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A study of some impacts of structure on the mechanical behaviour of geomaterials

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ABSTRACT

FACULTY OF ENGINEERING, SCIENCE AND MATHEMATICS

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A STUDY OF SOME IMPACTS OF STRUCTURE ON THE MECHANICAL BEHAVIOUR OF GEOMATERIALS

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This thesis presents a study on the structural behaviours of synthetic geomaterials due to its significant effects on soil strength and stiffness characteristics. Based on the review made of the previous studies on the fundamental soil behaviour, it has been seen that structural properties, which can be described as combination of the particle arrangement and the interparticle attractions, are as important as the specific volume and stress history to describe the engineering properties of geomaterials. The study firstly investigates the influence of interparticle attractions by creating artificial bonding through soil particles under controlled pressure and temperature conditions using sucrose ($C_{12}H_{22}O_{11}$). Therefore, a temperature controlled triaxial apparatus has been developed that includes instrumentation capable of measuring the stiffness of a specimen at small strain levels during the triaxial loading. Triaxial testing on the specimens resulted in a sharp decrease followed by gradual increase in the deviatoric stress, stiffness, and a structure permitted area in stress space, as well as a sharp increase followed by a gradual decrease in corresponding pore pressure. These successive drops and formations give a stick-slip nature to the fluctuations, which might be attributed mainly to the rate of loading, effective stress, pore fluid characteristics, crystal growth, compliance in the apparatus, and the initial fabric. The main reason of this jamming might lie in the fact that the internal forces do not uniformly propagate through the specimen but are localized through force chains of strained grains. It is interpreted that the process undergone by the sucrose-sand mixture may consist of the bonding of the sand particles. Furthermore, the study also provides an additional data set to compare the mica-rotund sand mixtures with some of the sucrose-rotund sand mixes in the triaxial apparatus, which supports the view that the high compressibility of the mixes is likely to be a result of particle shape.

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CHAPTER 1

INTRODUCTION

The chapter presents a review of the basic facts on soil behaviour at the microscale, and the essentials of methodology followed to develop a more reliable experimental technique to have a greater understanding of fundamental soil behaviour. The goal of the study as well as a brief discussion of background is also introduced to the reader.

1.1 General

The behaviour of soils in laboratory as well as field cannot be completely understood unless the effects of structure, which was first recognized by Terzaghi (1941) and later by Skempton (1944), are taken into account. Soils in situ usually possess natural structure which enables them to behave differently from the same material in a reconstituted state (e.g., Burland, 1990, Cuccovillo and Coop, 1999). Therefore, investigation of the amount and type of structure is of importance in understanding the engineering behaviour of both natural and synthetic geomaterials. For example, it is widely known that mechanical behaviour of natural clays can differ from the behaviour of reconstituted clays due to depositional and post deposition conditions. Burland (1990) proposed that in order to determine the influence of structure it is helpful to make comparisons between the behaviour of natural clays and those corresponding reconstituted counterparts which are obtained by remoulding. He then defined the properties of reconstituted clays as 'intrinsic' properties, which he considered to be inherent to the clay because of the absence of structure.

Two methods can be employed for such studies; either the use of high quality samples of natural soils or that of artificially bonded synthetic geomaterials. Even though, today, very high quality sampling techniques are available, implying very small disturbance, these high quality techniques (i.e., Sherbrooke, Laval) may still cause significant loss in structure, and further disturbance may be caused by sample transportation and specimen preparation for an experimental investigation in a laboratory (Clayton et al., 1992, and Hight et al., 1992). Numerous studies of artificially bonded materials have been made using various bonding agents in literature (e.g., Portland cement, Allman and Poulos, 1988; gypsum, Huang and Airey, 1998; calcite, Ismail et al., 2002; microorganisms, Yang et al., 1994, Ramachandran et al., 2001, Çabalar, 2002). However, these conventional approaches of artificially bonding techniques may introduce significant errors in describing structural influences. Bonding is achieved under zero effective stress and generally followed by isotropic compression, which does not accurately simulate the geological process in the natural environment.

This thesis presents the results of research that aims to generate a greater understanding of fundamental soil behaviour through the investigation of structure. Although the fundamental understanding of soil behaviour covers many parameters interrelated (e.g., size, shape and mineralogy of particles, interparticle arrangement, pore fluid and pore gas characteristics, the loads transmitted through the soil skeleton, particle level forces and contact level forces), the focus of this study research is on contact level forces, particularly pore fluid characteristics (i.e., viscosity); bonding; and the influence of particle shape phenomena. Following the overview made of the previous works dealing with mainly soil behaviour at the microstructural level, the thesis firstly describes a methodology in which weak bonding is achieved under controlled temperature and stress circumstances so as to more accurately mimic the bonding process in natural environment. For this purpose, an experimental programme in the temperature controlled triaxial cell has been designed. To understand the effects observed in some of the specimens tested, in the light of tribology, the study also presents a review and a significant discussion of the stick-slip behaviour in granular materials. Accordingly, the study provides a conceptual basis as well as an experimental work for the analysis of fundamental soil behaviour. Lastly, the study gives an additional experimental data set to make a detailed discussion of the effects of particle shape on the behaviour of a soil medium.

