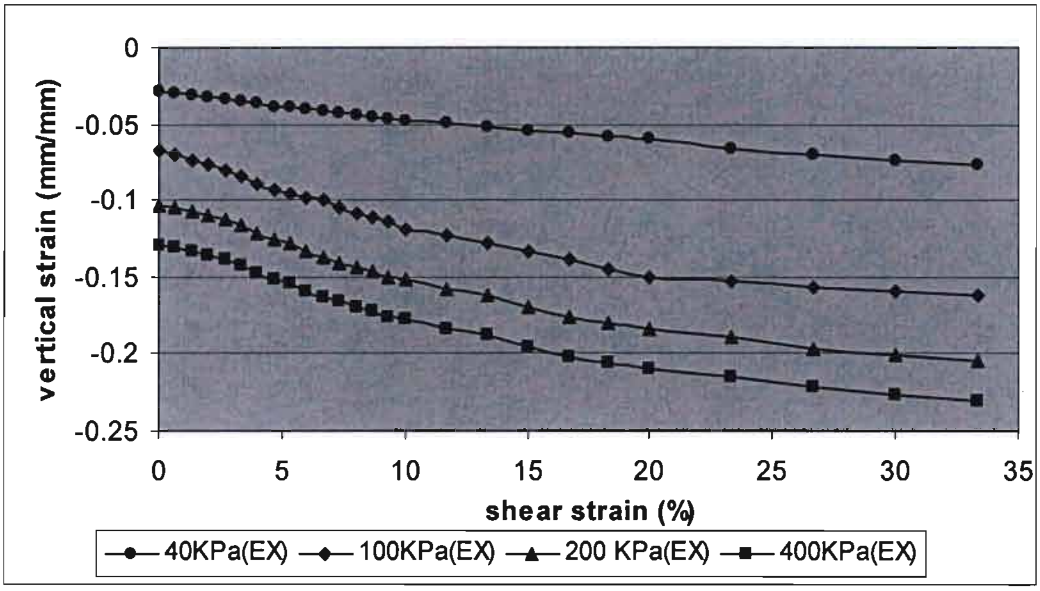


Fig 9.1 Experimental (EX) constant normal stress simple shear stress – strain curves of compacted Berea Red Sands The constant normal stress tests was conducted on samples compacted in the simple shear box to30mm thick, dry density of 1404kg/m^3 – 1424 kg/m^3 at moisture content of 16.44% - 18.49% and tested at moisture range of 0.37% - 0.76%

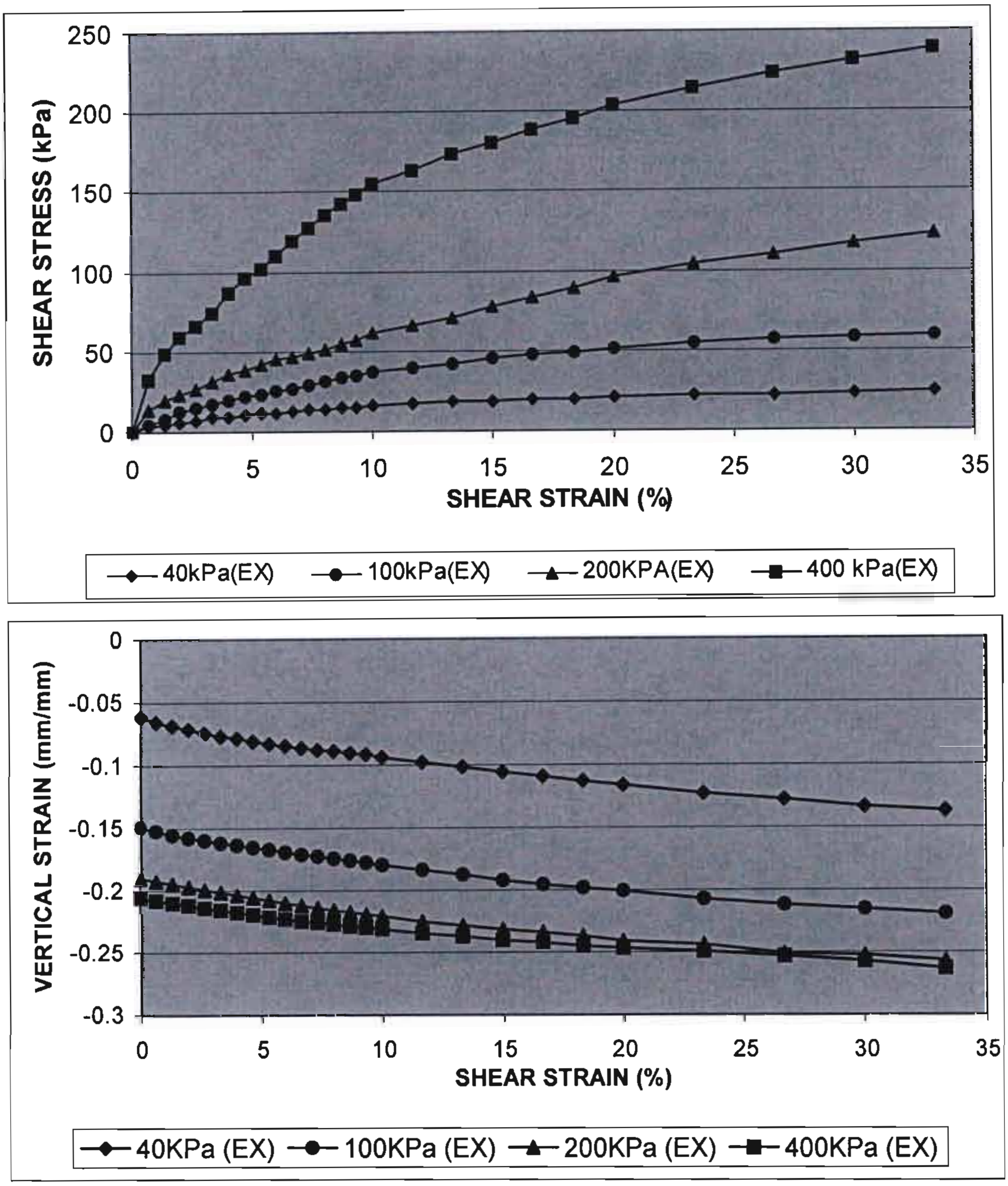


FIG 9.2: Constant normal stress simple shear curves of compacted Berea Sands. The experimental results of constant normal stress tests conducted on 30mm thick samples compacted into the simple shear box at 1401 kg/m^3 – 1454 kg/m^3 dry density and moisture content of 4.20% -5.08%

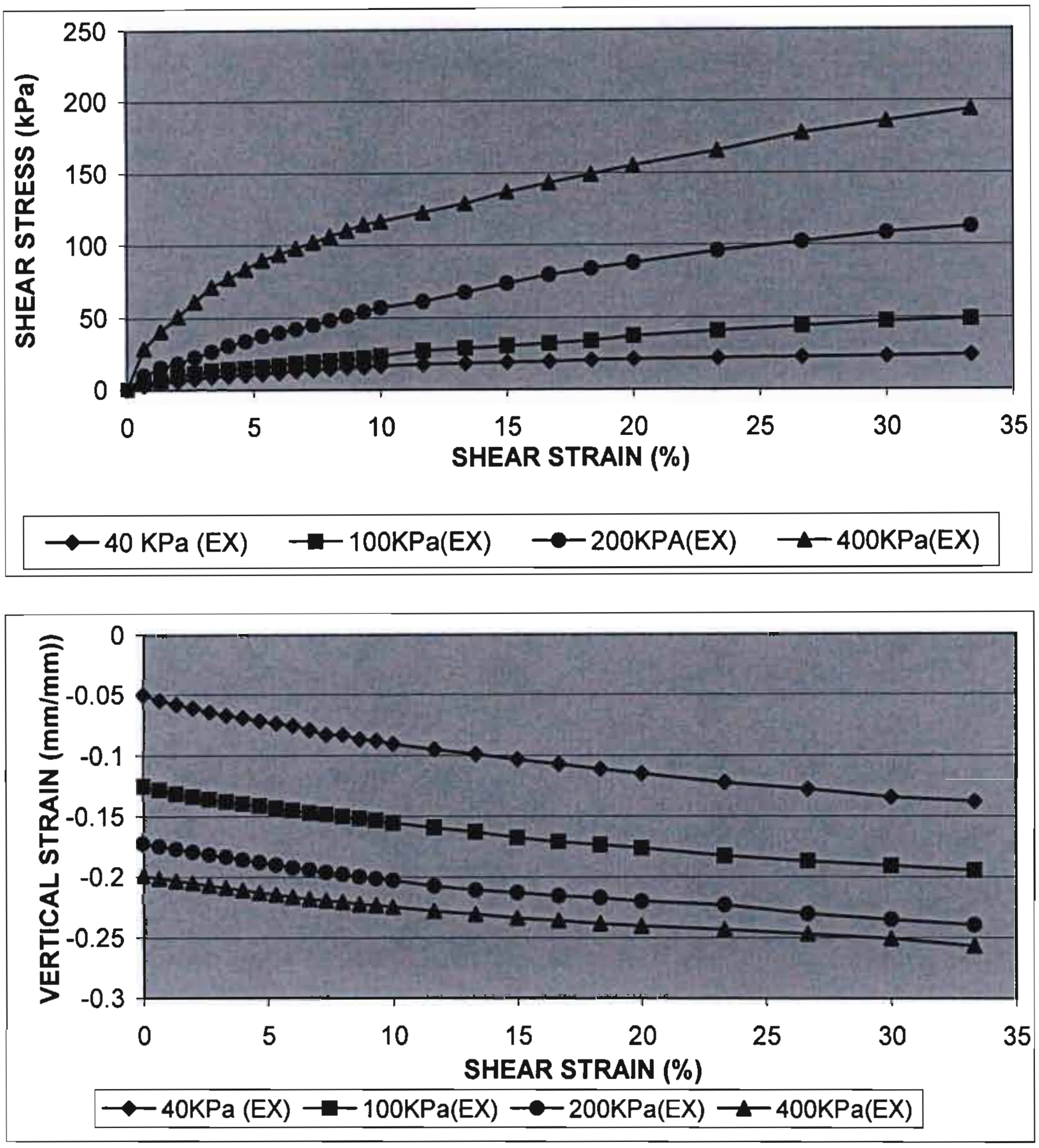
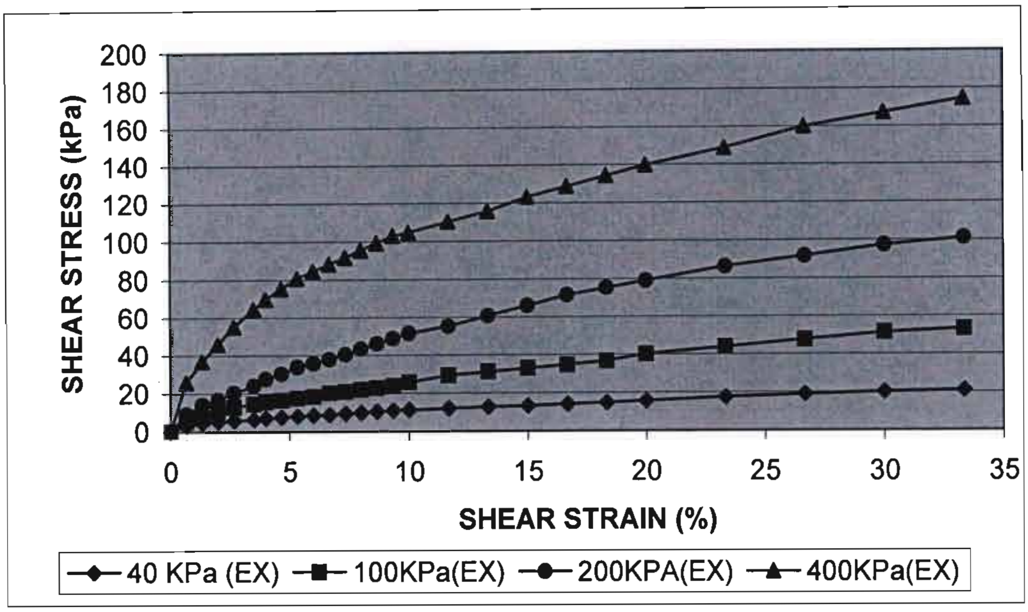


FIG 9.3: Experimental (EX) constant normal stress simple shear curves of compacted Berea Sands. Constant normal stress tests; The experimental results of constant normal stress tests conducted on 30 mm thick samples compacted into the simple shear box at 1411 - 1421 kg/m^3 dry density and moisture content of 7.07 %-8.25%



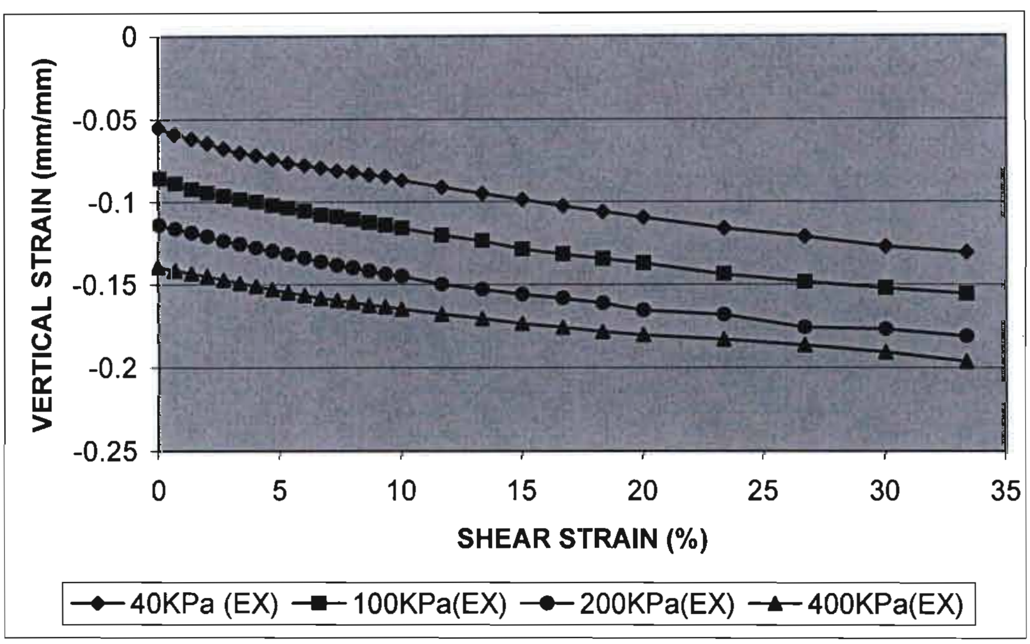
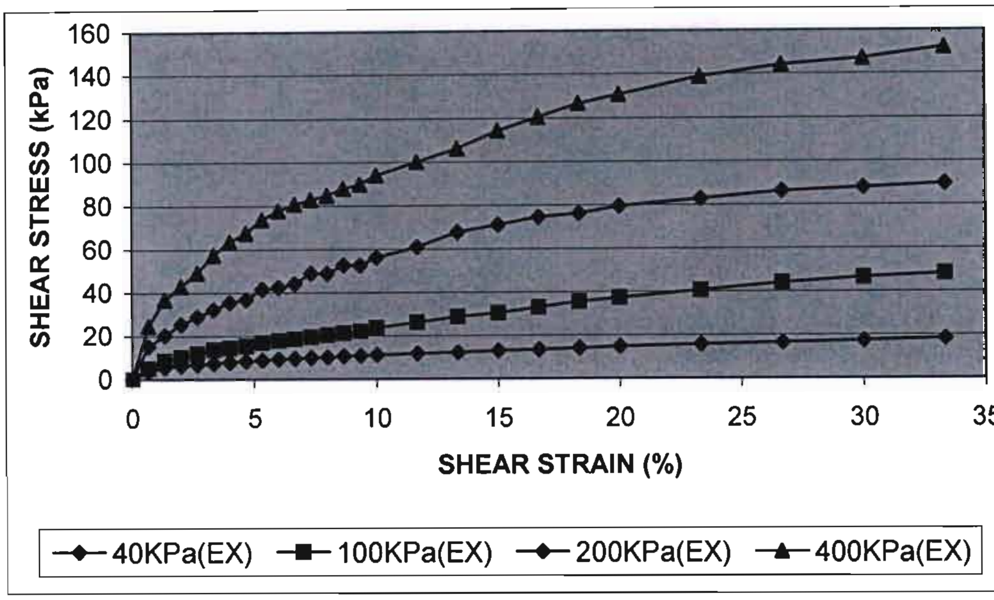


FIG 9.4: The experimental results of constant normal stress tests conducted on samples compacted into the simple shear box to 30mm height, dry density of 1414 kg/m^3 – 1433 kg/ m^3 at moisture content of 9.47% - 10.82 % are given below.



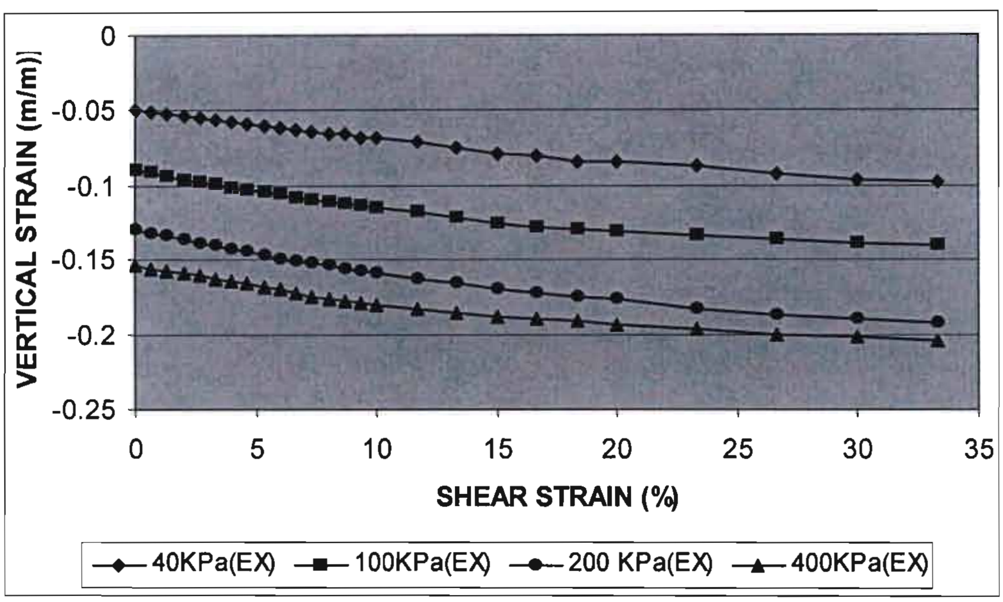
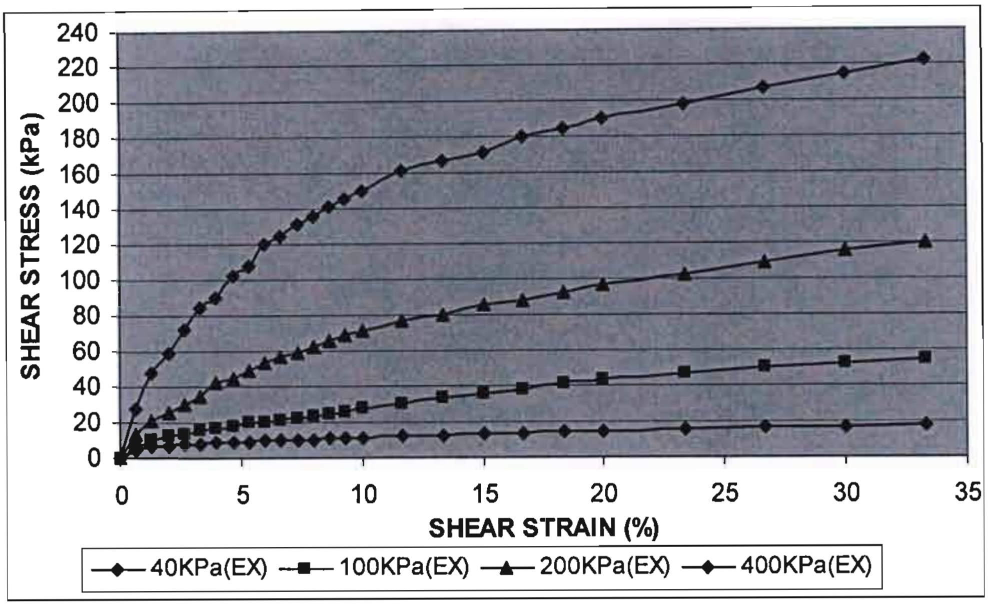


FIG 9.5: The experimental results of constant normal stress tests conducted on 30mm thick samples compacted into the simple shear box to dry density of 1419 kg/m^3-1447 kg/m^3 dry density and moisture content of 14.18 % - 17.45% and tested at moisture content of 13.17% - 14.50%.



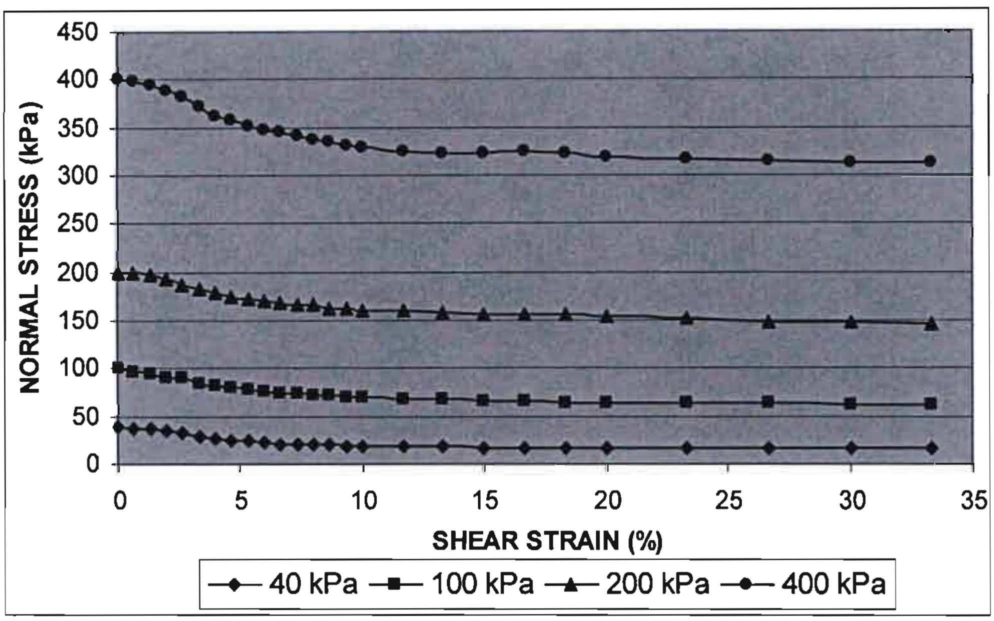


Fig. 9.6: Normal stiffness simple shear stress – strain curves for dry Berea Red Sands. The samples were tested at normal stiffness of 140N/mm. The constant normal stress tests were conducted on 30mm thick samples compacted into the simple shear box to dry density of 1411 kg/m^3-1433 kg/m^3 dry density and moisture content of 15.58 % - 16.44 % and tested at moisture content of 0.3% - 0.8%.