1.2 Aims and objective of the study

The fundamental behaviour of geomaterials will be investigated within a laboratory environment. Based on the hypothesis that bonding between the particles can be imposed on soils under controlled conditions after a stress history is applied, and without mixing cementing material before specimen formation, a more realistic experimental artificially bonding method utilizing a modified triaxial apparatus with a temperature control system will be introduced. Following the hypothesis that pore fluid can bind particles together by a combination of viscous and tack forces until solid bonds are formed, the effect of liquid viscosity will be studied by using (i) de-aired water, (ii) silicon oil, and (iii) sucrose solutions at various mix ratios. Furthermore, it is also hypothesized that the mechanical behaviour of sands can be significantly affected by presence of platy grains. Therefore, the study will present an experimental data set to compare the mica-sand mixtures with sucrose-sand mixtures.

Accordingly, the primary objectives of this research can be summarized as;

- To develop a laboratory technique for providing realistic bonding of a range of synthetic geomaterials (coarse-grained) under different states of stress.
- To investigate the influence of pore fluid characteristics (i.e., viscosity) on the specimens to be tested.
- To re-assess the effects of platy particles on the behaviour of a sand.

1.3 Organization of the thesis

Following this introduction, Chapter 2 presents an overview of: (i) bonding, fabric and stress-strain behaviour of soils in yielding zones, (ii) available artificial bonding techniques, (iii) the stick-slip mechanism, and (iv) the effects of particle shape on the behaviour of geomaterials. New insights into the problem from lessons learned from recent findings are also discussed in this chapter.

Chapter 3 describes the methodology followed to develop an experimental technique for the artificially-bonded soil studies. Particular attention is given to the modifying of a conventional triaxial testing apparatus.

In Chapter 4, data obtained from the experimental study are analysed and discussed.

Finally, a summary of the research, major conclusions, and recommendations for future areas of the study are presented in Chapter 5 and 6.

CHAPTER 2

LITERATURE REVIEW

This chapter deals with the influence of structure on the properties of geomaterials and describes some of the latest experimental observations related to stress, strain and yielding characteristics of soils. It starts by introducing the structural properties and their effects on soil behaviour. The chapter then describes the effects of bonding and fabric on the stress-strain behaviour of soils by defining yielding zones located within a bounding surface in normalized stress space. The chapter also reviews previous investigation into recent artificial bonding techniques. The mechanism of stick-slip behaviour in granular materials and the effect of particle shape on the behaviour of geomaterials are summarized.

2.1 General

Soil structure, which is closely linked with the engineering behaviour of the geomaterials (Collins and McGown, 1974), was defined as the combination of geometric arrangement of soil particles with respect to each other (fabric) and interparticle attractions (bonding) by Mitchell (1976). The importance of structure was first recognized by Terzaghi (1925) and confirmed by Casagrande (1932), and Skempton (1944). The influence of structure on the engineering behaviour of natural geomaterials has been more recently illustrated in a number of papers (e.g. Leroueil and Vaughan, 1990, Burland, 1990, Cuccovillo and Coop, 1993, Cotecchia and Chandler, 1997, Liu and Carter 1999-2000, Cotecchia and

Chandler, 2000, Fearon and Coop, 2000, Fernandez and Santamarina, 2001, Malandraki and Toll, 2001, Baudet and Stallebrass, 2004).

It is generally accepted that the structure of natural geomaterial is formed during its formation because of mechanical, chemical and biological factors. Although many different factors cause the development of structure including depositional conditions, ageing, cementation, chemical and biological factors, effects of different types of structure on the engineering behaviour of natural geomaterials is thought to be similar (Leroueil and Vaughan, 1990). In order to evaluate the structural effects, it can be helpful to make comparisons between the behaviour of natural soils and corresponding reconstituted soil (intrinsic behaviour) that is obtained by remoulding a natural soil to destroy cementation and to produce homogenous fabric at the macro level (e.g., Burland, 1990, Aversa et al., 1993, Cuccovillo and Coop, 1993, Kavvadas et al., 1993, Liu and Carter, 1999, Fearon and Coop, 2000).

2.1.1 Effect of preparation techniques on structural properties

Mitchell (1976) suggests that the behaviour of a reconstituted clay specimen will depend on the effort used in the reconstitution process. Rippa and Picarelli (1977) later prepared reconstituted materials by different methods to evaluate the effects of preparation techniques based on the Atterberg limits. Fearon and Coop (2000) have more recently studied the effect of the preparation method on compression and shearing behaviour of the soils. They used three different preparation methods; (i) standard, (ii) reconstituted and, (iii) minced. For the standard soil preparation technique, the soil was stored overnight at a water content slightly lower than the liquid limit, after being broken into small pieces, and was then remoulded by hand mixing. However, for preparation of the reconstituted soil, the natural soil was cut into small pieces, mixed with water to around 1.1-1.5 times the liquid limit, and then allowed to swell overnight; it was later remoulded by a mechanical mixer to form homogenous slurry. The minced samples were firstly prepared at a moisture content through a mincer, and mixed with water slightly lower than the liquid limit and mixed by hand for the liquid limit test (Fearon and Coop, 2000).

Figure 2.1 shows scanning electron micrographs of the natural, reconstituted and minced samples. It can be seen from Figure 2.1 that soils that have been reconstituted using different techniques have different fabric and are likely to have different properties. The clay particles of the reconstituted soil shown in Figure 2.1.b have a more homogenous fabric than that of the natural soil. However the clay particles shown in Figure 2.1.c in the minced sample have a very different fabric from that of the natural and reconstituted soil particles. Accordingly, Fearon and Coop (2000) concluded that intrinsic properties may depend on the preparation techniques used.

Burland (1990) proposed that a reconstituted sample can be prepared from a remoulded natural sample by mixing at water content between the liquid limit (w_L) and 1.5 w_L (without drying the soil), and then consolidating the slurry. The intrinsic properties were first defined and denoted by an asterisk (*) attached to the relevant symbols by Burland (1990). These were said to be independent of the structural effects on natural state and inherent to the soil. Examples of the intrinsic parameters include e^*_{100}, C_C^* and C_S^* for compression, swelling, and the intrinsic critical state angle of shearing resistance. Burland (1990) thought that the influence of soil structure could be measured by making a comparison between its natural behaviour and the intrinsic behaviour under all stress conditions. However, it is now known that reconstituted samples prepared in the laboratory, even if they are completely remoulded, are not completely unstructured, since they have a fabric because of reconstitution and limited bonding (Cotecchia and Chandler, 1997, Chandler, 2000).

2.1.2 Yield for structured geomaterials

It is believed that although structure in natural soils can be developed by many different factors (e.g. depositional conditions, ageing, cementation, chemical and biological factors), structured soils have similar mechanical properties (stress-strain-strength relations). The critical state framework (e.g. Rendulic, 1936, Hvorslev, 1937, Roscoe et al., 1958, Schofield and Wroth, 1968) was the main development in the understanding of the behaviour of remoulded or reconstituted soils in terms of stress-strain-strength relations as a function of shear stress, effective stress and void index (Burland, 1990). However, such a framework may not be always applicable to natural soils. As pointed out by Burland (1990) the critical state framework was developed as a model of the behaviour of reconstituted and remoulded soils, and cannot be used to predict properties such as the peak undrained strength (Su) that primarily depend on the structure of a clay. A more satisfactory framework would include the influence of structure. An important consideration for natural soils is that their yield stress, which is a measure of yield properties, is controlled by structure (Malandraki and Toll, 2001). A number of yielding definitions can be found in the literature. Malandraki and Toll (1995) have therefore attempted to reconcile some of the recent different definitions of yield for bonded geomaterials. They compared the yield definitions and the terminology used to describe yielding behaviour within a framework proposed by Jardine (1992). Based on the results of reconstituted soils, Jardine (1992) proposed an approach defining three yield conditions, in terms of kinematic sub-yield surfaces. In addition, the study of Clayton and Heymann (2002) on the very small strain stiffness levels of three different geomaterials has recently described the linear stress-strain behaviour and degradation rate of stiffness naturally bonded materials. The success of such research requires an understanding of the structured soil to model soil behaviour realistically.

2.2 Fabrics in natural soils

Although a number of definitions of the term 'fabric' may be found in the literature, following the studies of Mitchell (1976) and Terzaghi et al. (1996), it is used herein to mean the geometrical arrangement of soil particles with respect to each other. In contrast, Rowe (1972) defined it not only from an arrangement viewpoint, but also concerning size and shape. He described 'fabric' as the arrangement of particles of different sizes, and 'structure' as the arrangement of particles of similar sizes.