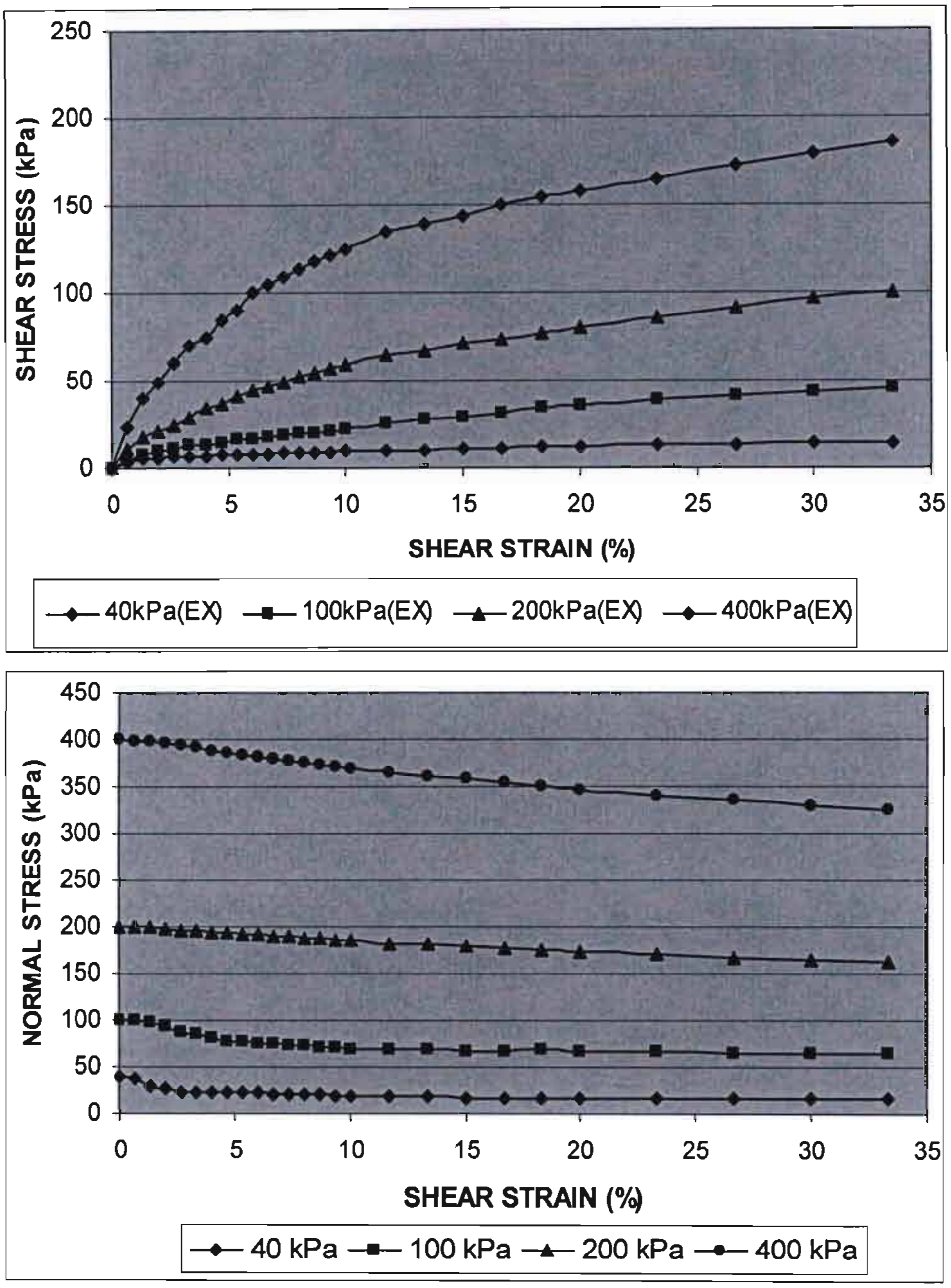
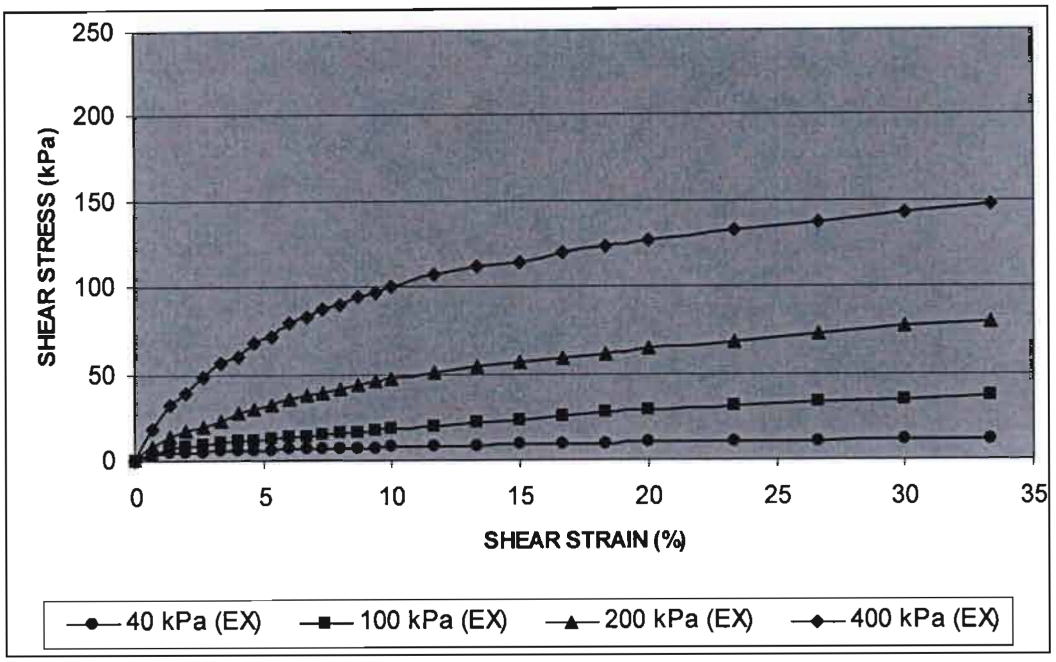


Fig. 9.7: Normal stiffness simple shear stress – strain curves for dry Berea Red Sands. The samples were tested at normal stiffness of 250N/mm. The constant normal stress tests were conducted on 30mm thick samples compacted into the simple shear box to dry density of 1411 kg/m^3-1424 kg/m^3 dry density and moisture content of 16.77 % - 17.55 % and tested at moisture content of 0.24% - 0.57%.



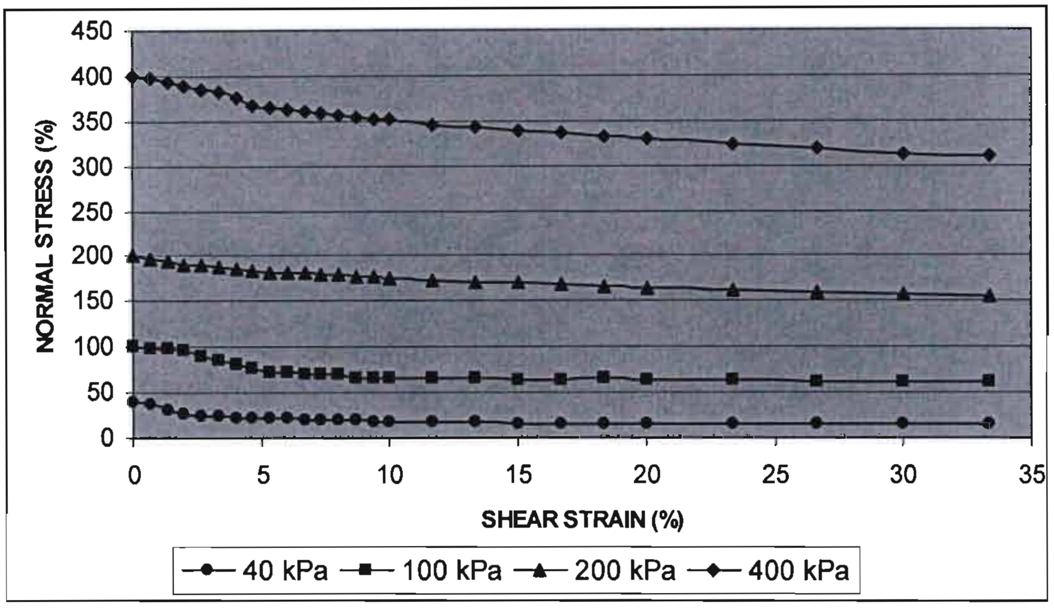


Fig. 9.8: Normal stiffness simple shear stress – strain curves for dry Berea Red Sands. The samples were tested at normal stiffness of 1660N/mm.
The constant normal stress tests were conducted on 30mm thick samples compacted into the simple shear box to dry density of 1415 kg/m^3-1427 kg/m^3 and moisture content of 15.55 % - 16.55 % and tested at moisture content of 0.22% - 0.60%.

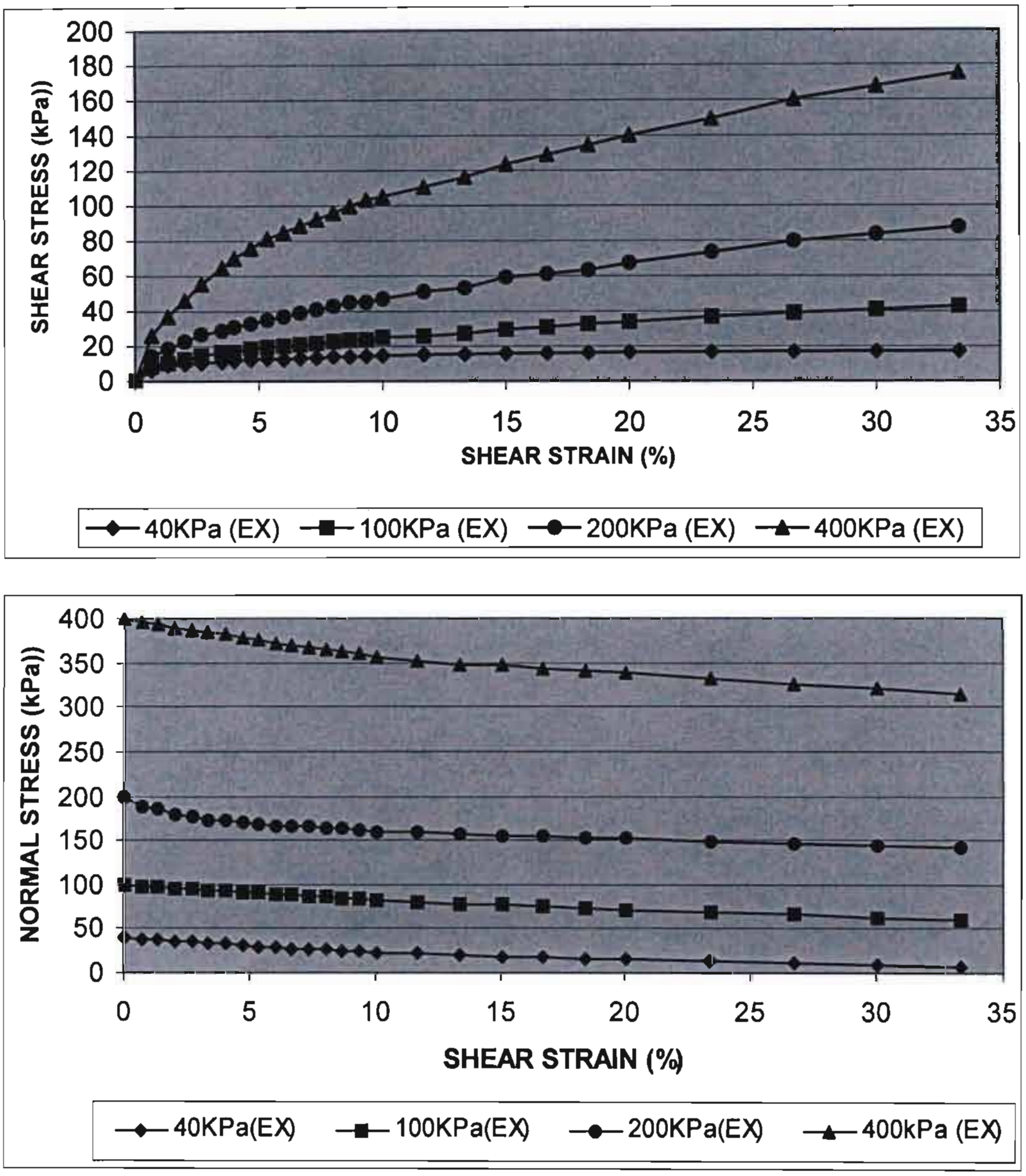
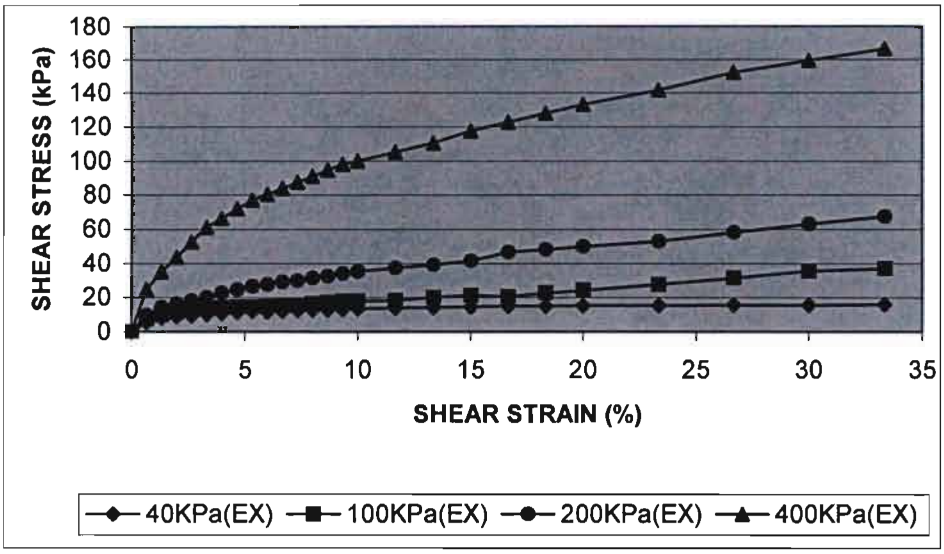


FIG 9.9: The experimental curve of constant normal stiffness tests conducted on samples compacted into the simple shear box to 30 mm thickness, 1411 kg/m^3 – 1433 kg/m^3 dry density and moisture content of 15.52% - 16.27 % and tested at moisture content of 7.55% - 8.67% with springs of normal stiffness of 140 N/mm



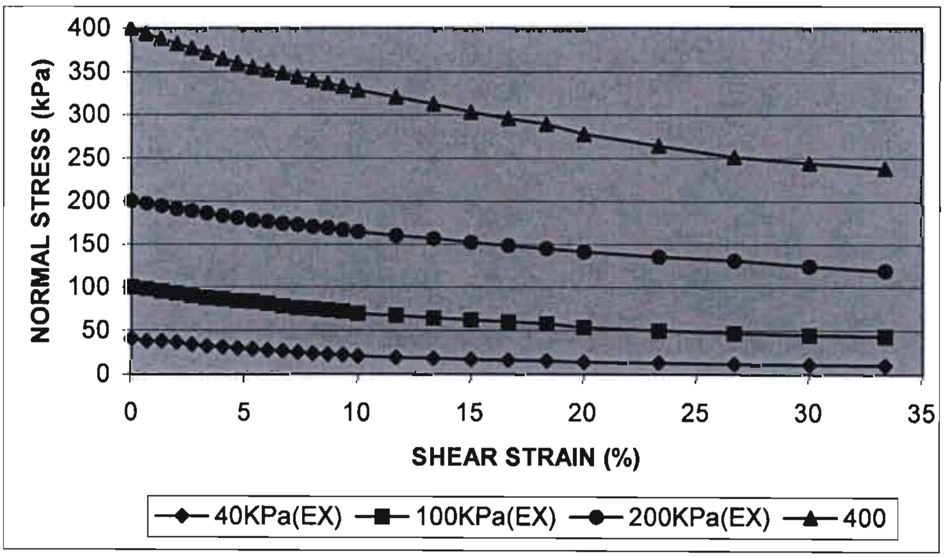
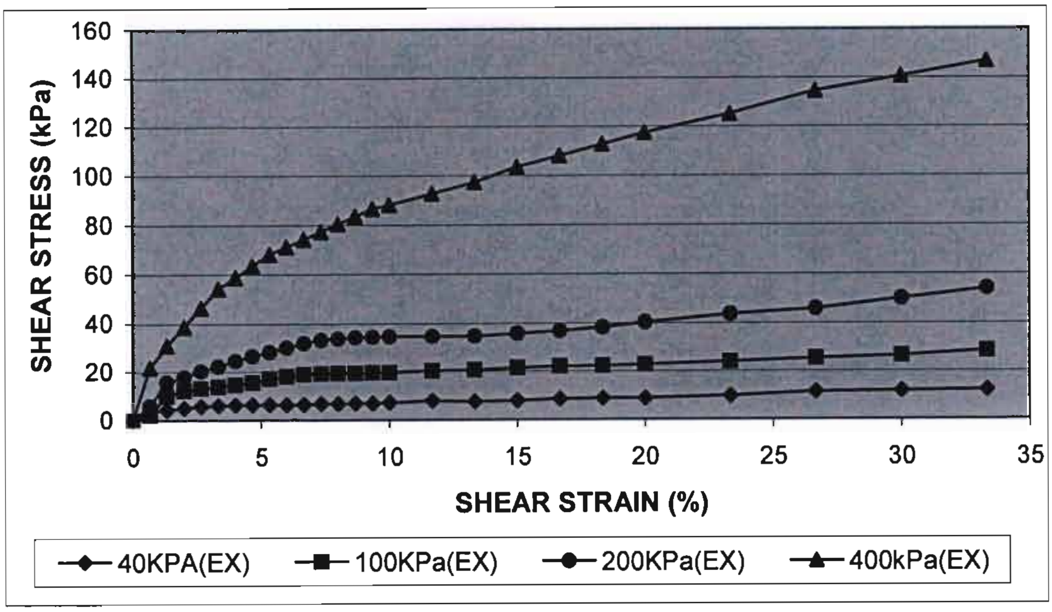


FIG 9.9: The experimental results of constant normal stiffness tests conducted on samples compacted into the simple shear box to 30 mm thickness, 1405 kg/m^3 – 1466 kg/m^3 dry density and moisture content of 15.61% - 16.04% and tested at moisture content of 7.51% - 8.59% with springs of normal stiffness constant of 250 N/mm



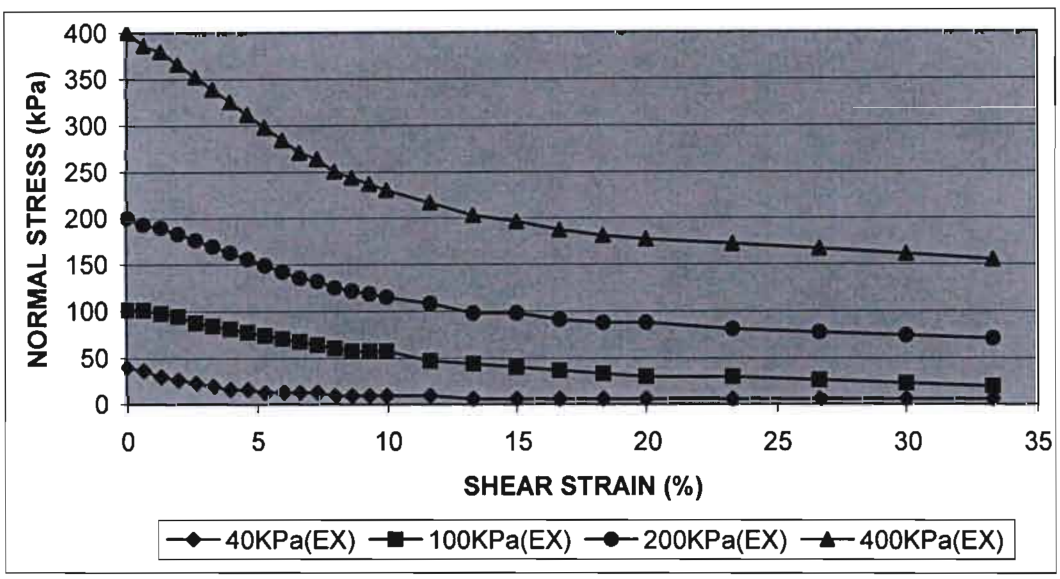
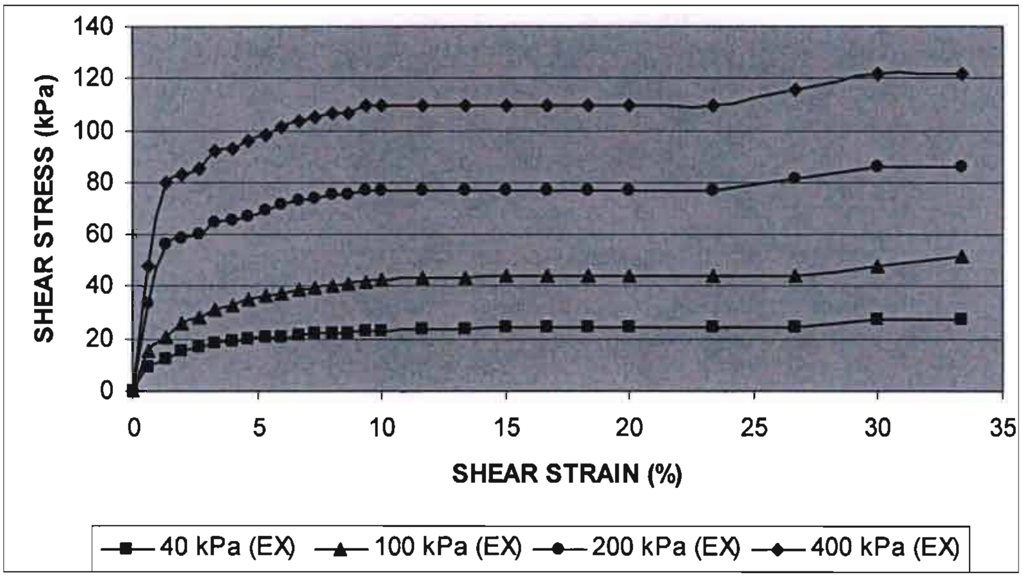


FIG 9.10: The experimental curve of constant normal stiffness tests conducted on samples compacted into the simple shear box to 30 mm thickness, 1426 kg/m^3 – 1475 kg/ m^3 dry density and moisture content of 15.52% - 16.27 % and tested at moisture content of 7.51% - 8.65% with springs of normal stiffness of 1660 N/mm



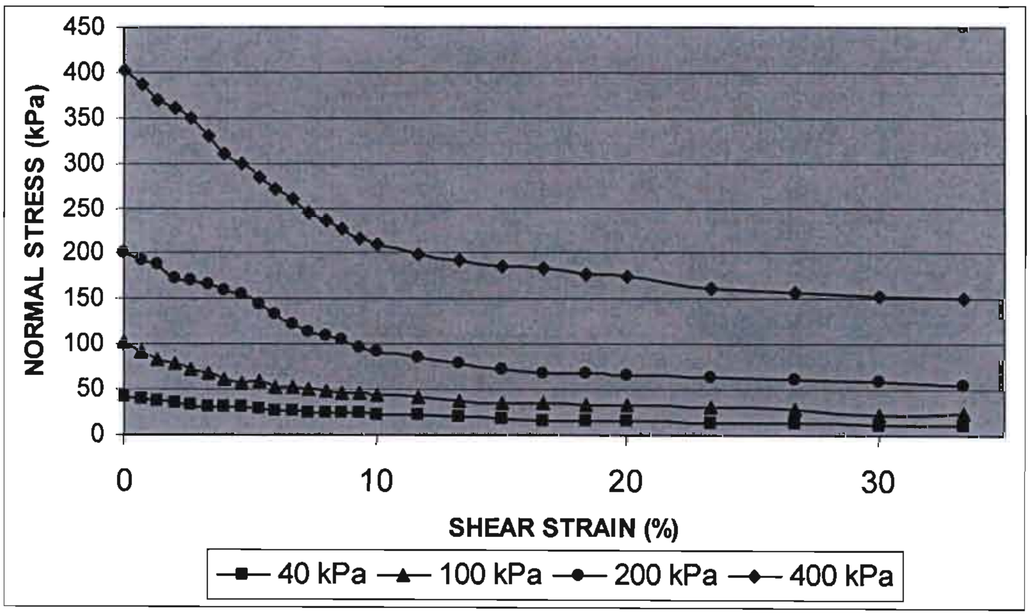


Fig 9.11 The experimental results of constant volume tests conducted on 30 mm thick samples compacted into the simple shear box to 1434 kg/m^3 - 1442 kg/m^3 dry density and moisture content of 15.04% - 15.63% and tested at moisture content of 0.44 % - 0.77% .