Fabric developed in soils can be seen into two groups as cohesionless and cohesive, based on the predominant size (Santamarina and Cascante, 1998), and shape (Mirghasemi et al., 2002, Clayton et al., 2004, and Santamarina and Cho, 2004) of particles within a soil matrix. Although it is not possible to determine accurately the size of irregular individual particles, for routine descriptions, it is usually possible to make a fairly accurate estimate of the various size classes (FitzPatrick, 1984). The size limits in Table 2.1 developed by different organizations in the United States can be applied for many situations but they may not be universally acceptable. In addition, Table 2.2 shows the recommendations which have been developed in the United Kingdom. Herein it is also worthy to make clear the definition of soil classification system. A number of definitions that can confuse the readers could be found in the literature. In order to prevent these contradictions, following the British Soil Classification System (BS5930:1999), the term 'soils' in the present study are considered under two groups as course (gravels and sands) and fine (clays and silts) based on the percentage by weight. When soils are defined as fine soils then silt and clay is classified by reference to Casagrande's A-line. Silts are said to plot below the A-line and clays above (Clayton et al., 1995).

Particle arrangements encountered in cohesionless soils having angular, subangular, subrounded or rounded particles (Figure 2.2) can be divided into single grain or multigrain (honeycombed) (Mitchell, 1976, Das, 1990). Particle roundness depends on method and distance of transportation of the material from the parent rock (Clayton et al., 1995), and has a significant influence on the behaviour of geomaterials (Mirghasemi, 2002). In the single-grain structural arrangement, individual particles contact each other without any bond or cohesion among them. The arrangement may be loose or dense depending on shape and size distribution of the soil particles and their relative positions (Figure 2.3) (Craig, 1983, Das, 1990). However, it is widely believed that grain interlocking (because of pressure) and grain bonding (because of cement) can often be seen in granular materials (e.g., Cresswell, 1999). In addition, in a honeycomb arrangement, relatively small particles may form small arches having large void ratios. A honeycomb type of arrangement as shown in Figure 2.4 is usually thought to exist in the particle size range of 2 to 74 μm (Das, 1990).

In contrast, cohesive soils, which show low strength, high compressibility and low permeability behaviour, have a platy appearance. Clay minerals in this group can be formed by means of hydration, carbonation and oxidation (McLean and Gribble, 1982), may contact with each other in different ways; as 'edge to edge' (EE), 'edge to face' (EF) or 'face to face' (FF) (Barden and Sides, 1971) because of the electrical interparticle forces (Santamarina and Cho, 2004). Figure 2.5 shows idealized fabrics in clay minerals. EE arrangement result when edges and faces different charges (Santamarina and Cho, 2004). FF arrangements form by van der Waals attractions at high ionic concentration. EF arrangements are an intermediate condition between EE and FF. The density and mechanical properties of the sediment will reflect the type of interparticle associations. For instance, FF aggregations typically form high-density flocs, and EF flocs experience strong Coulombian and van der Waals attraction forces (Palomino and Santamarina, 2005).

In clays, 'flocculation' can be described as an open structured EE or EF association of particles (Figure 2.5.a) and divided in to 'cardhouse' (formed by individual clay particles) and 'bookhouse' (formed by parallel oriented clay particles) (Van Olphen, 1963, Barden and Sides, 1971). On the other hand, another type of arrangement, 'honeycomb', can be described as more open flocculated (Casagrande, 1932) (Figure 2.5.b). This structure can be seen in both fine silt particles and clayey geomaterials. Aylmore and Quirk (1960) proposed 'turbostratic' structure as a type of dispersed fabric, which can be described as a rather close arrangement in which FF association between particles exists (Figure 2.5.c). The 'stack' shown in Figure 2.5.d is an example for domain described as a fabric unit consisting of parallel oriented clay plates (Barden and Sides, 1971).

2.3 Examples of structure in various soils

In practice, it is often difficult to observe clearly the influence of structure on soil behaviour. Evidence of structure in clays, for example, can be shown by comparison of oedometer test results comparing the compression curves obtained from natural and reconstituted samples of the same geomaterial. The curves of natural soils have higher void ratio than these of destructured soils (Casagrande, 1932, Terzaghi, 1941, Skempton, 1944). Burland (1990) has also shown that the relation between void ratio and effective overburden pressure stress for normally consolidated deposits (sedimentation compression line, SCL) is different from the relation between void ratio and effective overburden pressure stress for the reconstituted clay obtained in the laboratory (given by the intrinsic compression line, ICL). The individual sedimentation compression curves presented by Skempton (1970) are assembled by Burland (1990) showing the comparison between the normalized one-dimensional compression curve obtained on reconstituted clays and in situ conditions for many normally consolidated clays. As can be seen from Figure 2.6, the void ratio in natural geomaterials is higher than the void ratio in corresponding reconstituted materials at the given stress level. Burland (1990) states that the reason the SCL is higher than the ICL over the given stress range can be explained by structural effects in the natural soil. The figure also shows that even very young natural clays are influenced by structure, as pointed out by Leroueil and Vaughan (1990).

Leroueil and Vaughan (1990) used Figure 2.7 to compare the compression behaviour of structured and destructured soils in the oedometer test. They showed that one dimensional compression curves of the natural structured specimens move to the right side of the compression curve of the reconstituted specimen of the same geomaterial. They concluded that this results from the structural effects on soil. On the other hand, Aversa (1991) concluded that the influence of structure cannot be seen when the yield stress is higher than bond strength.