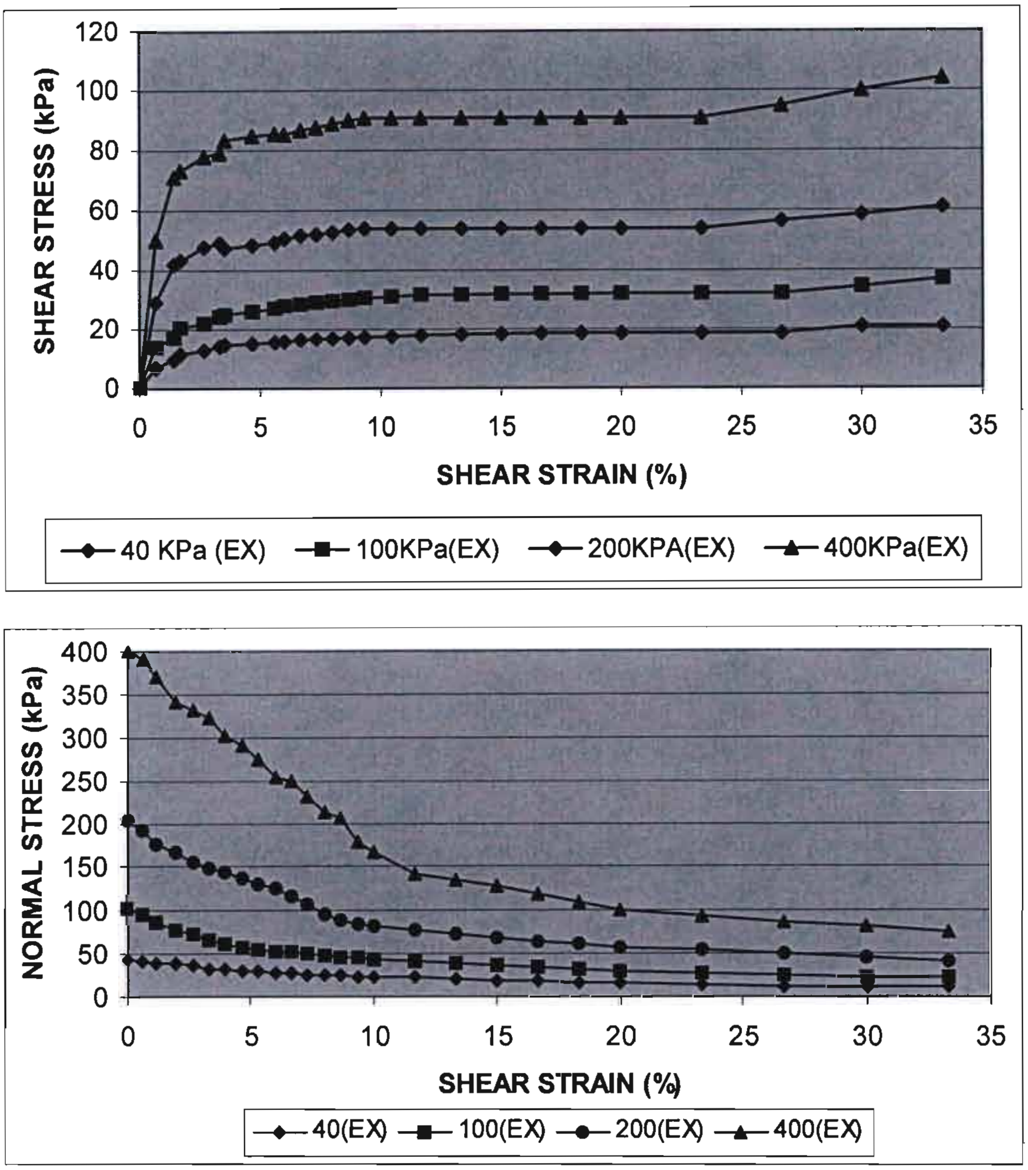
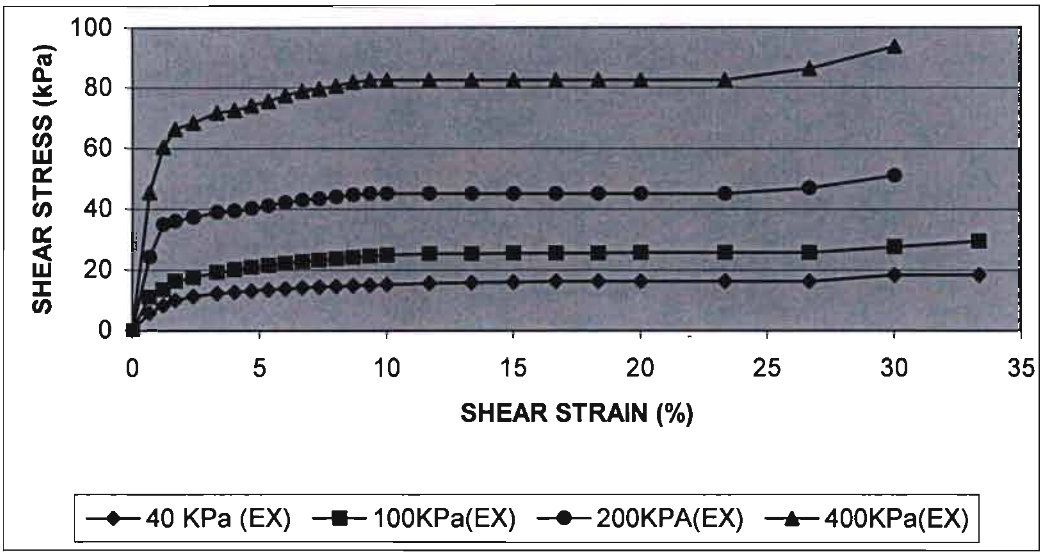


FIG 9.12: The experimental curve of constant volume tests conducted on samples compacted into the simple shear box to 30 mm height at 1422 kg/m^3 - 1433 kg/m^3 dry density at moisture content of 15.55% - 16.99% and tested at moisture content of 4.29% - 4.50%



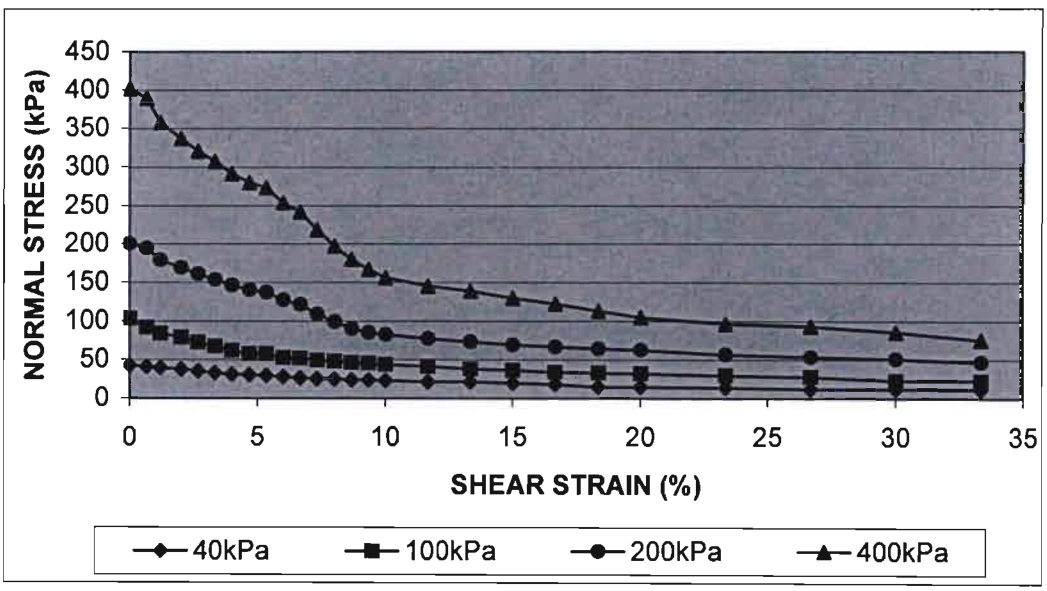


FIG 9.13: The experimental curve of constant volume tests conducted on samples compacted into the simple shear box to 30 mm thickness at 1431 kg/m^3 - 1464 kg/m^3 dry density and moisture content of 15.58% - 16.83% and tested at moisture content 7.50% - 8.63%

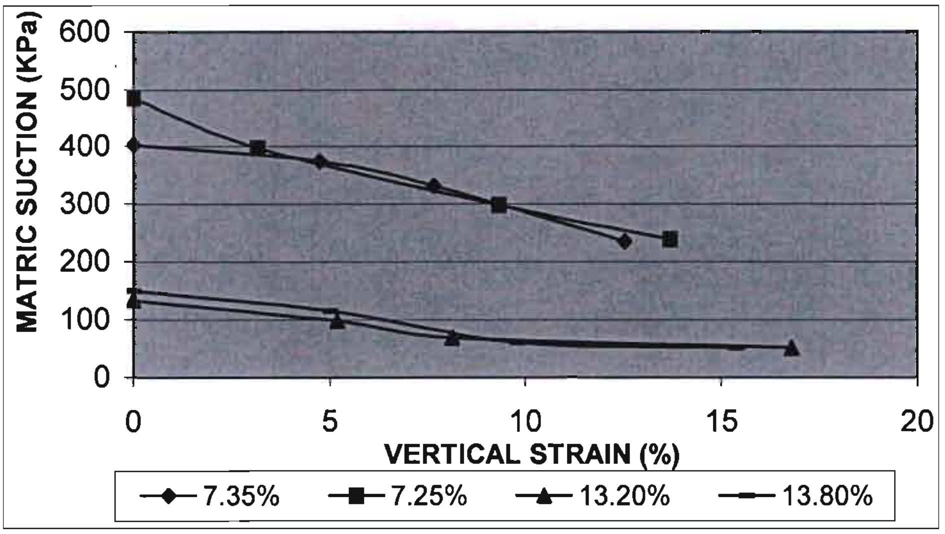


Fig 9.14: Matric suction – soil compressibility curves of Berea Red Sands compacted to dry density of 1403 kg/m^3 and average moisture contents of 7.30 % and 13.50 %

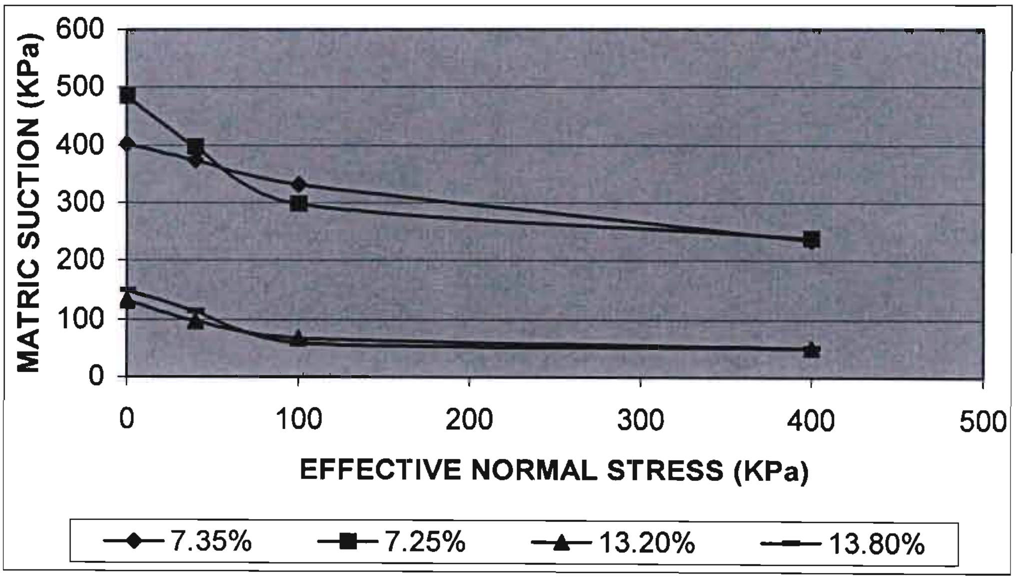


Fig 9.15: Normal stress – matric suction curves of Berea Red Sands compacted to dry density of 1403 kg/m3 and moisture contents of 7.3 % and 13.5 %

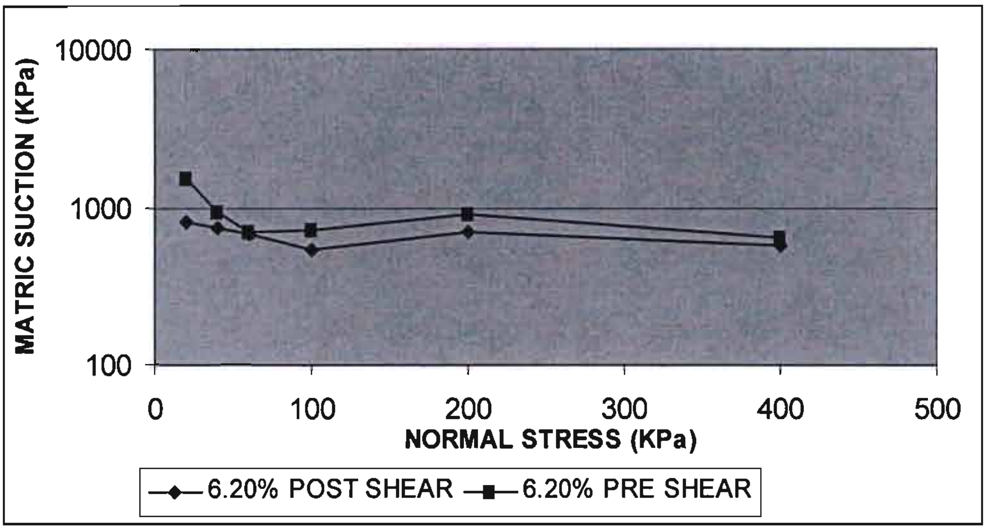


Fig 9.16: Matric suction – normal stress curves of Berea Red Sands at average moisture content of 6.2 % for samples tested in the direct shear box device This preliminary test was conducted to examine whether significant changes in matrix suction are to be expected due to shear induced compression of Berea Soil compacted to low density

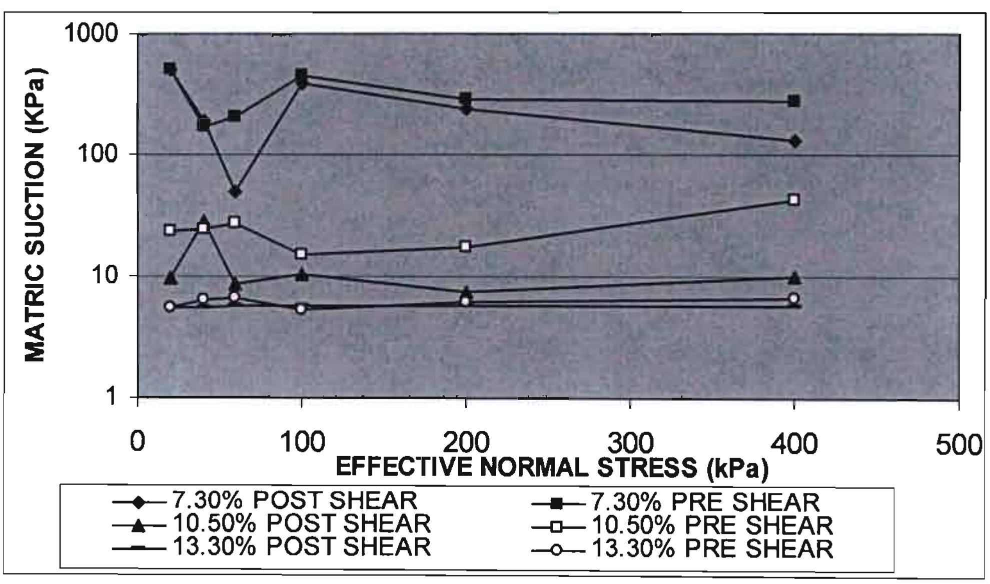


Fig 9.17: Matric suction – effective normal stress curves of Berea Red Sands tested at average moisture contents of 7.2 % and 10.35 % and 13.30 %.in simple shear device under constant normal stress conditions.

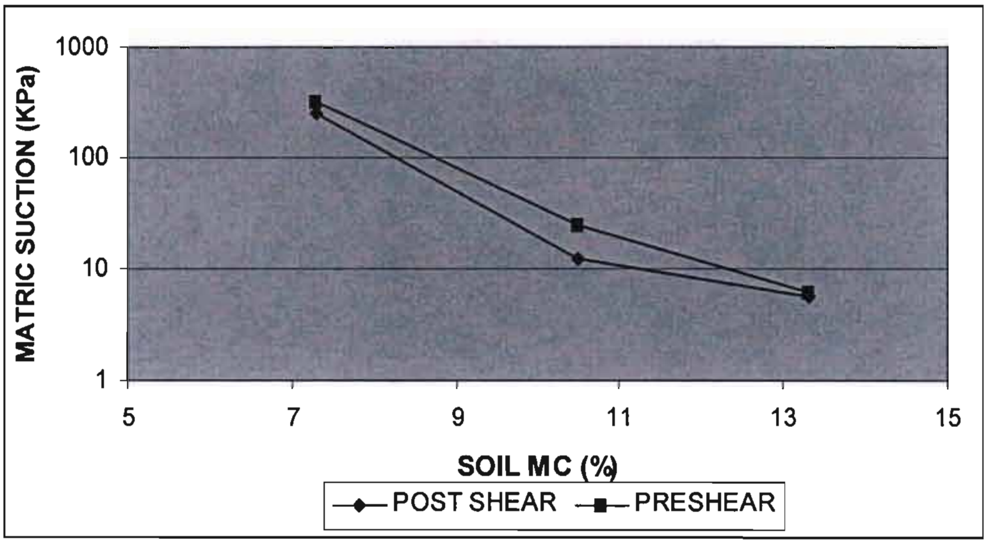


Fig 9.18: Matric suction – moisture content curves of Berea Red Sands samples tested at moisture contents of 7.2% and 10.35% and 13.30% for normal stress 100 kPa– 400 kPa.

CHAPTER TEN

LOAD SETTLEMENT BEHAVIOR OF VERTICALLY LOADED PILES IN BEREA SANDS

10.1 INTRODUCTION

The determination of the ultimate vertical load on a pile is not always sufficient to ensure functional operation of the supported structures. For design of structures especially in sands, residual soils and naturally occurring unstable formations, the load-settlement relation is needed to ensure adequate control of the allowable deformations.

The determination of capacity and load settlement behavior of vertically loaded piles installed in Berea Red Sand formations is presented in this chapter The numerical studies of the behavior of vertically loaded piles in Berea Sands entail an iterative scheme, which synthesizes the load - settlement curve of the pile top from laboratory data of series of simple shear tests, conducted on compacted Berea Red Sands in the newly developed simple shear device. The numerical technique, which is based on the load transfer concept presented by Coyle and Reese (1966), incorporated the effect of decreasing lateral stresses due to shear induced volume reduction of the soil close to the pile surface. The resistance of the soil below the pile tip was also incorporated in the modeling. The decreasing lateral stresses due to shear induced volume reduction of the soil close to the pile surface were determined from normal stiffness tests data. The major steps and details of the iterative scheme are outlined.

10.2 SOIL ZONES ALONGSIDE A VERTICALLY LOADED PILE

The different soil states applicable to a vertically loaded pile initially installed in an isotropic and homogenous sand deposit is shown in Fig 10.1. For the determination of pile load – settlement curves, the surrounding soil is divided into the following three major zones in relation to their response to pile head load;

Zone 1, the boundary layer adjacent to the pile shaft. The shaft capacity of the pile is mobilized in this narrow band of soil, thus the peak and the residual shear stresses are concentrated in this narrow band of soil, which can contract or dilate under the strain imposed by the pile settlement. During pile settlement, simple shear strain is imposed on this narrow band of soil by the pile interface.

Zone 2, the outer soil extending radially away from zone1 up to a limiting distance where the effect of interface shear stress is insignificant. Beyond the limiting radial boundary the geostatic stress state is defined by the effective overburden stress and coefficient of earth pressure at rest. The stress strain response in this zone is purely elastic.

Zone 3, the soil below the plane through the toe of the pile. The stress – strain behavior is elasto – plastic.

10.3 PILE DIMENSION

As the length of a pile increases, the ratio of the pile load to pile head settlement approaches a limiting ultimate value. If piles were to be used go reduce settlement it would be illogical to design piles of such geometry that this limit was reached or even approached (Fleming et al, 1992, Tomlinson 1995). For typical values of pile and soil parameters Fleming et al (1992) and Tomlinson (1995) indicated that reduction in skin friction starts to become significant where the pile slenderness ratio (pile length/diameter) is greater than 5% of the stiffness ratio (Pile modulus/soil modulus) Typical values of stiffness ratios of concrete piles in sands are generally greater than 1000, resulting in minimum slenderness ratio of 50.