As mentioned previously, the influence of structure can be seen not only in clays but also in most natural soils including soft rocks and sands. Naturally bonded sands are found in many places on the earth's crust. As Cresswell and Powrie (2004) have noted, structure in natural sands may result either for the bonding between particles (termed bonded structure) or particle interlocking (a term of fabric structure). Two of the first studies conducted on cemented sands were by Saxena and Lastrico (1978) and Clough et al. (1981). Saxena and Lastrico (1978) reported the findings for naturally and artificially cemented sands based on the properties of the Vincentown Formation on the East Bank of the Delaware River in the US. Clough et al. (1981) suggested that the amount and nature of cement, density, gradation, and confining stress are the governing variables.

Another example of this category can be found in the study of Asghari et al. (2003), which concerns the behaviour of cemented coarse-grained alluvium in a vast area of Tehran City including its suburbs, in Iran. In order to understand the mechanical behaviour of cemented materials, they performed a series of triaxial compression tests, as Saxena and Lastrico (1978) and Clough et al. (1981) had done, on destructured, artificially cemented and uncemented samples. Asghari et al. (2003) conclude that there is an obvious peak in shear stress-strain relationship for all cemented samples that is followed by strain softening, similar to the results demonstrated by Clough et al. (1981). They also conclude that, for a given confining stress, the strain associated with the peak strength decreases with an increase in amount of cementation. It is also worth pointing out that the experiments showed the influence of cementation decrease as the confining stress increases, which is one of the conclusions presented in Malandraki and Toll (1996). In addition, Ismail et al. (2002) studied the influence of cementation on the mechanical properties of the offshore calcareous sand. They studied the effect of cement type on shear behaviour of cemented calcareous soil and on the artificial bonding technique, which will be explained in more detail in the section of 'Artificial Bonding Techniques'.

2.4 Effects of cementation on stress-strain-strength behaviour

The effects of cementation on the stress-strain behaviour of naturally and artificially cemented soils have been investigated by many authors (Saxena and Lastrico, 1978, Clough et al., 1979, 1981, Acar and El-Tahir, 1986, Coop and Atkinson, 1993, Cuccovillo and Coop, 1997, Ismail et al., 2001). In almost all the experimental programs given in the literature, artificially cemented specimens have been used either for a greater understanding of fundamental soil behaviour or to study a ground improvement technique. Cementation reinforces the links between soil particles, and so increases stiffness and peak strength. Figure 2.8 illustrates typical undrained stress-strain triaxial test results. Destructured clay is less stiff and reaches a smaller peak strength at the same strain values. Structure also causes an increase in small strain shear modulus, even at the same void ratio. Structured soils have a higher shear modulus than destructured soils under the same confining stress (Leroueil and Hight, 2003).

A literature review enables some important characteristics of the behaviour of cemented geotechnical materials to be identified. Clough et al. (1981) and Ismail et al. (2002), for example, suggest that the type and amount of cement, density, gradation, and effective confining stress are the governing variables. It has also been suggested that soil behaviour is affected by geological and stress-strain history, temperature and principal stress direction (Gens and Nova, 1993). Schnaid et al. (2001) investigated the stress-strain-strength behaviour of an artificially cemented sandy soil produced through the addition of Portland cement. For cemented sands, they used the unconfined compressive strength as a direct measurement of the degree of cementation (Figure 2.9). The effects of cementation on soil behaviour were similar to those seen in terms of over consolidated clays. Basically, it comprises an initial stiff behaviour followed by plastic deformation as the soil approaches failure. Accordingly, cemented soils have a very stiff behaviour before yielding. The brittle behaviour changes to a ductile soil response as the stress level changes from low to high.

Leroueil and Vaughan (1990) showed that the stress-strain behaviour of soils is dependent on the critical state line of the non-structured remoulded soil, initial state and its position in relation to the yield curve. From the same concept, Figure 2.10 was suggested to describe the idealized behaviour of cemented soils by Coop and Atkinson (1993). According to Coop and Atkinson (1993), the idealized behaviour of cemented soil is divided into three different categorizes, as shown in Figure 2.10. The first (1) occurs on Figure 2.10 when the soil reaches its yield stress during isotropic compression. The second (2) occurs during intermediate stress applications. In this category, it is proposed that bonds are broken during shearing and the strength is mainly controlled by the frictional constituents of the soil. In the third category (3), the soil is sheared at lower confining stresses, comparing to the strength of bond material. A sharp peak in the stress-strain curve occurs at small strain levels and for stresses outside the limit state surface of the equivalent remoulded soil.