Randolph and Wroth (1978) reported that significant shear transfer in piles is to be expected for piles with slenderness ratio within the range of 40 – 80. Beyond 80, excessive slip at working load may cause significant reduction in shear load transfer.

Capacity and load settlement behavior of Reinforced concrete bored piles installed in low density Berea Red Sand formation was evaluated to assess whether compression of the soil in the shear zone will cause significant reduction in pile capacity. The geometry and properties of the pile material and soil properties are given below

REINFORCED CONCRETE PILE GEOMETRY

The geometry is selected to give a slenderness ratio greater than 40

Length (m)	20.0m
Diameter (m)	0.4m/400mm
Shaft Perimeter (m)	1.256m
Base area (m^2)	$0.125m^2$

Young Modulus of the precast concrete bored pile (Fleming et al 1992, Tomlinson 1995)
25×10^3 MN/m^2

BEREA SAND PROPERTIES

The properties of Berea Sands are taken from the test data in chapters 4, 6 and 7,
Average dry density = 1440 kg/m^3 (corresponds to that of samples tested at average moisture content of 7.3% - 8.3%, in the constant normal stress and the constant normal stiffness tests).

Specific gravity	2.7	(from Gs tests in chapter 3)
Dry unit weight	14.12 KN/m^3	(Computed from average dry density)

10.4 END BEARING LOAD –SETTLEMENT RELATIONSHIP.

The settlement of the pile tip was modeled as an impression of a rigid punch on an elastic medium by Randolph and Wroth (1978). The general form of the elastic- perfectly plastic equation is given as;

$$\Delta_b = \frac{P_b(1-\nu)\eta}{4rG} \quad \text{...10.1}$$

Pb = load on pile base

G = representative secant shear modulus

r = pile radius

η = correction factor for the effect of the rigidity of the overburden above the pile base level. η = 0.85 for point bearing piles and 1.0 for point and shaft bearing piles.

The elastic equation can be rewritten as;

$$k_i = \frac{P_b}{\Delta_b} = \frac{4rG}{(1-\nu)\eta} \quad \text{......... ....10.2}$$

Eq 10.1 represents the linear elastic stiffness of the pile tip.

When the value of k_i was used with $\eta = 1$ in the study of pile behavior, Armeleh and Desai (1986) observed that predictions did not provide satisfactory correlation with observed behavior of the tip. They reported that correlation with series of pile load tests in cohesionless soils indicated that tip resistance computed by the above equation is 2.6 times less than the measured field values. Vesic (1981) and Kraft (1981) reported that η be reduced to 0.5 – 0.78 due to depth effects.

To investigate the above issue further, a new pile tip model based on 3 D axisymetric elasto – plastic response was formulated by idealizing the stress state below the pile base as a triaxial stress state in which the pile tip resistance is equal to the deviatoric stress.

Results from laboratory based model pile tests indicated that the behavior of piles subjected to 3-D stresses can be studied by utilizing stress conditions experienced by a soil sample in the triaxial apparatus (Armeleh and Desai 1986). Duncan and Dunlop (1988) observed that analysis of settlement of deep foundations based on plain strain and triaxial parameters are closely correlated with the results of pile load tests than the results of elastic analysis when settlement of deep foundations closely predict.

Under specific form of stress dependence axisymmetric loading, the stress condition at the pile base can be idealized as one in which the soil below the pile base is subject to a uniform horizontal stress related to the effective overburden stress and a coefficient of earth pressure at rest. In addition the soil is subject to a vertical stress component of the pile head load. These stress combinations idealized a triaxial stress condition. However for this particular loading condition, the settlement of the pile due to the vertical stress is significant up to a particular depth below the pile base and is thus dependent on the elastic modulus of this soil thickness. Conventional estimate of this amount of settlement is not sufficient as the settlement is reduced by stress component due to the lateral stresses which can be computed and also affected by the rigidity of the overburden or depth effect which cannot be analytically computed. Factors accounting for overburden effects correlated from results of pile load tests at different depths have been suggested by various authors. The effect of the above factors on computed settlement is examined below.

The general stress state of the soil below the base of a pile can be studied as an axisymmetric elasto – plastic behavior of the form

$$\sigma_1 = \sigma_3 + C\varepsilon_1^n \text{ ....} 10.3$$

σ_1 is the major principal stress in an axisymmetric loading system and in this specific problem is equal to $\frac{p_b}{A}$.

The general differential form of strain is given as

$$d\varepsilon_1 = \frac{d\sigma_1}{E} - \frac{v(d\sigma_2 + d\sigma_3)}{E} = \frac{d\sigma_1}{E} - \frac{2v(d\sigma_3)}{E} \text{ ....} 10.4$$

$$K = \frac{d\sigma_3}{d\sigma_1} \text{ ....} 10.5$$

n = 1 for pile loading within the elastic range.

M is the modulus of the soil below the pile base

$$C = \frac{\sigma_3}{2v}$$

Differentiation of Eq.10.3 with respect to ε_1, and substitution of the value of ε_1 from the same Equation yields

$$\frac{d\sigma_1}{d\varepsilon_1} = M = \frac{E_o}{1 - CK}(\sigma_3) \text{ ....} 10.6$$

$$M = \frac{E_o}{1 - CK}(\sigma_3)$$

$$M = \frac{E_o}{1 - 2vK}$$

Thus the general equation for the tip resistance is given as

$$Pb = \frac{E_o \varepsilon A}{1 - 2vK} \text{ ....} 10.7$$

where A is the area of pile base.

Frank (1975) and Armaleh and Desai (1988) have noted that on the basis of instrumented pile load test on sands, the depth at which the end bearing pressure of piles is still significant i.e. the depth corresponding to 0.1Pb is at 0.5D below pile base for compressible piles.

Thus ε was calculated from w/0.5D; where w is the pile tip settlement

$E_o = 2(1+v)G_t = 9$ MPa. G_t is the value of G_o at the pile tip as G_o varies with depth.

$$G_o = 0.08z + 1.8 \text{ (MPa)..} 10.8$$

G_o was calculated from the stress – strain curves of the constant normal stress test at small imposed simple shear strain of 0.33 %. G_o should have been computed from triaxial test results in keeping with the 3-D axisymetric nature of the problem but at such small strain, the value of G_o determined from stress – strain curves generated from triaxial test results can be assumed equal to the value of G_o determined from simple shear based curves, provided the samples were tested at the same dry density and moisture content (Airey et al, 1985; Jardine et al, 1985; Jewell 1988).

Shaft capacity is mobilized at much smaller displacement of the pile than the base capacity. Fleming et al, (1992) and Smith (1993) defined the ultimate pile capacity as the load that cause settlement of 10% of pile diameter, shaft capacity is the load mobilized at settlement of 0.5% - 2.0% of pile diameter. This implies that for the load settlement analysis of a 400 mm diameter pile, the ultimate shaft capacity may be fully mobilized within 2mm to 8mm tip settlement of the 20m long pile.

The simple shear constant normal stress – strain curves did not indicate peak values of shear stresses, thus assuming the peak shear stress is mobilized at 20 % strain imposed on a 30mm high sample, then 20 % strain corresponds to 6mm pile interface slip or settlement. This value is well within the range recommended by Fleming et al, (1992) and Smith (1993) and although they have reported slip of 12mm – 15mm of bored piles in some sand deposits. however this value is approximate as the displacement at which ultimate shaft capacity is mobilized is the settlement at which the net increase in shear load transfer is zero or insignificant and the value can be predicted by evaluation of the pile load settlement behavior determined by the load transfer method as determination of ultimate capacity can only be by pile load tests.

For a pile with diameter of 0.4m, installed in Berea Red Sand deposits with average submerged unit weight of 9.07 kN/m^3. P_b estimated by the modified elastic method and P_b computed using the elastic method of Randolph and Wroth (1978) are shown in Fig 10.9. Fig 10.9 confirmed the findings of Armeleh and Desai (1988) who indicated that the elastic - perfect plastic model of Randolph and Wroth (1978) is marginally suitable for soils of high stiffness with significant elastic range of stress - strain response but gave unrealistic values for soft soils.

The general equation for the pile base load has been described in terms the bulk modulus i.e. a volumetric response of the soil in keeping with the triaxial stress condition at the pile base. From Fig 10.9 it can be seen that the tip resistance derived from the two model continue to increase with increasing settlement because of the assumption of elastic half space in the formulation of the equation. For settlement up to 5mm the modified values is within the range of 2 to 3 times the values computed by the Randolph's methods and thus satisfies the findings by Armeleh and Desai (1988) that correlation with series of pile load tests in cohesionless soils indicated that tip resistance computed by the above equation is 2.6 times less than the measured field values.

The general equation for the tip resistance has been formulated without incorporating a factor due to the rigidity of the overburden above the base of the pile. However Randolph and Wroth 1988, Luker 1988 and Vesic have suggested a value of one for the depth effect where the pile head load is shared by the base and shaft.

While the pile tip resistance computed from either of the two pile base equations may not have significant effect on mobilized capacity at small values of settlement, choice of tip resistance will have significant effect at large values of settlement. The values published in the proceedings of the symposium on piling along the Natal Coast (SAICE 1984) in which most of the report where based on driven piles gave few relevant data on bored piles of diameter of 360mm – 400mm and lengths between 16.5mm and 21m in predominanatly silty clayey sands which are all significantly above the value of 20kN at settlement of 4 mm– 7 mm

Perhaps the weakness of the general equation for tip load is that the tip load continues to increase infinitely with tip settlement. This is due to the adoption of the value of n = 1 for an assumption of elastic response of tip settlement to tip load. A factor less than one would modify equation 10.7 as

$$Pb = \frac{E_O \varepsilon^n A}{1-2\nu K} \quad \text{..........} 10.7(1)$$

Where n < 1.0

While the above modification may not ensure a limiting value of tip resistance at some significantly large value of settlement, the asymptotic relation implies that the tip resistance tends to a constant value. For this condition, the design or field value of n can then be back calculated from Pile load tests in Berea Sand Formations, an exercise recommended for further studies.

10.5 BOUNDARY LAYER MOVEMENT

The vertical movement of the soil in the shear zone is determined from the stress – strain curves of Berea Red Sands presented in chapter seven. For the analysis of pile behavior in Berea Sands constant normal stress and normal stiffness curves of samples within the moisture content range of 7 2% - 8.1% were used. These curves did not indicate peak values of mobilized shear stresses thus the shear stresses up to 33% strain were evaluated. Representative shear stress – strain graphs for values of normal stresses and initial normal stresses that are close to the range of radial stress and initial radial stresses that the entire pile length was subjected to for the constant normal stress and normal stiffness conditions are presented in Figs 10.2, 10.3, 10.4 and 10.5. Intermediate values of normal and shear stresses were determined by interpolation.

Stress and strain data extrapolated from the constant normal stress and normal stiffness stress – strain curves shown in Figs 10.2, 10.3, 10.4 and 10.5 were used for the pile load settlement analysis for slip of up to 33% simple shear strain or 9.3 mm displacement of a 30mm wide shear zone.

10.6 STIFFNESS OF THE ELASTIC ZONE

Before pile settlement, the initial stiffness of the plastic zone is the same as the stiffness of the undeformed elastic zone because the plastic zone emerged due to stress concentrated resulting from mobilized shear stress at pile interface due to pile settlement. The elastic shear modulus G was determined at very low strain of 0.33% of the compacted Berea Red Sands as the stress – strain behavior is assumed to be elastic at this small value of strain. Jardine et al, (1985) have shown that this can be done in the laboratory if accurate measurement is made on the specimen. They determined the shear modulus at 0.1% strain and showed that the value agreed well with the values from geophysical methods and careful back analysis of plate bearing and pile tests.

The elastic shear modulus G is estimated from the constant normal stress curves of Fig 8 at 0.33% strain. Fig 10.2 indicated an increase in modulus with applied normal stress. It is assumed that the applied normal stress on the sample is the same as the soil overburden stress over the depth of pile embedment, thus the change in soil modulus with depth is;

$G_o = 0.08z + 1.8$ (MPa) 10.9

where z is the depth of soil.

The value of shear modulus G at half the pile depth is 2.2 MPa and the associated Young Modulus E is given as 2 G (1+v) and with poisson ratio v = 0.3 for drained conditions, E = 6.76 MPa at mid depth of pile and E = 8.84 MPa at pile tip.

Spings of different elastic properties were used to study the elastic behavior of the outer boundary close to the pile surface. The elastic properties of the springs used were the stiffness constant ie the ratio of the applied load to the compression of the spring, and the elastic modulus ie the applied stress per unit strain of the spring. The stiffness constant and modulus of the spring selected are 140 N/mm, 8.4 MPa; 250 N/mm, 15MPa; and 1660 N/mm, 50MPa.

The elastic modulus of the springs selected are close to 6.76 MPa, the value determined from laboratory tests, springs of higher moduli were also selected to facilitate the study of the effect of the magnitude of normal stiffness on the stress – strain behavior of sands in simple shear.

10.7 THE LIMITING LATERAL RADIUS.

Randolph and Worth (1978) modeled the movement of the soil alongside the pile as the elastic deformation of concentric cylinders with a hyperbolic relationship between shear stress and strain The shear stress τ_o at the pile surface r_o is related to the shear stress τ_r at any radius r by the expression

$\tau_r = \tau_0 r_0 / r$ 10.10

The deformation Δ_r associated with the shear stress τ_r at a short interval of radius ($r - r_o$) or δr:

$\Delta_r / \delta r = \gamma = \tau_r / G$ 10.11

where:

γ = shear strain

τ_r = shear stress at r

G = shear modulus applicable to the soil region.

$$\therefore \frac{\Delta_r}{\delta r} = \frac{\tau_0 r_0}{Gr} \quad \text{..........} 10.12$$

From equilibrium, $$\therefore \Delta_r = \frac{\delta r \tau_0 r_0}{Gr} \quad \text{..........} 10.13$$

The deformation Δr is significant at values of r close to r_o, and decreases to negligible displacement at a value of r equal to r_m.

The method of determining r_m is to start with a low value of r - r_o and progressively increase it, calculating Δr for each value of $r - r_o$ until the change in Δr becomes negligibly small for an increment of r - r_o.

The initial value of r is a small value greater than ro. For the determination of r_m, r = 1.0 mm was used

r_o is the pile radius = 0.2 m

$\tau_{o\ is}$ is the shear stress at the plastic zone. This is equal to the shear stress corresponding to 20% simple shear strain for the corresponding normal stress.

The vertical deformation profile of soil close the pile shaft due to pile settlement is shown in Fig 10.6 .The curve clearly indicates that at 10m from the pile surface, the vertical movement of the soil is insignificant, thus r_m is taken as 10m or 20 radii from the pile surface. This value is based on average shear modulus along pile depth. Fig 10.7 demonstrates the effect of increasing the soil stiffness on the vertical movement of the soil. The curves show that increasing the soil stiffness decreases the vertical movement.

Randolph and Wroth (1978) suggested that the value of r_m can be approximated from the expression;

$$r_m = 2.5\ L\ (1 - \nu) \quad \text{..........} 10.13$$

Thus for a pile length of 20m and Poisson ratio of 0.3, r_m is 35 radii from the pile surface. This value is greater than 20 radii determined from the elastic modeling of concentric rings. The difference is due the effect of shear modulus on Δr. The effect of shear modulus on Δr is not accounted for by Randolph and Wroth (1978).

10.8 THE HORIZONTAL STRESSES ON PILE SHAFT

The effective horizontal stress σ'_h normal to the pile shaft is computed from the average effective vertical stress using the expression;

$$\sigma'_h = K\,\sigma'_v$$

The appropriate value of K will depend on the in-situ earth pressure coefficient, the method of pile installation, and the initial density of the sand. Fleming et al (1992) examined the results of series pile tests results on sand deposits and noted a similar trend for the skin friction as been observed as for the end bearing pressure i.e. the rate of increase in skin friction with depth gradually reduces and there is a tendency towards some limiting value. They concluded that it is reasonable to expect the value of K to vary in a similar fashion to N_q and proposed a relationship between K and N_q of the form;

$K \sim N_q/50.$

N_q is determined from the chart by Meyerhoff (1976).

This equation implies that the value of K may vary with depth depending on the soil stratification. The drained frictional angle ϕ' for Berea Red Sands reported by Brink from series of triaxial tests is 31 degrees. Assuming this value to be the average value over the depth of pile embedment then $K = 0.8$.

Tomlinson (1995) suggested that for bored piles, or driven cast-in-situ piles where the casing is removed, loosening of the surrounding soil may occur, followed by some increase in stress level as the concrete is placed. The appropriate value of K in the above equation will depend on the details of the construction method. For conventional bored piles Tomlinson noted that it is common practice to adopt values of 0.9 (for sandy soils) down to 0.6 (for silts and silty sands). In general, the roughness of the hole will ensure that the full soil friction is mobilized at the pile edge and generally since failure occurs in the plastic zone close to pile surface, the drained frictional angle ϕ' rather than the pile soil interface angle is used for computation of ultimate shaft capacity. For driven cast-in-situ piles, while the value of K may be 1.2 or higher outside the casing, some reduction in normal stress may occur during extraction of the casing.

For assumed settlements of the pile tip, the associated plastic zone compression results in expansion of the outer zone and subsequent reduction in horizontal stress $\Delta\sigma'_h$. The amount of reduction is dependent on the stiffness of the elastic outer region.

The reduction in horizontal stress $\Delta\sigma'_h$ is computed as follows,

$$\Delta\sigma'_h = \frac{[PZM][EZSM]}{[EZM]} \quad \text{..1014}$$

PZC = Plastic zone compression

EZSM = Elastic zone shear modulus

EZW = Elastic zone width

In the above expression, the plastic zone shear strain is equal to the vertical strain of laboratory samples in the constant normal stiffness tests. The width of the plastic zone is assumed equal to the initial height of soil samples tested in the new simple shear device.

10.9 AN ITERATIVE SCHEME FOR PILE LOAD - SETTLEMENT BEHAVIOR

The load transfer method proposed by Coyle and Reese (1966) for clays and Coyle and Sulaiman (1967) for sands was modified to determine the load at the top of the pile and the settlement at the top of a bored pile installed in Berea Red Soil the mechanical properties of this soil. Details of the method are as follows;

Assume a small pile tip movement. It is convenient to assume an initial tip movement of 0.1mm (Coyle and Sulaiman 1967)

Compute the tip load Pb corresponding to the movement. The tip load Pb was computed from the modified elastic formula for pile tip load presented in 7.

Divide the pile into several segments. The bored pile installed in Berea sands was divided into five segments.

The estimation of the mid point movement of the bottom segment. The movement of the middle point of this segment is the difference between the tip movement and the elastic compression of the lower half of segment due to tip load. Here the tip load is assumed to be acting downwards from the mid point of the segment.

Assuming a linear variation of the load distribution for small segments, then the slip of the last segment is the tip movement less the segment elastic compression. It is the pile settlement relative to the adjacent soil i.e. the pile – soil movement Δ_{slip}

The shear force associated with the segmental slip is the product of the shear stress associated with the slip and the surface area of the segment. This is the load transferred to the surrounding soil. The relevant shear stress for a given slip can be determined from the stress strain curves presented in chapter 9

The load at the top of the bottom segment is determined as the sum of the load transferred to the surrounding soil and the tip load. While the load at the mid point of the segment is the average of the sum of the load at the tip and the load at the top of the segment. By applying the load at the the top of the segment, a new mid point movement is computed and a new slip is thus determined and compared with the initial slip.

If the above two slips do not agree within a specified tolerance, the above steps are repeated and a new midpoint movement is computed, the iteration is continued until convergence is achieved

With convergence achieved, the movement of the top of the bottom segment, now becomes the tip movement of the segment above the bottom segment. Thus working up the pile length, a value of the pile head load and the pile head movement is determined

The load transferred to the surrounding soil by the movement of the segment was computed by dividing the difference between the load at the bottom of the element and the load at the top by the element surface or shaft area. Thus the equilibrium of each element during pile loading was satisfied before the analysis of the next element. By the same process, the forces and movement at each segment are computed, working up the pile.

The major steps are summarized below;

$Fs = Fs_{max}(\Delta_{slip}/\Delta max)$ 10.15

Δmax is the maximum assumed pile tip settlement at which Fs_{max} is developed.

Δ_{slip} is the assumed tip settlement at which Fs is developed.

$Fs_{max} = \tau_s \pi d \, \Delta max$ 10.16

πd = perimeter of the pile element

d = pile diameter

τ_s = interface shear stress

The pile settlement relative to the adjacent soil i.e. the pile – soil movement Δ_{slip} is given as

$\Delta_{slip} = \Delta_t - h_e$ 10.17

Δ_t = assumed settlement of the pile tip.

h_e = compression of element at mid height.

$h_e = QL_e/2AE$

Q is the computed tip load due to assumed settlement of the pile tip.

L_e is the length of the element.

E is the Young modulus of the pile material.

A is the cross sectional area of the pile.

Based on the value of Fs + Q a value of Δ_{slip} is computed, and the iteration is repeated until two subsequent calculation of pile element compression is sufficiently close i.e. convergence occurs and pile element equilibrium of forces between the tip and the top of that element is satisfied.

For different assumed tip movements, different values of load and settlement of the pile top are determined and thus load settlement curves can be plotted.

10.10 EFFECT OF SHEAR ZONE COMPRESSION ON PILE CAPACITY

The effects of compression of the shear zone were determined from series of pile load settlement analyses carried out in the following stages

A; PILE LOAD SETTLEMENT ANALYSIS EXCLUDING SHEAR INDUCED COMPRESSION

The load – settlement behavior of piles subjected to constant normal stress condition was first evaluated. The data for this analysis were obtained from constant normal stress strain curves of compacted Berea Red Sands in simple shear shown in Table 10.1 and Fig10.2. The shear stresses corresponding to 0.1% 1.0% 3% 6% and 9% were used. The vertical strain associated with the above stresses was here ignored. This implies that the effect of the compression of the soil in the shear zone close to the pile surface on load – settlement behavior was ignored.

B; PILE LOAD SETTLEMENT ANALYSIS INCLUDING SHEAR INDUCED COMPRESSION

The load – settlement behavior of piles subjected to constant normal stress condition was first evaluated. The data for this analysis were obtained from constant normal stress strain curves of compacted Berea Red Sands in simple shear shown in Tables 10.2 and 10.3 and Fig 10.2. The shear stresses corresponding to 0.1% 1.0% 3% 6% and 9% were used. The vertical strain associated with the above stresses were converted to vertical displacements that were used to determined the reduced values of normal stresses and shear stresses, assuming the vertical displacement of the soil sample in the simple shear device is equal to the compression of the shear zone. The modulus of the adjacent soil was determined from the stress - strain curves at 0.33% strain. The effective lateral dimension of the soil adjacent to the pile is r_m.

C; PILE LOAD SETTLEMENT ANALYSIS BASED ON NORMAL STIFFNESS TEST DATA

The load – settlement behavior of piles subjected to different normal stiffness conditions was evaluated. The data for these analyses were obtained from constant normal stiffness stress - strain curves of compacted Berea Red Sands in simple shear shown in Table 10.4 -10.6.and Fig 10.3 – 10.5. The normal stresses and shear stresses were used in pile analyses assuming that the normal stiffnesses used in the simple shear tests are equal to that of the elastic zone ($r_m - r_0$) close to the pile surface. The normal stiffnesses are thus 140 N/mm, 250 N/mm and 1660 N/mm.

Shear stresses at the pile interface were interpolated from constant normal stress and normal stiffness simple shear stress – strain curves corresponding to values of normal stresses within the range of 40 kPa – 200 kPa. The values were presented in tables 10.1 – 10.6 for simple shear strain up to 30% corresponding to pile interface slip of 9.0mm

10.11 PILE LOAD SETTLEMENT BEHAVIOR IN BEREA RED SOILS

The distribution of the pile head load with depth and the load settlement curves for assumed tip settlement of 3mm, 6mm and 9mm assuming constant horizontal stress normal to pile shaft are presented in Fig 10.10 – Fig 10.16, while the combined load settlement curves were present in Fig 10.16.

Similar to the e – log P curves which are used for the modeling and characterization of one dimensional settlement behavior of soils and which also facilitates clear delineation of bilinear or non linear response and location of preconsolidation stress for over consolidated soils,. logarithmic scale was adopted for the derived load settlements curves because two digit changes in magnitude of P is often required to produce small changes in settlement, in addition it would facilitate easy identification of non linear and linear ranges of load – settlement behavior. A more economical normalization approach is to plot the ratio of pile head load to tip load versus the tip settlement. This method of normalization allows for easy determination of the influence of assumed tip load on the load settlement behavior.

Fig 10.10 – Fig 10.16 indicate that increasing the tip settlement by 100% from 3mm to 6mm resulted only in 25% increase in pile capacity and subsequent 100% increase in settlement to 9mm also resulted in 30% increase in pile capacity. Fig 10.10 – Fig 10.16 also revealed that over 90% of the applied load or the pile head load is carried by the shaft. The tip settlement – pile head curves did not indicate clearly defined values of pile capacity. The tip settlement – pile head curves obtained here are suited for design conditions where capacity is based on an allowable value of tip settlement. This is general method of determining pile capacity in sands.

The effect of the compression of the plastic zone alongside the pile shaft on capacity and load settlement behavior were shown in 10.17 and 10.18 .These figures indicate that average shear induced strain of 10 % in the plastic zone resulted in a decrease of below 5% of mobilized capacity for tip settlement of 9mm, and indicated no difference in load settlement curve. The curves shown in Fig 10.18 are based on average soil modulus of 6.76 MPa and 8.4 MPa. The curves show that there is no significant difference in pile load settlement behavior when the elastic modulus is increase from 6.76 MPa and 8.4 MPa, for the same magnitude of shear compression of the plastic zone. The general conclusion here is that if the soil adjacent to the pile is soft compression of the soil in the shear zone will result in insignificant change in pile capacity.

The distribution of the pile head load with depth and the load settlement curves for assumed tip settlement of 9mm assuming the pile shaft subjected to normal stiffness of the outer elastic soil region were presented in Fig 10.19 – Fig 10.24, while the combined load settlement curves were present in Fig 10.25.

Fig 10.19 – Fig 10.24 revealed that mobilized shaft capacity decreases with increase in normal stiffness and up to 50 % decrease may be expected for elastic modulus of 50 MN/m^2 .The curves also revealed that the horizontal space between the pile head load and the shaft load increases with increase in stiffness. This implies that as the outer elastic boundary becomes stiffer relative to the stiffness of the soil in the shear zone, more of the applied vertical loads are carried by the pile tip.

An important observation from normal stiffness laboratory tests results presented in chapter seven is that the higher the normal stiffness, the closer the stress - strain response approaches constant volume behavior. This implies that only small changes in thickness of the shear zone is required to cause significant reduction in horizontal stress and shaft capacity provided the outer soil is very stiff. This is confirmed by the pile analysis results shown in Fig 10.19 – Fig 10.24

The curves of Fig 10.12, 10.17, 10.18, 10.19 and 10.20 revealed that the elastic modulus of the outer elastic soil zone estimated from the simple shear stress strain curve is close to that of the elastic spring of 8.4 MN/m^2 as there is no significant difference between corresponding set of curves.

The important conclusion drawn from pile analysis presented in this chapter is that reduction in shaft capacity may not be significant if the elastic modulus of the outer soil is low irrespective of the magnitude of the compression of the plastic zone, but significant reduction in horizontal stress and shaft capacity would be expected if the elastic modulus of the outer soil is high. Thus significant reduction in pile capacity would be expected where significant percentage of the applied load is carried by the shaft i.e. in friction piles. If the proportion of total load carried by the shaft is not significant, significant reduction in shaft capacity would not translate to significant reduction in pile capacity.

The capacity of friction piles in Berea Red Sands is dependent on both the compression of the soil close to the pile surface as well as on the stiffness of the outer soil. The pile load - settlement curves and the pile load - depth curves clearly revealed that if the outer soil is very stiff, very small changes in the thickness of the plastic zone result in significant decrease in horizontal stress, shaft capacity and pile capacity.

TABLE OF CONSTANT NORMAL STRESS AND NORMAL STIFFNESS SIMPLE SHEAR TESTS ON COMPACTED SAMPLES OF BEREA RED SANDS

TABLE 10.1

Shear stresses from Constant Normal Stress Condition

pile depth	vertical stress	horizontal stress	horizontal displacement of sample in the shear box				
			0.1mm	1.0mm	3.0mm	6.0mm	9mm
			Constant normal stress simple shear stress (kPa)				
(m)	kPa	kPa	2	3.4	6.7	7.4	8
6	54.42	43.536	6	10	20.2	22.2	25
10	90.7	72.56	7.5	12.5	22.8	30	36
14	126.98	101.584	9	15	25.2	37	48
18	163.26	130.608	11	18.6	37.5	50	68

TABLE 10.2

Vertical compressions from Constant Normal Stress Condition

pile depth	vertical stress	Horizontal stress	horizontal displacement of sample in the shear box				
			0.1mm	1.0mm	3.0mm	6.0mm	9mm
			Vertical compression of the sample in the shear box				
2	18.14	14.512	0.29	0.78	1.29	1.89	2.34
6	54.42	43.536	0.285	0.57	1.14	1.71	2.28
10	90.7	72.56	0.285	0.35	0.92	1.56	2.06
14	126.98	101.584	0.285	0.142	0.7125	1.425	1.85
18	163.26	130.608	0.268	0.1	0.54	1.31	1.68

TABLE 10.3

Reduced Shear Stress assuming vertical compression in Table 10.2 equal to lateral compression of the shear zone close to pile surface.

pile depth	vertical stress	horizontal stress	horizontal displacement of sample in the shear box				
(m)	kPa	kPa	0.1mm	1.0mm	3.0mm	6.0mm	9mm
			Reduced shear stress				
2	18.14	13.522	1.864	3.168	6.243	6.895	7.454
6	54.42	42.444	5.850	9.749	19.693	21.643	24.373
10	90.7	71.345	7.374	12.291	22.418	29.498	35.397
14	126.98	100.258	8.883	14.804	24.871	36.517	47.373

18	163.26	129.24	10.885	18.405	37.107	49.476	67.288

TABLE 10.4

Normal stiffness test data for 140 N/mm

pile depth	vertical stress	horizontal stress	horizontal displacement of sample in the shear box				
(m)	KPa	KPa	0.1mm	1.0mm	3.0mm	6.0mm	9mm
			Shear stresses (kPa) for normal stiffness of 140 N/mm				
2	18.14	14.512	2	2.7	5	6	6
6	54.42	43.536	6	8	15	18	18
10	90.7	72.56	7	10.5	19.5	24.5	30
14	126.98	101.584	8	13	24	35	41
18	163.26	130.608	10	17	33	48	54

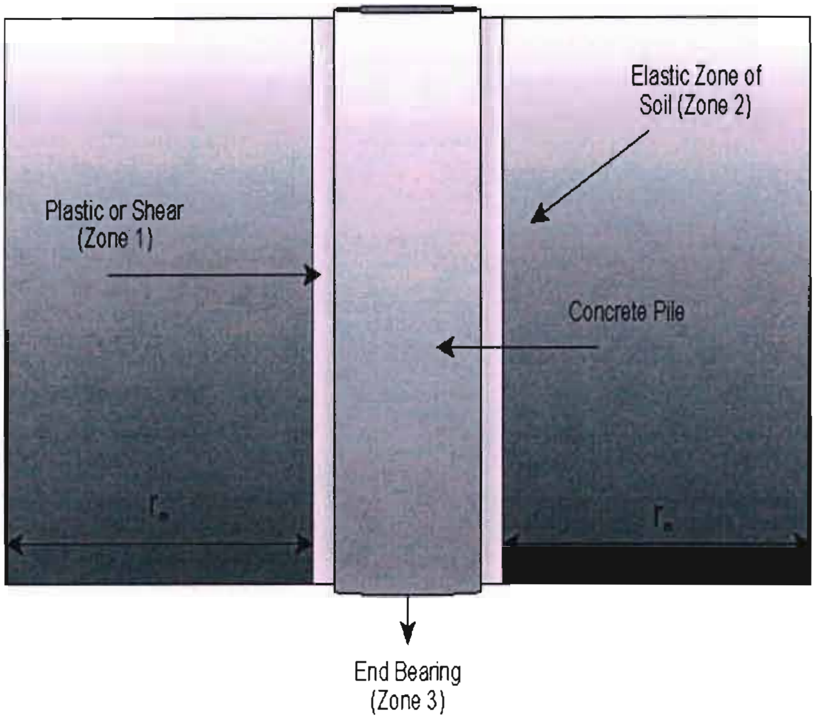
TABLE 10.5

Normal stiffness test data for 250 N/mm

Pile Depth	Vertical Stress	Horizontal stress	horizontal displacement of sample in the shear box				
(m)	(KPa)	(KPa)	0.1mm	1.0mm	3.0mm	6.0mm	9mm
			Shear stresses (kPa) for normal stiffness of 250 N/mm				
2	18.14	14.512	1.7	2.7	5	5.6	6
6	54.42	43.536	5	8	15	17	18
10	90.7	72.56	6	10	18	23.5	27
14	126.98	101.584	7	12	21	30	36
18	163.26	130.608	8.3	15	26.5	38	43

TABLE 10.6
Normal stiffness test data for 1660 N/mm

Pile Depth	Vertical Stress	Horizontal stress	horizontal displacement of sample in the shear box				
(m)	(KPa)	(KPa)	0.1mm	1.0mm	3.0mm	6.0mm	9mm
			Shear stresses (kPa) for normal stiffness of 1660 N/mm				
2	18.14	14.512	1.4	2	2.3	3	4
6	54.42	43.536	4	6	7	9	12
10	90.7	72.56	5	9	13	15.5	20
14	126.98	101.584	6	12	20	23	28



18	163.26	130.608	6.7	15	26	30	35

FIG 10.1 Schematic illustration of the zones of soil surrounding a vertically loaded pile and the limiting lateral radius r_m

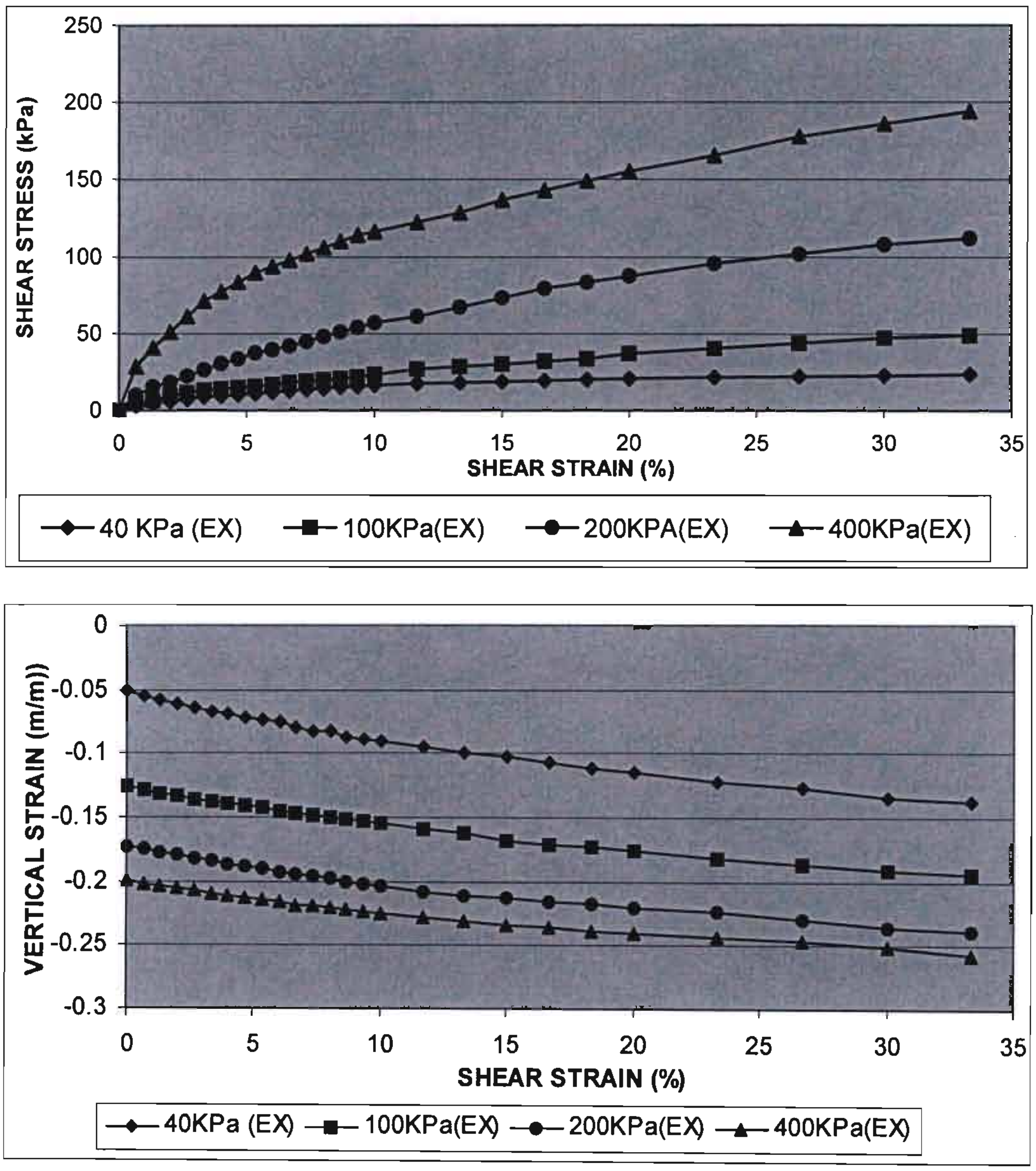
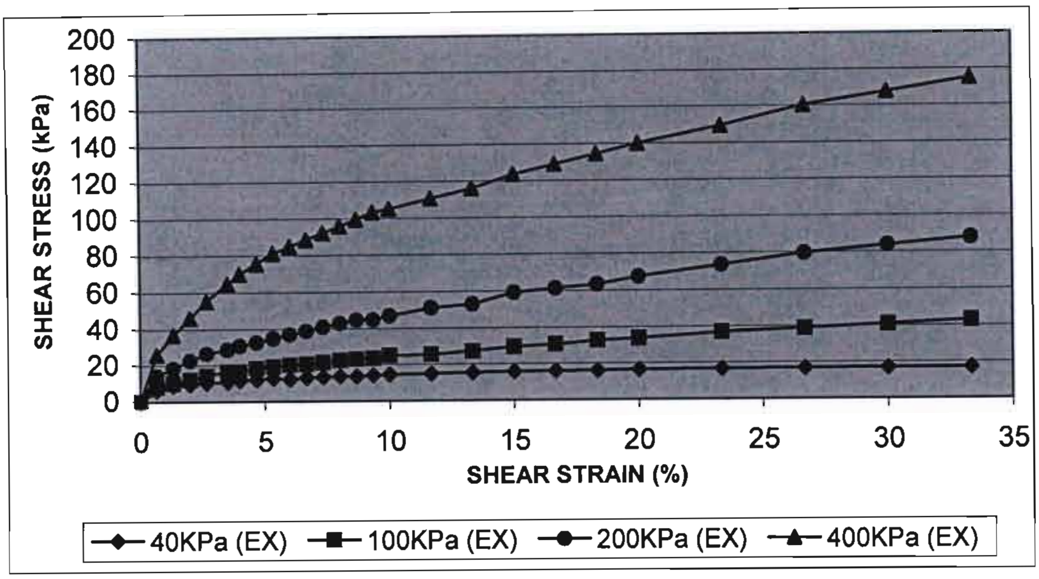


FIG 10.2: Experimental (EX) constant normal stress simple shear curves of compacted Berea Sands. Constant normal stress tests; The experimental results of constant normal stress tests conducted on 30 mm thick samples compacted into the simple shear box at 1411 kg/m^3 - 1421 kg/m^3 dry density and moisture content of 7.07 %-8.25%



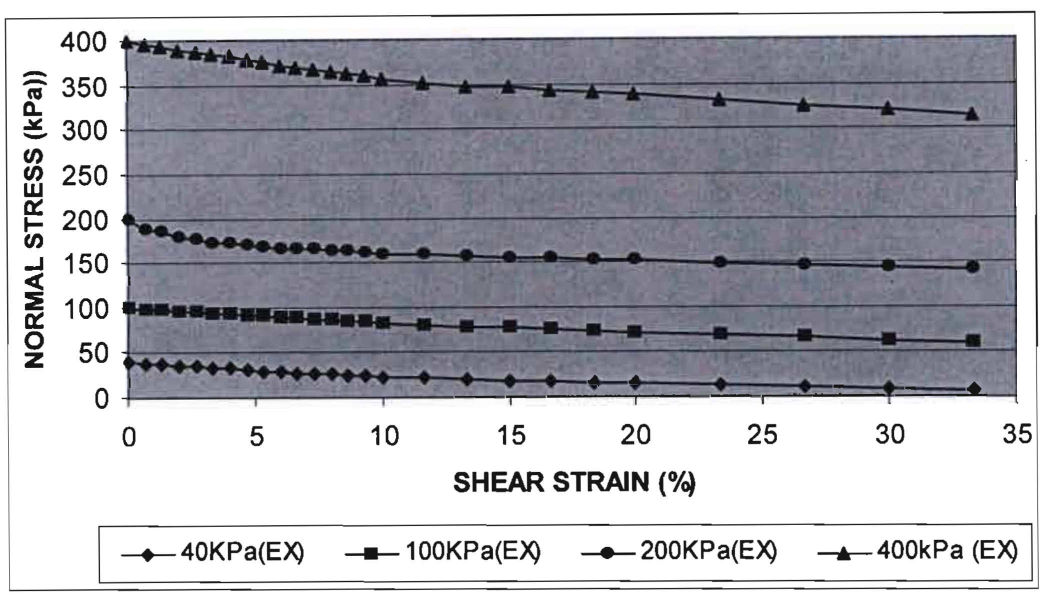
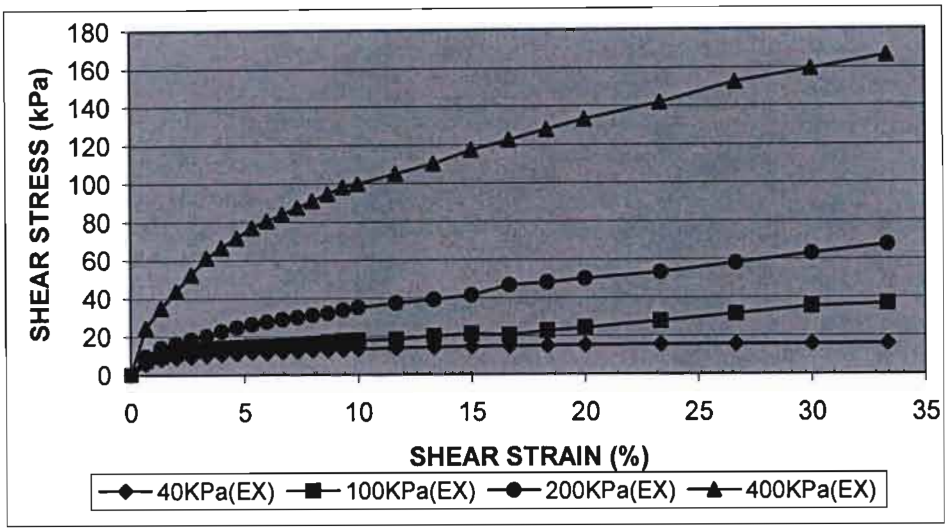


FIG 10.3: The experimental curve of constant normal stiffness tests conducted on samples compacted into the simple shear box to 30 mm thickness, 1411 kg/m^3 – 1433 kg/ m^3 dry density and moisture content of 15.52% - 16.27 % and tested at moisture content of 7.55% - 8.67% with springs of normal stiffness of 8.4 MN/m^2 (140 N/mm).