2.5 Yield and bounding surfaces for geomaterials

To generate more understanding of the deformation behaviour of soils at small strains (<0.01%), which is dependent on many factors including stress state, stress history, soil fabric, aging and bonding, practitioners must also take account of yielding, plasticity, linear and non-linear ranges as well as anisotropy, fabric and bonding. At large strains soil which is loaded, unloaded and reloaded either isotropically or one-dimensionally deforms plastically during loading if it is normally consolidated. During unloading and reloading, when the soil is overconsolidated, it behaves elastically (Jardine, 1992). At very small strains (0.001%) the behaviour of soil is perfectly elastic; this can be mainly explained with the presence of interparticle bonds. As shown by Leroueil and Vaughan (1990), the presence of interparticle bonds is an important factor in determining their strength and stiffness, which can be seen clearly by comparing the results of one dimensional compression curves on both undisturbed and reconstituted specimens. Based on the yielding points, it has been shown by Burland (1990) and Leroueil and Vaughan (1990) that 1-D compression curves of undisturbed soils extend in to a space of impossible states for reconstituted specimens. Since yielding occurs at small strain levels, the significance of the development of local displacement transducers can be seen. Local strain instrumentation with high accuracy and precision in laboratory testing techniques has shed new light on soil behaviour at small strains (e.g., Burland and Symes, 1982, Jardine et. al, 1984, Clayton and Khatrush, 1986, Heymann et al., 1997, Clayton and Heymann, 2001).

Much research on the pre-failure deformation behaviour of geomaterials studies has been published, providing experimental and theoretical characterization. Intensive studies have been carried out at many centres, in particular in Japan and in the UK. Recognizing these developments, the first International Symposium on Pre-Failure Deformation Properties in Hokkaido, Japan, and the Géotechnique Symposium in Print on Pre-Failure Deformation Behaviour of Geomaterials held at the Institution of Civil Engineers, London brought together many research results on yielding and pre-failure deformation of geomaterials.

2.5.1 Definitions of yield surfaces

Defining the term 'yield' as a discontinuity in stress-strain behaviour under monotonic stress change and defining 'yield of structure' as an irreversible post-yield change in the stiffness and strength of structured clay, Leroueil and Vaughan (1990), in their classical paper, identified three parts of the yield curve where yield occurs in different ways, as shown in Figure 2.11. The parts of the yield curve are;

(i) 'Compression yield' which occurs away from the peak shear strength envelope because of the increase in average and/or shear stress,
(ii) 'Shearing yield' which occurs just before the shear failure,
(iii) 'Swelling yield' which occurs away from the failure envelope in swelling.

Rolo (2003) has suggested that the term 'yielding' describes the end of elastic range (linear or non-linear), which is the classical meaning of yield in elastic-plastic theory. However, these definitions seem to be insufficient for geomaterials. Today, it is widely accepted that the yielding of soils involves multiple kinematic yield surfaces moving within less mobile bounding surfaces (Rolo, 2003). Jardine (1992) showed that sub-yield surfaces are mobile and may be re-positioned and modified by moving the current stress point. Conversely, Malandraki and Toll (2001) suggested that the yield surface is kinematic in the meaning of expandable and shrinkable, but it is not a movable surface. Malandraki and Toll (1996) defined yield of artificially bonded soil from the changes in the tangential stiffness (E_{tan}) with respect to axial strain on logarithmic scale. Using external strain measurements, they identified first yield at the point where the first change in stiffness occurred and a second yield at the point where a major drop in stiffness initiated, which was much clearer and easier to define for them. Malandraki and Toll (1996) also presented a comparative study between the yield results from Maccarini (1987) and Bressani (1990) and their results; however Maccarini and Bressani used local strain measurements whereas Malandraki and Toll used external strain measurements.

Jardine (1992) described clay stress-strain behaviour by identifying three different surfaces as can be seen from the Figure 2.12; an outer bounding surface and three characteristics zones, soil deformation in normalised stress space may be summarised as follow;

Within the *Y_1 surface*: soil exhibits true linear elastic behaviour. It is expected that the zone enclosed by Y_1 might be in highly cemented geomaterials. The soil particle contacts are believed to remain practically unchanged, with the whole soil matrix behaving almost as an arrangement of elastic particles connected by elastic contacts and without relative movements occurring among them. As shown by Jardine (1995) and Cuccovillo and Coop (1997), for lightly cemented sediments this limit is not likely to exceed the $6\text{x}10^{-5}$ strain level, and for reconstituted soils it can have a maximum level of around $2\text{x}10^{-5}$. It is therefore clear that sensitive small strain measurement gauges are required to assess the limits of this region.

The *Y_2 surface*: represents the limit of non-linear recoverable (capable of being recovered or regained) behaviour. Jardine (1992, 1995) and Smith et al., (1992) showed that for clays the hysteresis loops developed in this zone dissipate energy but still close. The energy dissipated is believed to be the result of fretting at the inter particle contacts and small scale local yielding. On the other hand, for sands Kuwano (1999) showed that hysteresis loops developed in zone 2 fail to close. Although the Y_2 boundary differs for various soils, Smith et al., (1992) indicated it to be around $3\text{x}10^{-4}$ strain for uncemented natural soils, $7\text{x}10^{-4}$ for cemented soils, and $5\text{x}10^{-5}$ for reconstituted soils. Similar to the soil's behaviour in zone 1, the behaviour of soils in the zone 2 is also dependent on recent stress path direction ($\Delta q/\Delta p'$), since the behaviour of soils is not isotropic.

Within the *Y_3 surface*; representing complete breaking of structure among soil particles the key feature of the soil deformation is the development of irrecoverable strains. It is thought that the soil particles start to slide relative to one another in zone 3 enclosing by Y_3 surface. The stress level at the Y_3 surface is identified as a point where sharp change in the stress-strain response occurs, as in the classical definition of oedometer tests yield points (Rolo, 2003).

The *Bounding Surface (BS)* provides a boundary for an element of soil undergoing plastic deformation. It separates possible states from impossible states for all possible void ratio-effective stress states. The effective stress-void ratio principles for reconstituted soils were described by Roscoe et al. (1958) of Cambridge University for soils sheared under drained or undrained conditions. They proposed a unique relationship between effective stress and void ratio within the concept of the 'critical state', which provides a basic approach to the understanding of reconstituted soil behaviour. However according to observations by Gens (1982, 1985), reconstituted clays that are normally consolidated along different consolidation paths K_o ratios do not necessarily show unique relationships. From the studies of Gens (1982, 1985), Jardine (1992) developed the concept of a *Local Boundary Surface* (LBS). Behaviour within the limits of LBS enclosing the Y_1 and Y_2 surfaces is inelastic and non-linear, and the position of the LBS is dependent on the previous consolidation stress history (Leroueil and Vaughan, 1990).

Smith (1992) investigated the influence of structure on the shape and size of the state boundary surfaces. He showed that normalization by the equivalent pressure (p_e') providing a quantitative measure of sensitivity of the soil is a useful approach for investigating the shapes of the LBS and SBS of clay. In his thesis, Smith (1992) identified the SBS for the Berthierville, Bothkennar and Queenborough Clays presented in Figure 2.13. In terms of normalized stress, the size of the boundary surface increases with increasing structure.

It is helpful to refer in this research to the study of Clayton and Heymann (2001) on the very small strain stiffness levels of three different geomaterials (Bothkennar clay representing soft clay, London clay representing very stiff clay, and Chalk representing weak rock), which has recently described the linear stress-strain behaviour and degradation rate of stiffness of naturally bonded materials. They showed in their study, which was conducted to make judgement on the stiffness at small strains obtained using field geophysical techniques, that the stress-strain behaviour appeared to be linear for all three materials at very small strain levels (see the Figure 2.14). They noted that the extent of this linear behaviour was surprisingly similar for all three materials, despite the maximum stiffness (E_{max}) ranging over two orders of magnitude. The examples presented in their study indicate that structure gives congruent behaviour although the materials are very different.

The effects of cementation on the stresses at yield are shown in Figure 2.17. As can be seen from Figure 2.15, the shape of yield locus changes and it expands as cement content increases. The addition of the cement increases the apparent preconsolidation pressure and it shifts the normal compression line towards to right. Huang and Airey (1998) showed that cementation increases the stiffness, peak strength, size and shape of the yield locus for a wide range of confining stresses.

2.6 Recent studies on artificial bonding techniques

Investigations into the influence of structure (more specifically 'bonding') and bonded soils as well as weak rocks provide one of the major challenges for geotechnical engineering. According to Gens and Nova (1993), from a mechanical viewpoint, these types of geomaterials are in an intermediate area between rock and soil mechanics. It has often been noted that there is a lack of framework to integrate the behaviour of such materials in a consistent way. In the past 30 years, many authors have made significant researches in this field presenting various behaviours for such type of soils (e.g., Saxena and Lastrico, 1978, Clough et al. 1979, 1981, Acar and El-Tahir, 1986, Maccarini, 1987, Bressani, 1990, Leroueil and Vaughan, 1990, Gens and Nova, 1993, Coop and Atkinson, 1993, Malandraki, 1994, Cuccovillo and Coop, 1997, Liu and Carter, 1999, 2000, Ismail et al., 2000, Malandraki and Toll, 2001). Studies of structured soils have involved laboratory testing either of artificially bonded soils or of high quality samples. In the light of the studies of Clayton et al. (1992), and Hight et al. (1992), which show that even high quality sampling techniques may cause a significant loss in structure, the first is favoured as it reduces soil variability and allows qualitative comparison.