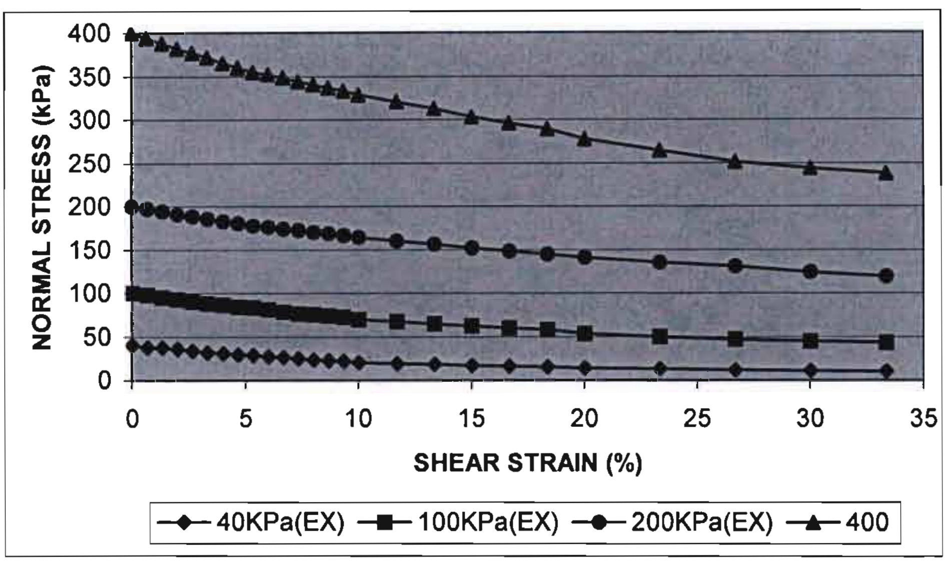
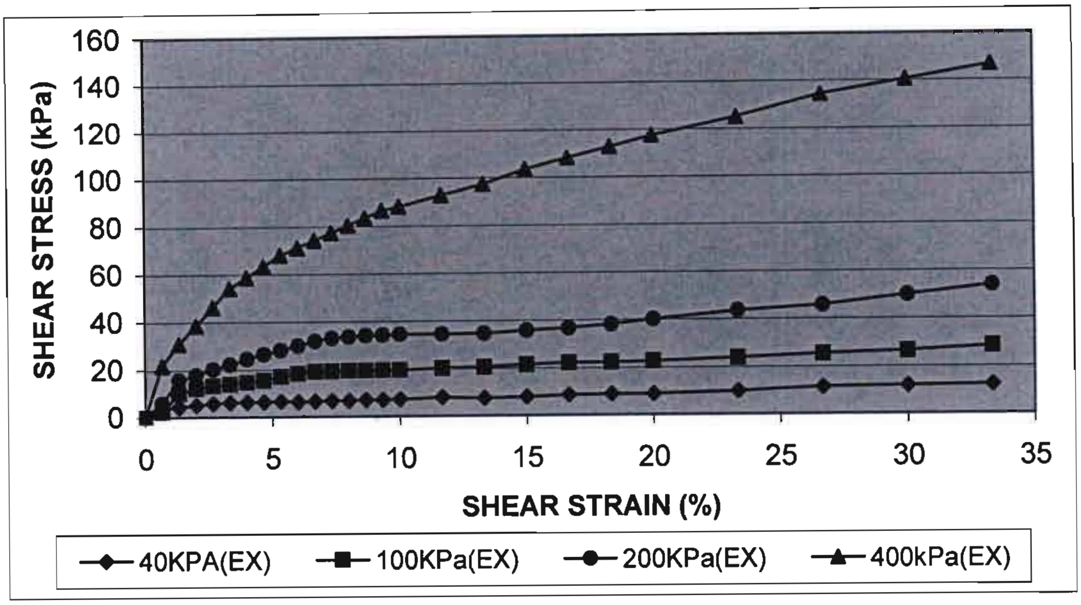


FIG 10.4: The experimental results of constant normal stiffness tests conducted on samples compacted into the simple shear box to 30 mm thickness, 1405 kg/m^3 – 1466 kg/m^3 dry density and moisture content of 15.52% - 16.27 % and tested at moisture content of 7.51% - 8.65% with springs of normal stiffness constant of 15MN/m^2 (250 N/mm)



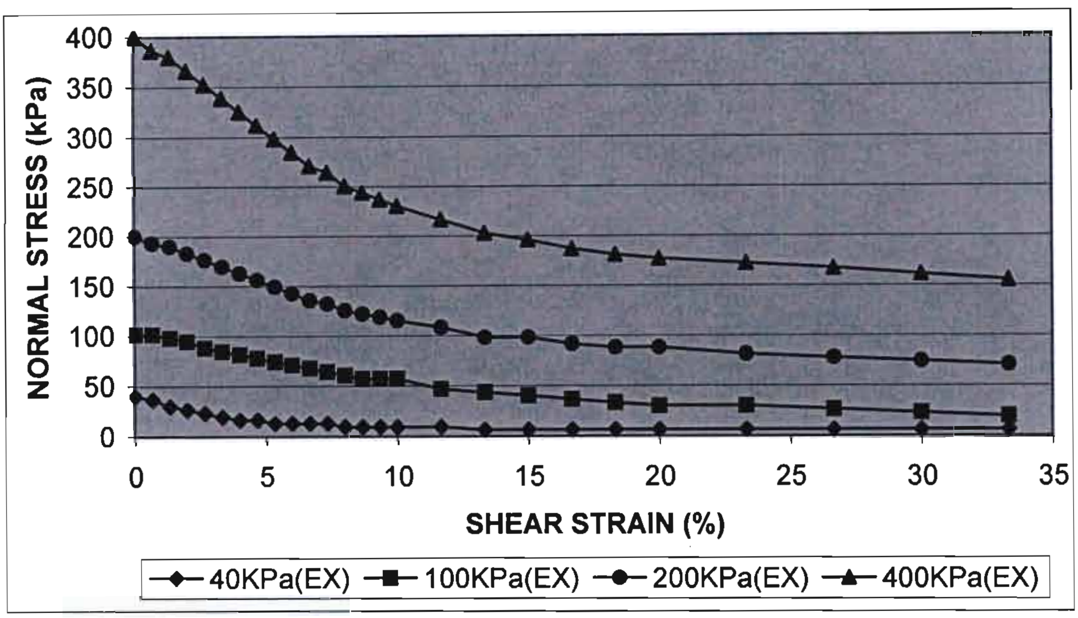


FIG 10.5: The experimental curve of constant normal stiffness tests conducted on samples compacted into the simple shear box to 30 mm thickness, 1426 kg/m^3 – 1475 kg/ m^3 dry density and moisture content of 15.52% - 16.27 % and tested at moisture content of 7.51% - 8.65% with springs of normal stiffness of 50MN/m^2 (1660 N/mm)

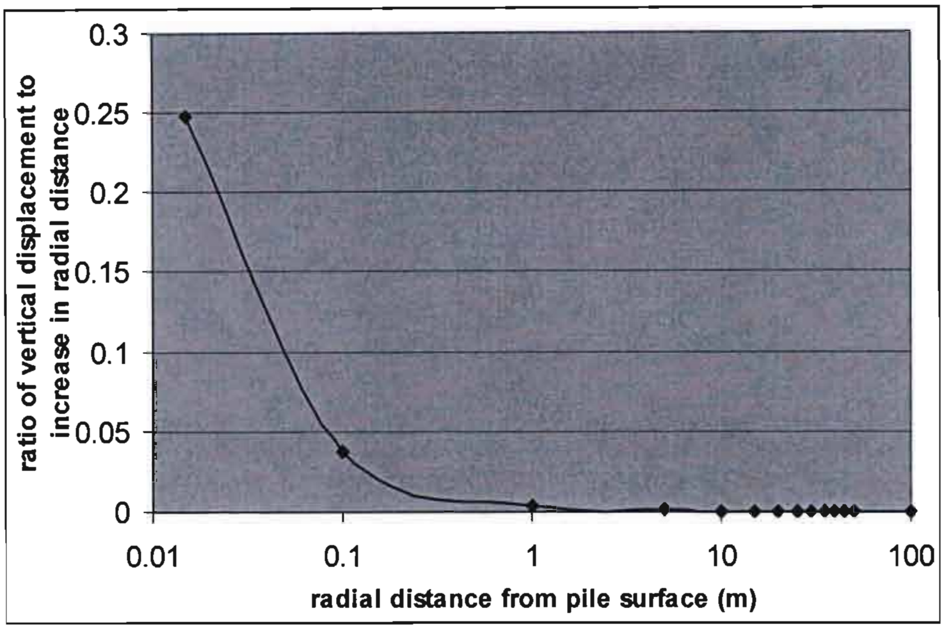


FIG. 10.6: Curve of vertical movement of soil alongside a vertically loaded pile. r_m the radial distance beyond which the vertical movement of soil is zero is at 10m from the pile surface. The average young modulus of the elastic region is 6.86 MPa

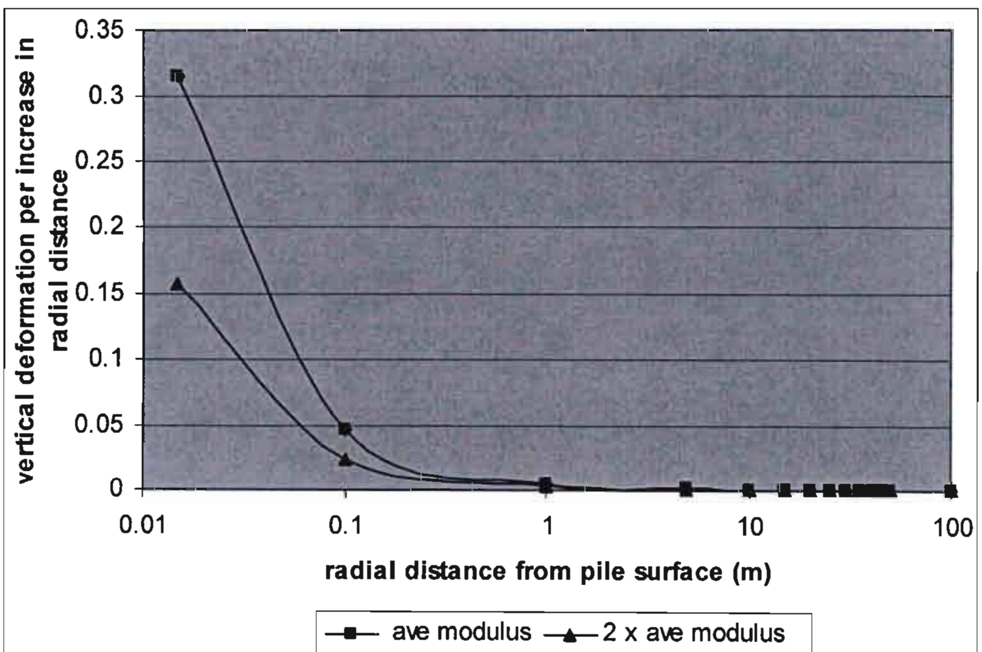


FIG 10.7 Curves of vertical movement of soil alongside a vertically loaded pile, indicating the effect of soil modulus on vertical movement.

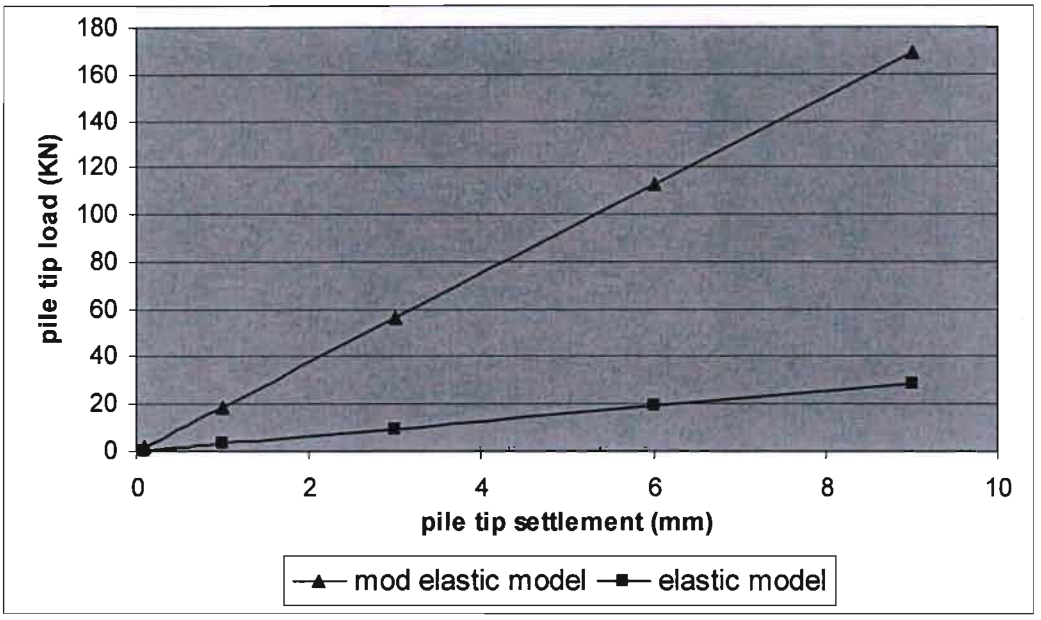


FIG 10.9 Pile end bearing resistance due to pile tip settlement. The elastic model is the model proposed by Randolph and Wroth (1988). The model formulated and applied in the t – z method of pile load settlement analysis is referred to as the modified elastic model

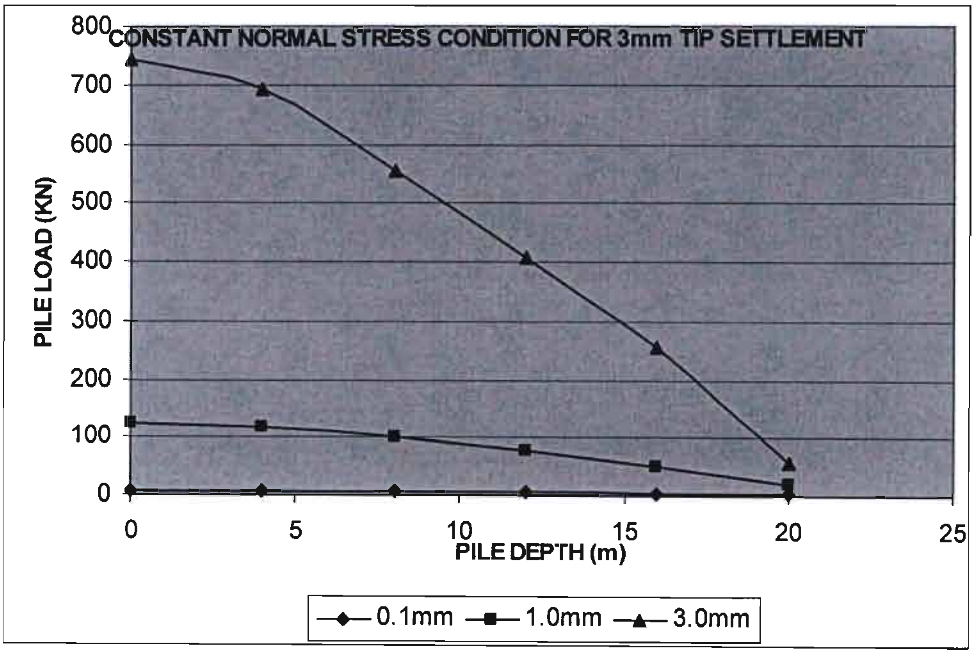


FIG 10.10 Curves of pile load at different pile depths for 3mm tip settlement of pile, where pile shaft is subjected to constant normal stress

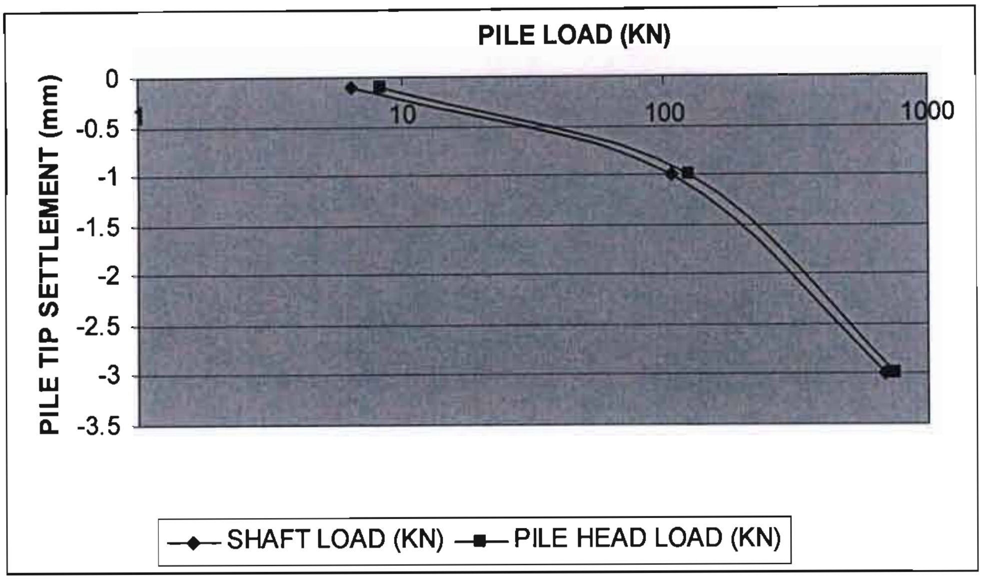


FIG.10.11: Load settlement curve of piles for 3mm tip settlement of pile, where pile shaft is subjected to constant normal

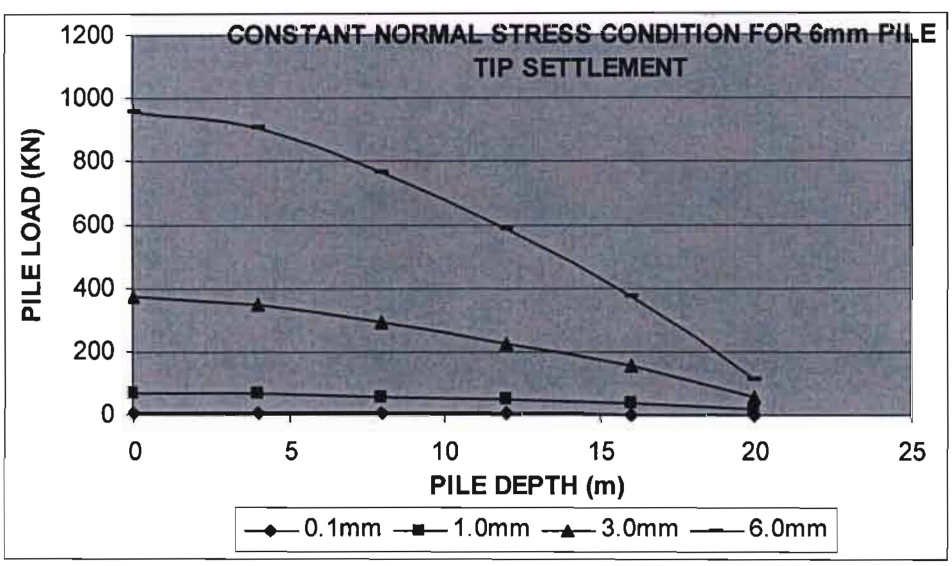


FIG 10.12 Curves of pile load at different pile depths for 6mm tip settlement of pile, where pile shaft is subjected to constant normal stress.

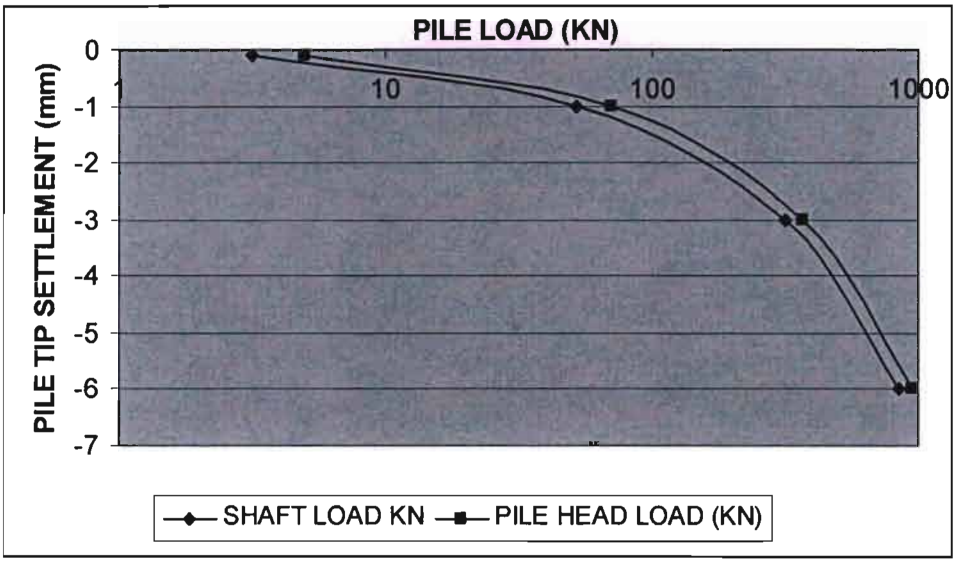


FIG 10.13 load settlement curve of piles for 6mm tip settlement of pile, where pile shaft is subjected to constant normal stress.

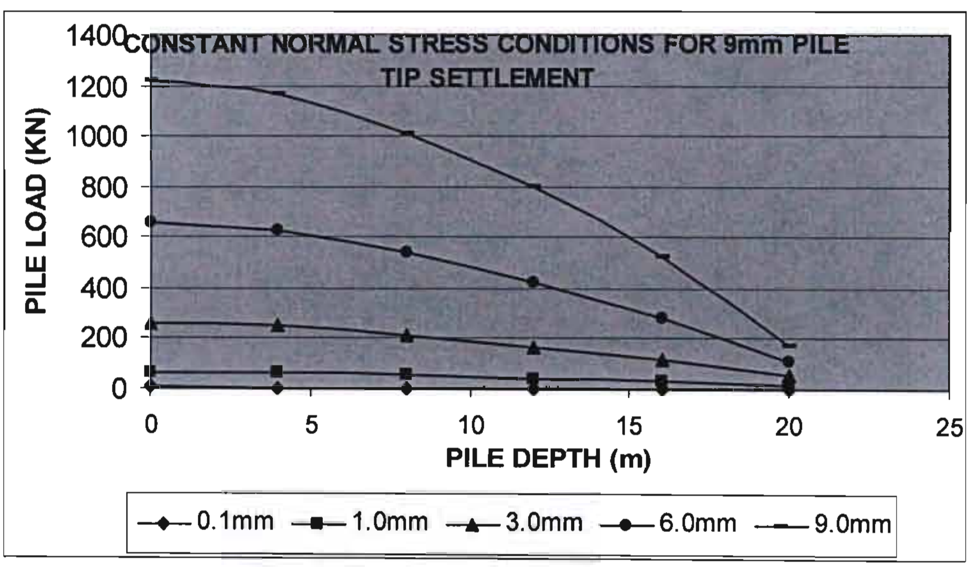


FIG 10.14 Curves of pile load at different pile depths for 9mm tip settlement of pile, where pile shaft is subjected to constant normal stress

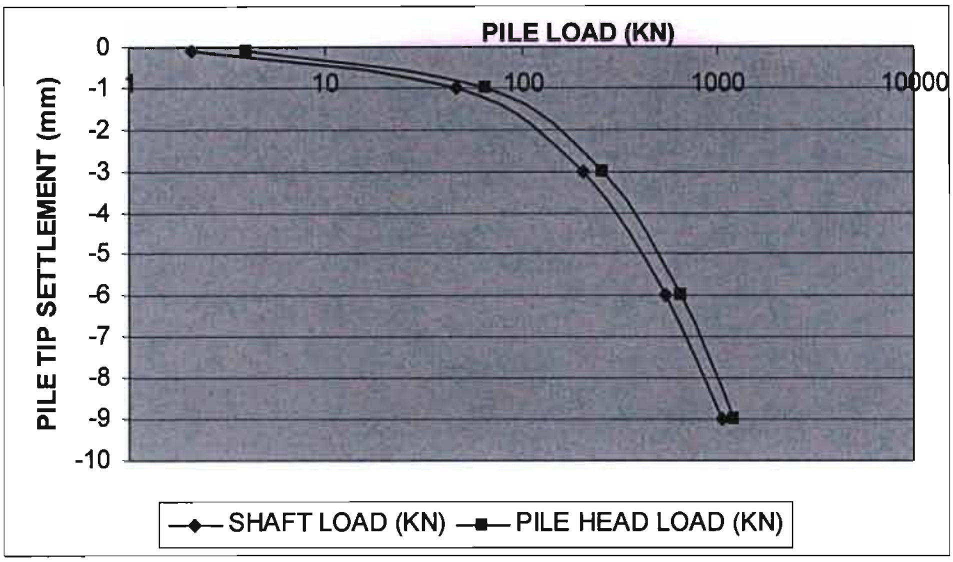


FIG.10.15: Load settlement curve of piles for 9mm tip settlement of pile, where pile shaft is subjected to constant normal stress.

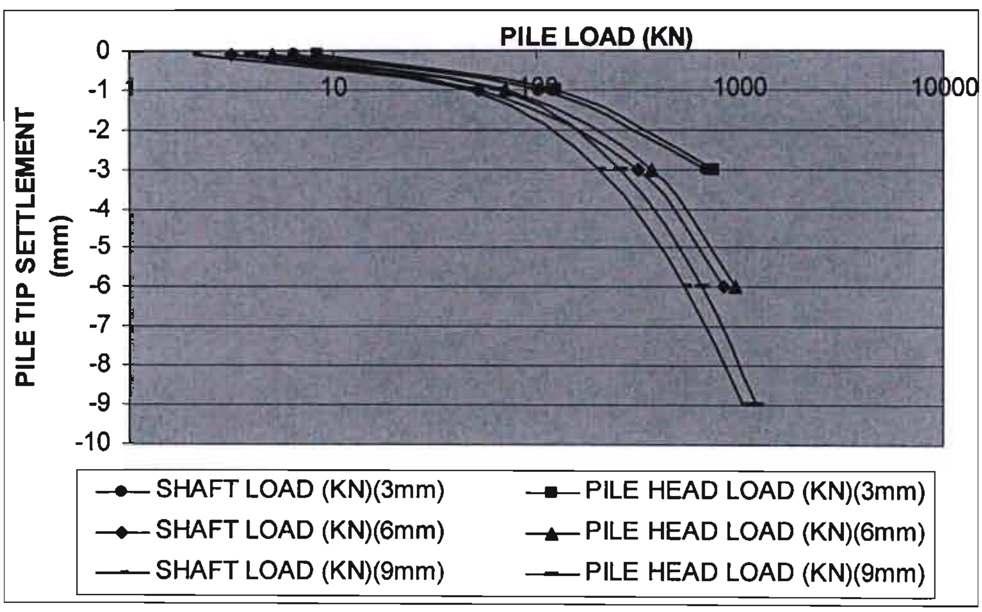


FIG.10.16: Pile load settlement curve where pile shaft is subjected to constant normal stress.

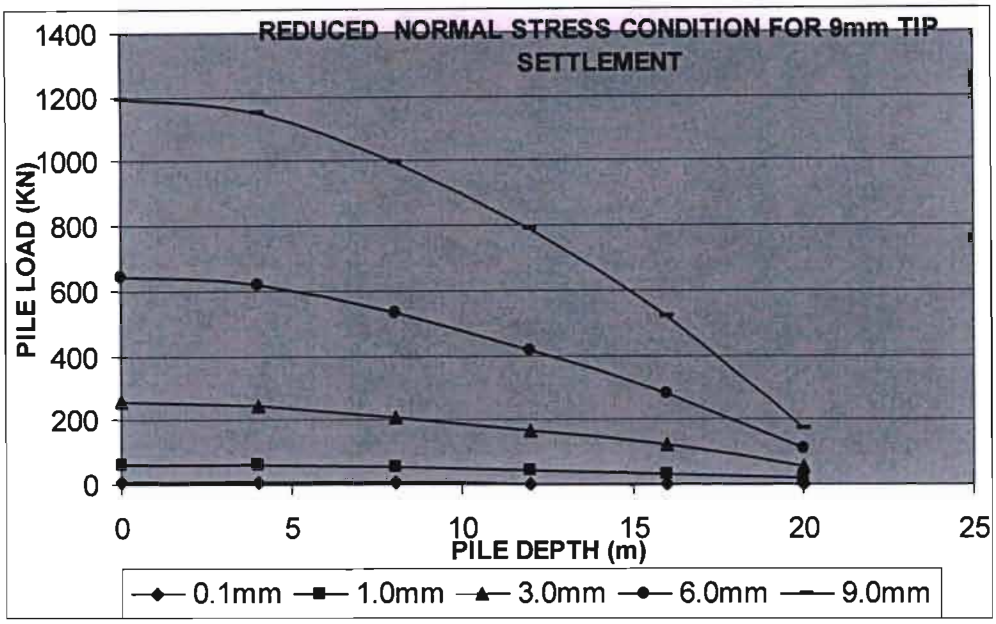


FIG 10.17 Curves of pile load at different pile depths for 9mm tip settlement of pile, where pile shaft is subjected to constant but reduced value of normal stresses due to compression of the plastic zone of soil close to the pile surface.

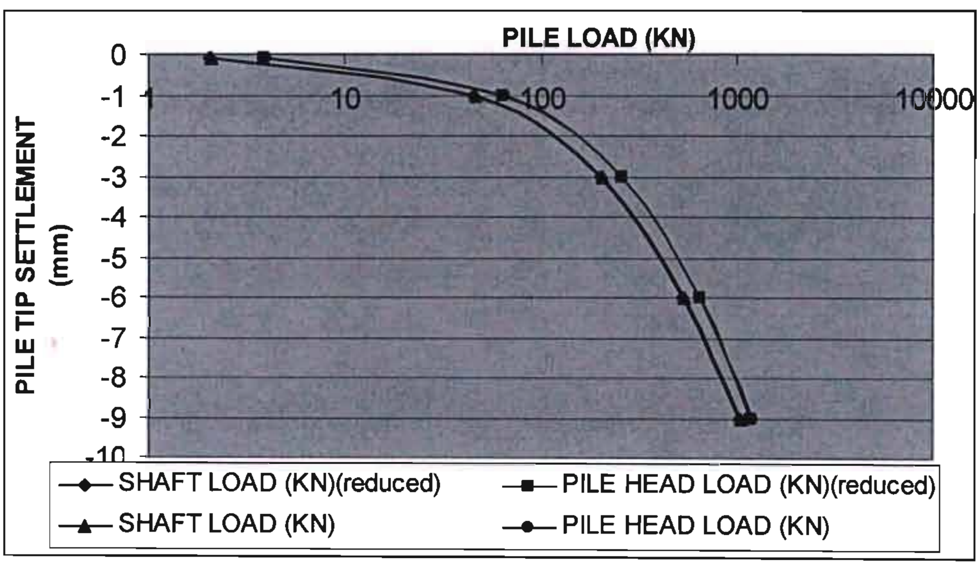


FIG 10.18: Load settlement curve of piles for 9mm tip settlement of pile, where pile shaft is subjected to reduced constant normal stress.

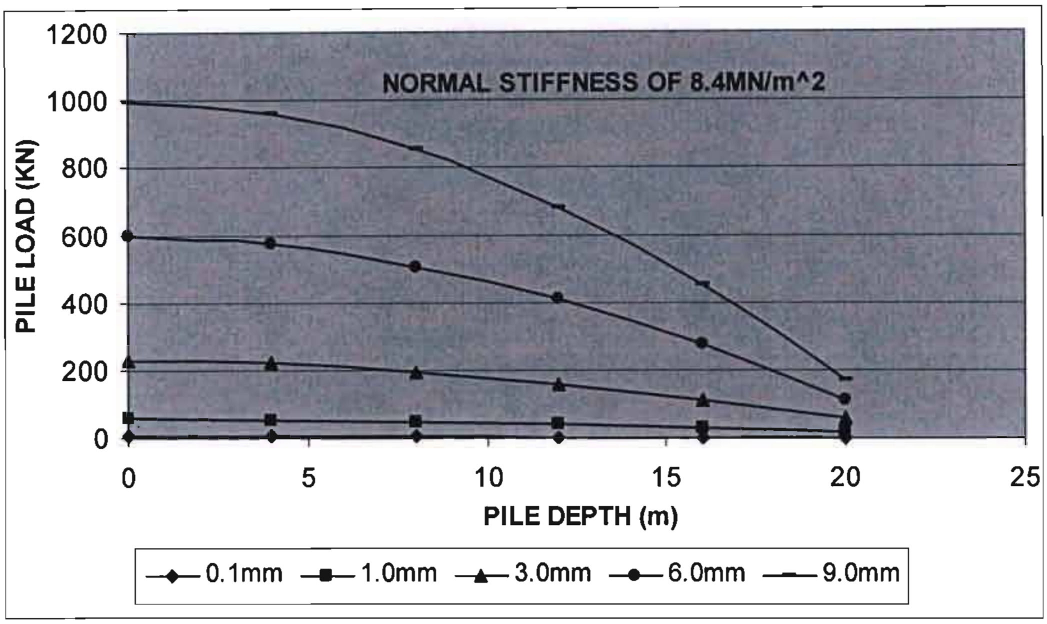


FIG.10.19: Curves of pile load at different pile depths for 9mm tip settlement of pile, where pile shaft is subjected to normal stiffness of 8.4 MN/m^2.

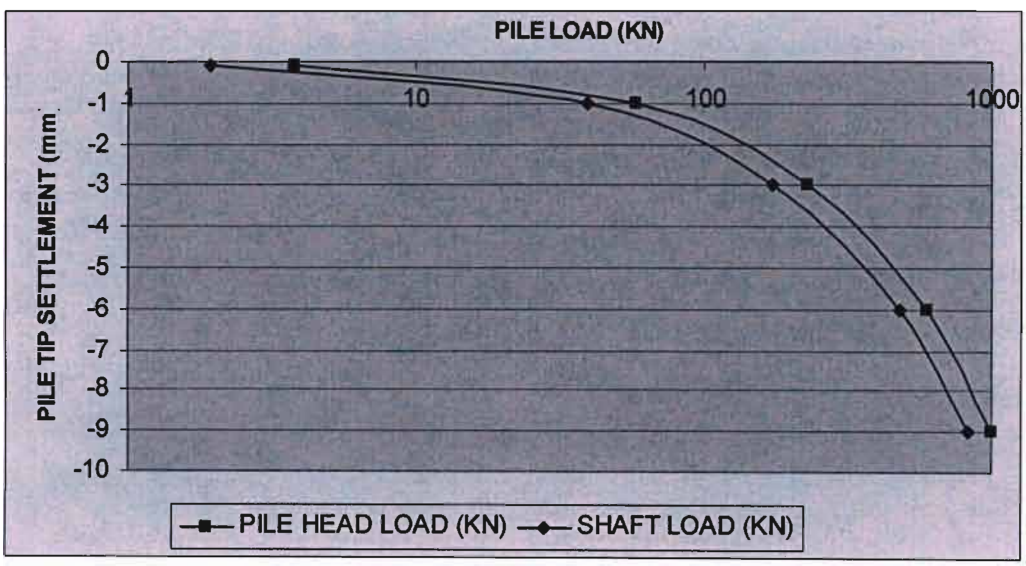


FIG.10.20: Load settlement curve of piles for 9mm tip settlement of pile, where pile shaft is subjected to normal stiffness of 8.4MN/m^2.

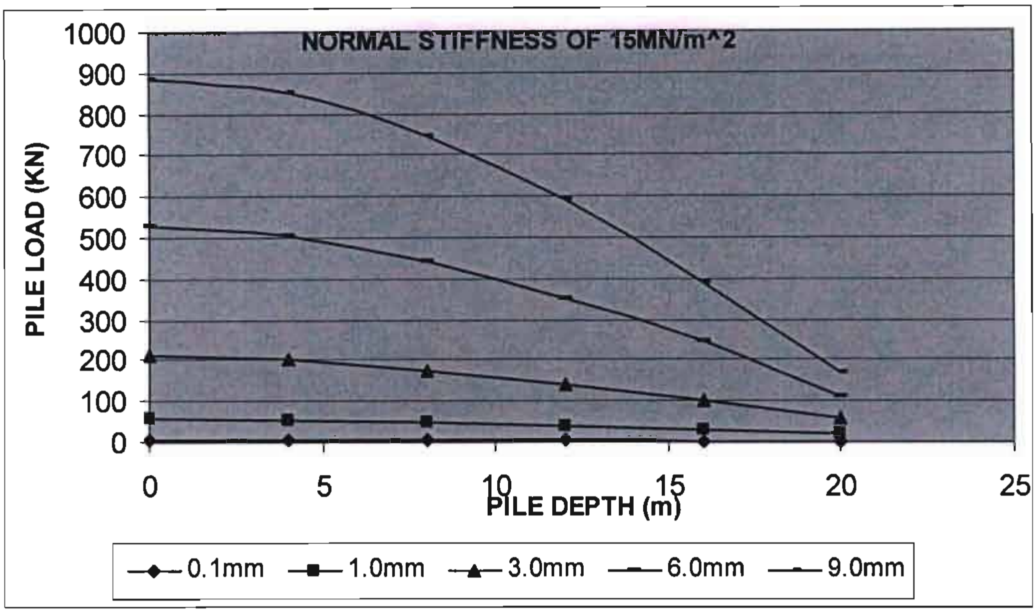


FIG. 8.21: Curves of pile load at different pile depths for 9mm tip settlement of pile, where pile shaft is subjected to normal stiffness of 15 MN/m^2

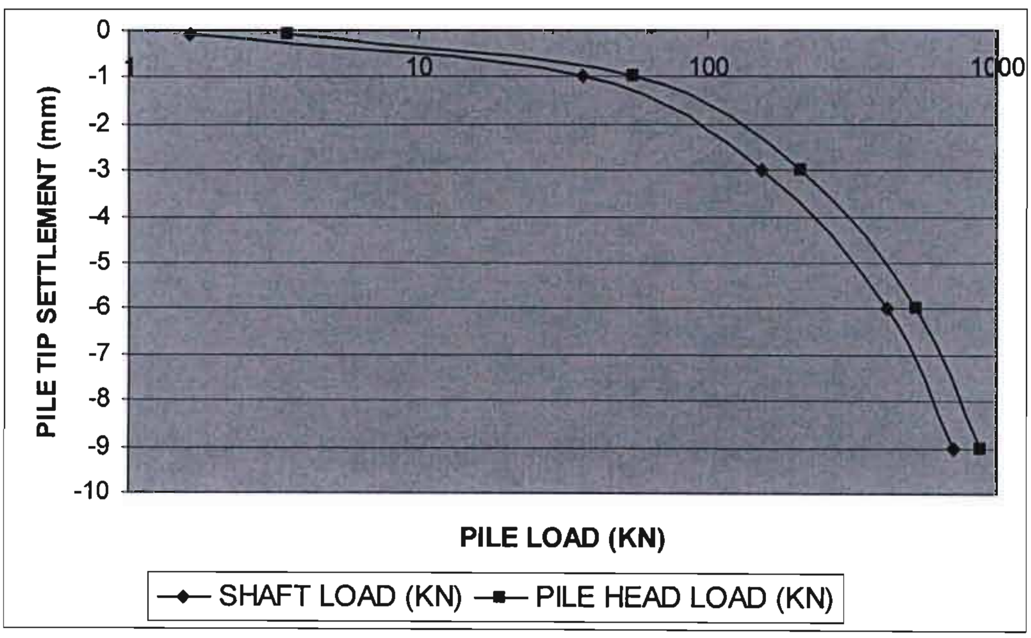


FIG. 10.22: Load settlement curve of piles for 9mm tip settlement of pile, where pile shaft is subjected to normal stiffness of 15MN/m^2.

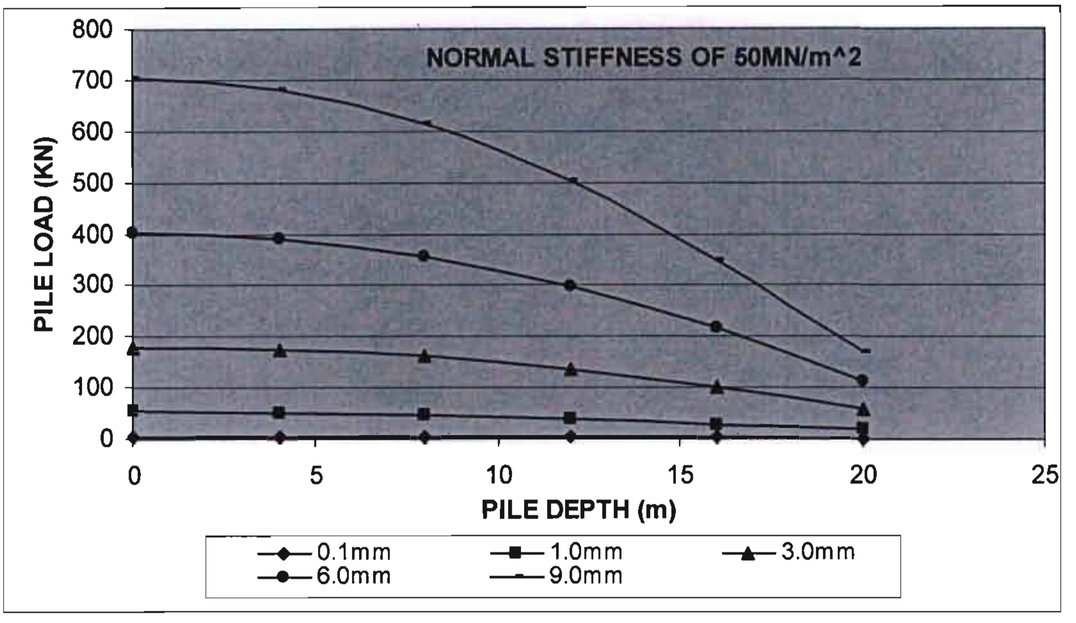


FIG 10.23: Curves of pile load at different pile depths for 9mm tip settlement of pile, where pile shaft is subjected to normal stiffness of 50 MN/m^2

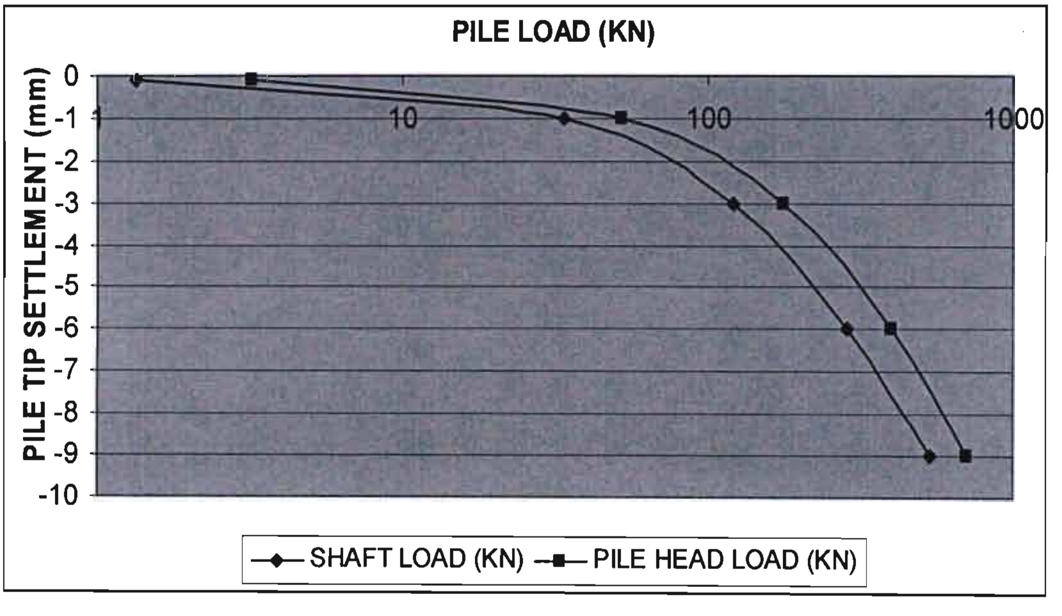


FIG 10.24: Load settlement curve of piles for 9mm tip settlement of pile, where pile shaft is subjected to normal stiffness of 50MN/m^2.

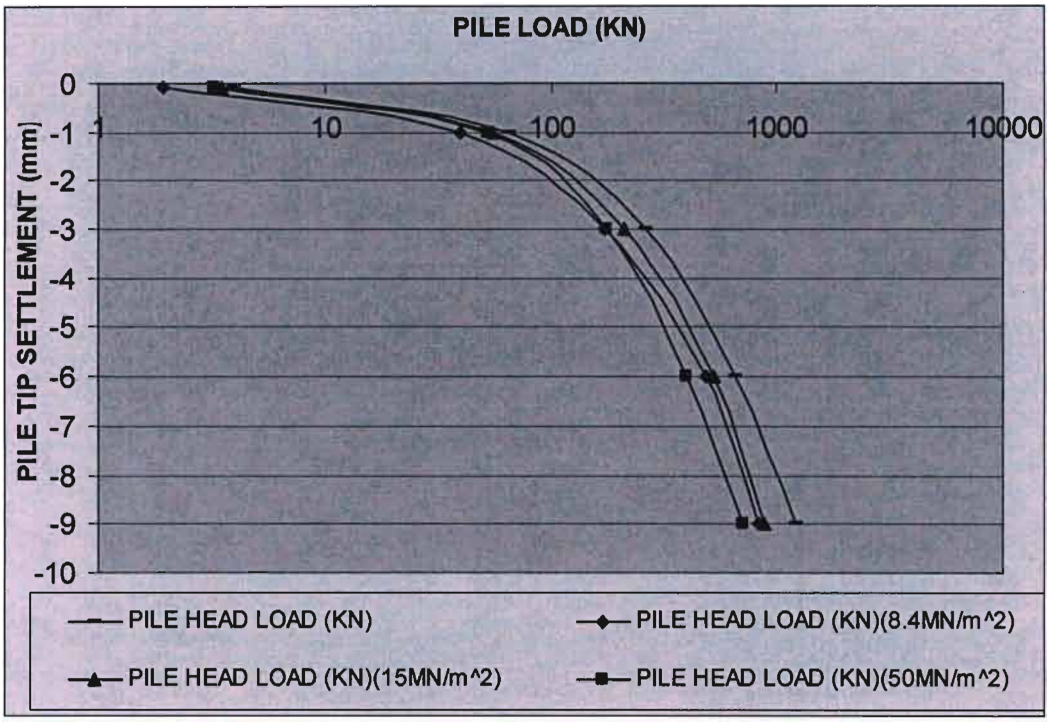


FIG 10.25: The combined load settlement curves of piles for 9mm tip settlement of pile showing the effect of different elastic stiffness of the elastic zone on load settlement curves.

CHAPTER ELEVEN

GENERAL DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

11.1 INTRODUCTIONS

It is a major engineering concern that excessive settlement of piled foundations of off shore and on shore structures in low density sands has been reported in some cases where detailed program of site investigation preceded design and development program. It is thus necessary to evaluate the influence of the stress – strain properties of low density sands on the load - settlement behavior of piles installed in low density sands.

Berea Red Sands are reddish dune quarzitic sands, resulting from the weathering of felpars and ferromagnesium rocks. Excessive settlement of structures in Berea Red Sand Deposits has been reported and while excessive settlement of piled structures in Berea Deposits has been linked to the collapse of the soil structure after heavy flooding, excessive settlement of piled structures have also been reported in low density carbonate deposits offshore, thus raising the question as to whether collapse of Berea Deposits can only be induced by water.

Unfortunately pile load tests cannot be conducted for all structures because of the limitations in terms of cost and scope as the result of one pile test cannot be extrapolated to various depths and soil conditions. The above limitations make the laboratory modeling and study of field problems very attractive and indispensable.

The aim of this research is to investigate whether or not significant contractile strains are induced in low density unsaturated residual sands subject to simple shear strain and then to assess the effect of such contractile strains on the shear and radial stresses around the shaft of a vertically loaded pile, pile load settlement behaviour and pile capacity assuming the pile was installed in Berea Red Sands deposit.

The series of studies that embody this research work were divided into four major sections. The summary is detailed below.

11.2 GENERAL DISCUSSIONS AND CONCLUSIONS

11.2.1 PHYSICAL PROPERTIES OF BEREA RED SANDS

Preliminary testing program were initiated to determine the physical properties of Berea Red Sands samples. The results of the tests are summed up below.

The particle size analysis revealed that Berea Red Sands are fine sand deposits made up of 10 percent clay, 5 percent silt, 82 percent fine sand and 3 percent coarse sands. The low percentage of fines indicates that the samples collected are still at the early stage of weathering.

The specific gravity of Berea Red Sand is 2.70. The specific gravity of quartz is 2.65, kaolinite is 2.61 and that of haematite is between 4.90 and 5.30. The specific gravity of Berea Red Sands, which are quartzitic sands, is due to the contribution from the iron oxide coating.

The compaction properties of Berea Sands were determined by the Proctor method. The maximum dry density of Berea Red Sand is 1785kg/m^3 and the associated optimum moisture content is 9.2%. Since samples compacted dry of the optimum moisture content tend to exhibit moisture induced collapse, all the samples used for the study of mechanical properties were prepared at dry densities below the maximum dry density and studied at moisture contents wet and dry of the optimum value.

11.2.2 OEDOMETER AND DIRECT SHEAR INDUCED COMPRESSION OF BEREA SANDS

Series of oedometer tests were conducted to study the effect of degree of saturation, moisture content and soaking stress on the collapse settlement behavior of compacted Berea Red Sands.

For compacted Berea Red Sands there are combinations of initial dry density, moulding water content and applied normal stress at which insignificant volume change and maximum collapse will occur when the soil is inundated. Samples compacted above dry density of 1716 kg/m^3 and average molding water content of 7.8 % will exhibit minimal collapse settlement irrespective of

soaking pressure while samples compacted at dry density of 1407 kg/m^3 and average molding water content below 4.2 % will exhibit collapse settlement of 12 % and above.
For any given set of condition, the amount of collapse of Berea Red Sands generally decreases with increasing precollapse moisture content, increasing precollapse dry density and decreasing soaking pressure.

A general trend of decreasing angle of friction, from 31° to 28°, with increase in moisture content at which samples were tested was indicated although a constant value is expected. The difference in value may be due to decrease in intergranular friction resulting from increasing lubrication of the surface of the sand grains with increase in moisture content. The angles of friction of 28° - 31° are slightly lower than the triaxial values of 30° - 33° reported by Brink (1984).

Within the range of moisture content and dry density of interest, compacted Berea Sand exhibited a decrease in sample thickness due to imposed direct shear strain. The shear-induced compressions tend to increase with increase in normal stress. This is clearly indicated in samples tested at moisture contents less than 13.5 %. The above trend is not clearly indicated in samples tested at average moisture content of 13.5 %. It must however be noted that shear stress mobilization in the direct shear box is limited to a narrow band of soil close to the pre selected shear plane at the middle of the box. The shear induced compression is therefore not representative of the entire thickness of the sample and so contractile vertical strain cannot be computed from shear induced compression of Berea Sands in a direct shear box apparatus. And thus the need to develop a simple shear device.

11.2.3 DESIGN, DEVELOPMENT AND EVALUATION OF A SIMPLE SHEAR APPARATUS

The stress conditions in the NGI and Cambridge type simple shear apparatus differ because of the different types of imposed test boundaries however the regular geometry and rigid nature of the Cambridge type simple shear box permits measurements and study of stresses and strains conditions in the boundaries of a sand specimen with reasonable accuracy.

The analysis of sand specimens subjected to simple shear strain in the Cambridge type apparatus was carried out in stages.

The direction of the major principal stress at failure may be such that the major principal stress is already orientated at ± (45 - ψ/2) to the horizontal direction when failure occurs. Geometrically from the Mohr circle therefore, the horizontal plane through the soil sample is a plane of maximum shear stress and is determined the stress ratio at failure has a maximum value at an intermediate value of *k*, is equal to zero at active and passive pressure states.

The boundary conditions imposed by the walls of the Cambridge type simple shear apparatus does not permit either K_a or K_p stress conditions at failure, since this are limiting infinite conditions. For residual sands compacted to low density, stress conditions during constant volume simple shear tests imply that k changes from K_o to 1, i.e. $\sigma_x = \sigma_y$ since k = 1 correspond to the limiting stress value $\tau_{xy} = \sigma_y \sin\varphi$. For the same soil tested under constant normal stress conditions, bulk volume collapse of soil structure within fixed lateral boundaries implies that k changes from K_o at start of test, then asymptotically approach 1, K_m or K_p. However bulk volume dilation of soil structure within fixed lateral boundaries implies that k changes from K_o at start of test then asymptotically approach K_a.

However since the typical Cambridge type apparatus imposes simple shear strain on a specimen confined in a rigid square or rectangular wall by the rotation of the end flaps or plates, the simple shear strain imposed on a sample compacted into the shear box does not result in the pure shear conditions represented by the Mohr circle because of uncomplimentary stresses on the vertical surfaces of the specimen and end effects (Roscoe 1953, Duncan and Dunlop 1982 and Airey et al, 1982), and thus the need to evaluate the stresses at the boundaries of a specimen subject to simple shear strain.

The average shear stress mobilized by the sample of length L and height H is the externally applied force per horizontal cross sectional area of the sample. Thus considering unit width of sample, the simple shear stress equation is of the form;

$$\frac{F\cos\alpha}{L} = \frac{K_u\gamma' H^2\cos\alpha}{6L} + \frac{K_u qH \cos\alpha}{2L} + \frac{K_u\gamma' H^2\sin^2\alpha}{6L} + \frac{K_u qH\sin^2\alpha}{2L} - \frac{K_o\gamma' H^2}{6L} - \frac{K_o qH}{2L}.$$

The above equation is the simple shear stress equation for a sample subjected to simple shear strain assuming unit width of the specimen. It satisfies the conditions that no shear strength is mobilized by the specimen when subject to zero imposed simple shear strain. For a soil specimen of given dimension, the shear strength can be determined for any value of simple shear strain α and applied stress q and dry density γ. The above equation is limited in that it cannot determine the effect of changes in water content of the sample on shear strength. Because the shear strength of soils decreases with increasing water content, shear strength determined by the simple shear stress equation represents the upper bound value or the shear strength of the dry sample for given values of applied stress q and dry density γ.

The effect of the dimension i.e. the height/length ratio of the specimen on the mobilized simple shear stress was evaluated by considering the stress ratio at different failure conditions.

The range of H/L for which the specimen can deform in simple shear is given as

$$0 \leq \frac{H}{L} \geq \frac{2 \tan \phi}{k \cos \phi + k \sin^2 \phi - K_o}$$

Thus the maximum height to length ratio of the specimen that permits simple shear deformations is dependent on the internal frictional angle of the sand specimen (Palmer and Rice 1973, Chowdhury 1978). Beyond the maximum height to length ratio, deformation of the specimen is no longer of simple shear mode. For Berea Sand, $K_o = 0.484$, $K_p = 3.126$, $K_a = 0.3198$ and H/L = 0.83.

A square cross sectional area of 70mm by 70mm and of thickness of up to 60mm was selected for the new device. The choice of area was dictated by the need to accommodate the device within the existing structure for the conventional direct shear box apparatus. The new device was modeled after the Cambridge device because of the need to ensure fairly accurate measurement of changes in vertical height of the sample under imposed simple shear strain. A construction problem common to Cambridge type devices is that a perfect fit cannot be maintained between flap and the base of the device as the flap rotates. This problem was overcome in the new device by constructing the flaps with extended pins, which are located at the center of rotation of the flap there by ensuring that the flap remains in perfect alignment and close fit with the base of the device.

Two models of the simple shear device were constructed and tested. The first model induces simple shear strain on the top and bottom surfaces of the samples by a pair of steel ropes connected to the top platen. The curves indicated that the shear strength of Berea Sand compacted to low density is equal to the applied normal stress for all values of normal stresses used. This is unacceptable in soil mechanics and thus the need for another model.

The second model (the type B device) imposes simple shear strain on compacted samples by means of stiff horizontal rod on the flap. The stress - strain curves of model B device are similar to the curves determined from direct shear box test data on same soil. Thus the type B device was adopted for further evaluation and study of the mechanical behavior of compacted Berea Red Sands.

Further evaluation of the new device in relation to test conditions that may affect the results of tests conducted in the new apparatus was undertaken. Test program initiated to investigate whether changing the height to length ratio of the test specimen leads to changes in the stress-strain behavior of a soil stress subject to simple shear strain revealed that their may be a limiting ratio of specimen dimension, beyond which the mobilized shear stress begins to decrease. For compacted Berea Red Sands this ratio is between 0.42 and 0.58. The tests indicated that for samples with height to length ratio below 0.42, the same value of mobilized shear strength is to be expected.

The vertical strain – shear strain curves for the 15 mm, 30 mm and 41 mm samples indicate shear induced volume compression and decreasing rate of reduction in soil volume with increasing shear strain. Vucetic and Lacasse (1982) noted similar behavior from tests on Haga soils.

The 30mm height or height to length ratio of 0.42 was selected as the sample dimension for the investigation of the mechanical properties of compacted Berea. Sands. Potyondy (1961) and Uesigi and Kishida (1986) concluded that beyond 35mm it was difficult to ensure uniform stress and strain distribution within the soil mass in direct shear. Conducting simple shear tests with 30mm thick samples rather than with 15mm thick samples ensures that a reasonable thickness of the soil mass becomes the shear zone, permits direct measurement of a wider range of data on sample deformation, allows for a more consistent measurement of strains and ensures that changes in the mechanical properties are direct reflections of changes in initial state i.e. moisture content and dry density at which the samples were prepared and sheared.

Compacted Berea Red Sands samples were tested without membranes, with whole samples covered with membranes and also with samples were tested with membranes on flaps only. The stress – strain curves show that the membranes resulted in reduction in measured shear stress.

However the reduced value may not be representative or intrinsic of the mechanical behavior of compacted Berea Sands as simple shear deformation is associated with cross diagonal contraction and extension of the membrane that causes reduction in mobilized shear stress. Vucetic and Lacasse (1982) reported unreliable measurement and inconsistent vertical strain data when very thick (2 mm) rubber membranes were used in the NGI device. Since correct measurement of contractile strain that can be related to pile capacity is a primary objective of this research, it was decided to carry out the remaining investigation on 30mm thick samples tested without the membrane.

Series of constant normal stress tests were conducted on dry Berea Red Sands samples. The samples were dried down to moisture contents less than 1% .at room temperature. The stress strain curve of dry Berea Red Sands was compared with the stress – strain curves generated by the use of simple shear stress equation since the simple shear stress equation cannot predict shear stress response to changes in soil moisture content.

The general trend observed is that simple shear stress can be predicted at small values of imposed simple shear strain since non uniformity of internal stresses and strains within the samples are not significant at small values of imposed shear strain. Simple shear devices impose simple shear strain on samples but do not impose complimentary shear stresses on the vertical surfaces of the samples. Complimentary shear stresses were incorporated in the formulation of the simple shear stress equation. The effect of the complimentary stresses does to result in increase but a decrease in mobilized shear stress, and the magnitude of the complimentary stresses increases with increase in imposed simple shear strain. The primary deduction is that reliable values of shear modulus can be determined from the stress – strain curves generated from constant normal stress simple shear tests data using the new apparatus.

Series of normal stiffness tests were conducted on dry Berea Red Sands samples. For tests with normal stiffness of (140N/mm), 8.4 MPa, the analytical stress - strain curves diverged from the experimental curves at imposed strains of 5 – 8 %, the behavior is similar to that of samples tested under constant normal stress conditions. For imposed strains higher than 8 %, the experimental shear stress is higher than the analytical value and the difference may be due to the fact that internal stress and strain non uniformity in sands increases with increase in simple shear strain as well as the effect of non complimentary stresses on mobilized shear stress increases with increase in imposed simple shear strain.

The normal stiffness tests were also conducted with springs of stiffness constant of 250 N/ mm on compacted samples. The predicted normal stiffness stress – strain curves overestimates the experimental curves at imposed strains up to 15 %-20% and underestimates the experimental at strains higher than this range. Normal stiffness tests were also conducted with springs of stiffness constant of 50 MPa (1660 N/ mm). The Analytical l curves indicated stiffer stress - strain response to imposed simple shear strain and intersects the experimental curves at imposed strains within the range of 15 %-20 %.

For all compacted samples tested with stiffness constant of (140 N/mm) 8.4 MPa, (250 N/mm) 15 MPa and (1600 N/mm) 50 MPa and initial normal stresses of 40 kPa to 400 kPa, the experimental values of simple shear stresses at which the experimental and analytical curves started diverging i.e. for samples tested with 140N/mm or intersect i.e. for samples tested with 250N/mm and 1660N/mm are slightly less than the peak shear stress indicated by the analytical curves. These shear stress values may be taken as the peak values since the experimental curves continue to increase with increase in imposed shear strain beyond these points.

The experimental stress – strain curves for samples tested at normal stiffness of 140 N/mm, 250 N/mm, and 1660 N/mm indicate that the stiffness of the sample increases with increase in imposed normal stiffness.

Constant volume simple shear tests were conducted on dry Berea Red Sands. All the samples tested indicate peak values of shear stress. The curves indicate large difference between the experimental and the predicted stress – strain behavior which may imply that the simple shear equation cannot predict the constant volume response of compacted Berea Red Sands. However the corrected analytical curves of the constant volume tests are similar in shape to the curves presented by Bouy and Carter (1988) and Jewell (1988) from series of tests conducted on artificially cemented sands with cement content of 5% – 15%, using the University of Sydney constant normal stiffness direct shear device.

While the experimental curves are generally bilinear, the analytical curves indicated strain softening behavior for all initial normal stresses, implying that dry samples of compacted Berea Red Sands exhibit brittle behavior under constant volume shear condition.

11.2.4 SIMPLE SHEAR BEHAVIOR OF COMPACTED BEREA SANDS

The effect of changes in moisture content on constant normal stress simple shear, normal stiffness simple shear and constant volume simple shear behavior of compacted Berea Red Sands was also investigated. These types of simple shear tests are relevant to the study of stress state of the soil around a vertically loaded pile. Before and after the constant normal stress tests, the matric suction of each sample was tested using Whatmann No 42 filter paper following the method of Chandler and Gutierrez (1988) and Chandler et al (1992).

For the constant normal stress tests, no defined peak was observed from the stress - strain curves within the range of normal stress and moisture contents used for these tests. The shear stress continued to increase beyond the maximum imposed strain of 33 % used for the series of tests.

The samples also indicated shear induced bulk volume compression that decreases with increase in the magnitude of applied constant normal stress. The shear induced compression behavior in simple shear is thus different from shear induced compression behavior of samples tested in conventional shear box.

Series of constant stiffness tests were conducted on samples compacted into the simple shear box at moisture content of 0.2% - 0.8% and 7.55% - 8.45%.The initial normal stresses were applied by springs of (8.4 MPa) 140 N/mm, (15 MPa) 250 N/mm and (50 MPa) 1666 N/mm Modulus.

The stress strain curves are similar to the constant normal stress curves, with shear stress increasing continuously for normal stress range and different moisture contents at which the samples were tested. However for all the stress range used, the measured shear stresses are lower than the constant normal stress plot due to the gradual decrease in the applied initial normal stresses with increase in imposed shear strain.

The vertical strains and mobilized strengths of compacted Berea Sand measured in the constant stiffness stress tests using spring of 140 N/mm stiffness are lower than the values recorded for constant normal stress tests for corresponding values of initial normal stress. It can also be seen from the constant stiffness stress – strain curves that irrespective of the moisture content at which the samples were tested, the ratio of the maximum shear stress to the initial normal stress indicated by compacted samples of Berea Red Sands decreases with increasing initial normal stress. This implies that the structural rigidity of compacted Berea Sands is significantly destroyed at high initial normal stress.

The constant volume curves are generally bilinear. They all indicated well defined peak values of shear stress at imposed strains of between 8% and 12%. The samples however did not exhibit the post peak drop in shear stress typical of brittle materials. The largest stress ratio is indicated by sample tested under the lowest initial normal stress of 40 kPa and decreases with increase initial normal stress. The trend is a clear indication of the effect of magnitude of normal stress on the structural rigidity of compacted Berea sands. At low normal stresses, the structure of compacted sands is not significant destroyed and so the initial structural rigidity makes a significant contribution to mobilized shear strength and stress ratio.

In order to assess whether the filter paper method can measure changes in matric suction resulting from small changes in the volume or height of a soil sample under constant moisture content, matric suction tests were conducted on samples of Berea Red Sands which were subjected to incremental stages of vertical compression stress. The results indicate that the filter paper method can reproduce results when the test procedure is carefully adhered to. The results show that the change in matric suction of Berea Red Sands at moisture content of above 13% with respect to normal stresses and vertical strain are not significant. The slope of the 7.3% samples indicates that for the relatively dry samples, the matric suction is extremely sensitive to small changes in saturation. Thus it is expected that contribution of matric suction to the mobilized strength of compacted Berea Sands beyond 14% moisture content is insignificant.

Preliminary test on the relationship between effective shear stress and matric suction at constant moisture content carried out on 4.2 % moisture content samples in a direct shear box indicates that changes in matric suction is independent of the effective normal stress. At relatively higher values of normal stresses, the larger total deformation result in significant net increase in degree of saturation and a fairly constant arrangement of menisci, thus significant change in matric suction is expected from increasing normal stresses yet a constant value was indicated for normal stresses higher than 100kPa.. Similar trend are also indicated for samples tested at different moisture content in the new simple shear device.

11.2.5 LOAD SETTLEMENT BEHAVIOR OF PILES IN BEREA SANDS

The capacity and load settlement behavior of vertically loaded piles installed in Berea Red Sand formations was studied. The numerical technique, which is based on the load transfer concept presented by Coyle and Reese (1966), incorporated the effect of decreasing lateral stresses due to shear induced volume reduction of the soil close to the pile surface as well as the resistance of the soil below the pile tip. Both the constant normal stress data and the constant normal stiffness data used in load settlement iteration were taken from the stress – strain curves generated from data on series of tests con ducted with the new simple shear device.

The distribution of the pile head load and shaft load with depth and the load settlement curves for assumed tip settlement of 3mm, 6mm and 9mm assuming constant horizontal stress normal to pile shaft were determined. The results indicate that pile capacity increases with increase in tip settlement from 0 to 6mm. Beyond 6 mm the increase in pile capacity is constant thus settlement to mobilize the pile capacity 6mm. The pile head – pile depth curves revealed that over 90% of the applied load or the pile head load is carried by the shaft.

Investigation of the effect of the compression of the plastic zone alongside the pile shaft on capacity and load settlement behavior revealed that the average shear induced strain of 10% in the plastic zone resulted in a decrease of below 5% of mobilized capacity for tip settlement of 9mm, and indicated no difference in load settlement curve for pile analysis where the elastic modulus of the outer elastic zone is equal to 6.76 MN/m^2, the modulus determined from the constant normal stress curves.

The distribution of the pile head load with depth and the load settlement curves for assumed tip settlement of 9mm assuming the pile shaft is subjected to normal stiffness of the outer elastic soil region revealed that the pile capacity decreases with increase in normal stiffness and up to 50% decrease may be expected for elastic modulus of 50 MN/m^2 . The pile analysis also revealed that as the outer elastic boundary becomes stiffer, more of the applied vertical load is carried by the pile tip.

The important conclusion drawn from pile analysis is that reduction in shaft capacity may not be significant if the elastic modulus of the outer soil is low irrespective of the magnitude of the compression of the plastic zone, but significant reduction in horizontal stress and shaft capacity would be expected if the elastic modulus of the outer soil is high.

The shaft capacity of piles in Berea collapsible soil is dependent on both the compression of the soil close to the pile surface as well as the stiffness of the outer soil. The pile load –depth curves clearly revealed that if the outer soil is very stiff, very small changes in the thickness of the plastic zone results in large reduction of horizontal stress and pile shaft capacity.

11.3 RECOMMENDATIONS

Simple shear behavior of Berea Red Sands compacted to low density was studied and the initial tangent modulus approximated from the constant normal stress curves at low imposed simple shear strain were found to be close to that estimated from tests conducted with normal stiffness of 140 N/mm (8.4 MPa). Determination of initial tangent modulus required the use of high precision instrument. These instruments are very expensive and since the determination of the correct value of soil initial modulus is im portent in pile load settlement analysis. Cost effective laboratory and predictive methods of determining soil modulus need to be developed.

The insitu deposits of Berea Red Sands and most residual soils are weakly cemented , the bond strength vary over a wide margin and the stress and compression properties of cemented soils are influenced by bond strength. The behavior of bonded soils in simple shear and the effect of different bond yield and deformation on pile capacity can be modeled if careful program of laboratory production of cemented soils is initiated. The shear compression of compacted low density Berea Sand is not large and so is the reduction in pile capacity. The deformation of bonded in situ deposits results in very significant reduction in soil volume, thus loads settlement behavior and pile capacity need to be investigated.

It has so far not been possible to separate the compression associated with normally consolidated soils with that of compacted soils and over consolidated soils. In the present study it is assumed that the measured compression is a combination of the two. Production of cemented samples would enable the study of the separate effects of bonding and remoulding on measured mechanical properties.

All the soils tested were unsaturated and the matric suction was determined by the filter paper method, both before and after the simple shear tests. While this test is adequate to evaluate the change in matric suction associated with shear induced volume change, it is limited in the assessment of the effect of changes in matric suction on mobilized shear stress. The later in needed for the mechanical characterization of unsaturated soil in terms of effective stress and suction. Further tests using a controlled suction direct shear device is thus necessary.

A simple shear device that imposed simple shear strain has been built. While efforts has been made to overcome some of the problems associated with the design and construction of the Cambridge simple shear device, the question as to whether the rigid wall adopted by the Cambridge type device or the flexible wall adopted by the NGI group or a combination of the two generic models will imposed simple shear strain closest to the conditions experienced by the soil in situ remains a mystery. A device that incorporates some of the attributes of the two generic models is worthy of investigation as it may help resolve the problem associated the continuous increase in measured shear stress when samples are tested within laboratory standard strains.

The t –z method was used to study the load settlement behavior and capacity of a vertically loaded pile in Berea Red Sands. This method enables the study of the effect of plastic zone compression in load settlement behavior and mobilized capacity. The width of the plastic zone was assumed to be equal to 30mm which is the initial height of the samples tested in the simple shear device. The results obtained in pile analysis will be viewed with very high confidence if controlled and instrumented pile load testing program is conducted in Berea Sand formations, as this would facilitate the development of useful correlation between laboratory, analytical and the field results and pile design charts.

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ARCSMFE; African Regional Conference Soil Mechanics and Foundation Engineering

ICSMFE; International Conference Soil Mechanics and Foundation Engineering

ASCE; American Society of Civil Engineers

ASTM American system for the testing of materials

SMFE; Soil Mechanics and Foundation Engineering

TRRL; Transport and Road Research Laboratory

SAICE South African Institute of Civil Engineering