As shown by Ismail et al. (2002), shearing response is cement-dependent (see Figure 2.16). They carried out tests on cemented samples in a standard triaxial apparatus at different level of density and cementation, maintaining consistency of unconfined compressive strength and sample density among the different cementing agents, which were calcite, Portland cement, and gypsum. Despite having the same unconfined compressive strength and density, the effective stress paths and post-yield responses were different, because of the different volumetric response upon shearing. As can be seen from Figure 2.18, samples prepared using Portland cement showed ductile yield and strong dilation afterwards, samples cemented by calcite and gypsum exhibited brittle yield, usually followed by contractive behaviour.

2.6.1 Portland cement

Acar and El-Tahir (1986), for example, conducted a series of torsional resonant column tests to study the influence of artificial cementation on the low strain dynamic properties of Monterey Sand. The artificially cemented specimens they used were prepared using 1, 2, and 4 per cent Portland cement by weight. The experiment results demonstrate that maximum shear modulus increased and damping ratio decreased with an increase in cement content. Similar results using Portland cement have recently been also produced by Ismail et al. (2002b). Acar and El-Tahir (1986) pointed out the importance of such results for geotechnical earthquake engineering, including ground response analysis and liquefaction potential.

A similar approach was used by Rotta et al. (2003). Rotta et al. (2003) simulated the formation of sedimentary deposits in which cement bonding occurs after burial and under geostatic stresses by using variable Portland cement contents. In their study, after consolidating the samples to the uncemented normal compression line, the specimens were allowed to cure for 48 hours before testing. In conclusion, they pointed out the importance of the degree of cementation and also the void ratio during the formation of cement bonds, the experimental results they presented demonstrated that the variation in yield stress with void ratio and cement content is dependent on the independent of the stress history.

2.6.2 Fired kaolin

Maccarini (1987), Bressani (1990) and Malandraki (1994) used an artificially weakly bonded material to investigate the influence of density and bond strength on yield. A mixture of sand made from kaolin and quartz sand was fired at 1100 °C. The artificial sand was then mixed with a small amount of kaolin slurry and air-dried. The mixture was then fired, and the kaolin slurry formed a bond between the sand particles. The strength of bond between the sand particles was varied depending on the time of firing, temperature and density.

2.6.3 Calcite in situ precipitation system

Ismail et al. (2000) identified that gypsum presents an alternative model of carbonate bonding; but the percentage of cement required to achieve a given bond strength is higher than for a natural cementing process. Therefore they used the 'CIPS' (calcite *in situ* precipitation system) process (Kucharski et al., 1996), which allows calcite cement to be deposited through the soil particles while the soil is under a confining pressure. CIPS is a chemical material consisting of two liquid components that as combined trigger a chemical reaction resulting in the precipitation of calcite crystals onto surfaces of the soil particles. CIPS seems to be highly effective way that bonds between soil particles, however the composition of its components still remains confidential.

2.6.4 Electrokinetics

Since the exploration of gas and oil reservoirs at offshore sites has led to a number of platforms being built on calcareous soils, the engineering properties of offshore calcareous soils have been investigated by researchers extensively in recent years including Ismail et al. (2000, 2002a, 2002b), Micic et al. (2003), Shang et al. (2004), and

Mohamedelhassan et al. (2005). The erection of foundations for offshore constructions can disturb the natural structure of the soil, including cementation. This event can lead the low shear strength resulting in low load carrying capacity. Originating from the problem facing offshore foundation engineering, Micic et al. (2003) focused on using 'electrokinetics' to enhance the load-carrying capacity of skirted foundations embedded in soft marine deposit. When a direct current (dc) electric field is applied in clayey soil, three different electrokinetic processes are observed, namely; (i) 'electroosmosis', which is mainly attributed to the fact that the technique facilitates the movement of pore water and dissolved ions, (ii) 'electrophoresis', which is the tendency or movement of negatively charged soil particles toward the anode, (iii) 'ionic migration', which is the movement of ions in the pore water toward electrodes. The experimental study carried out in a modal tank by Micic et al. (2003) showed that load-carrying capacity of the modal skirted foundation increased three times using the electrokinetics. Similarly, Shang et al. (2004) have studied on the calcareous soil and steel foundation. Shang et al. (2004) performed a series of experiments to define the effects of two chemical agents ($CaCl_2$ and $Al_2 (SO_4)_3\ 18H_2O$) used in 'electrochemical' stabilization tests, which were selected because of having high water solubility at 25^oC. They have reported the significant increases in undrained shear strength and effective cohesion. Mohamedelhassan et al. (2005) conducted an experimental study on the electrochemical cementation of calcareous sand for offshore foundations using $CaCl_2$ granules as the cementing agent. They assessed the results of treatment by a pullout resistance test, which showed that the pullout resistance of the foundation modal after electrochemical treatment increased as compared with that of the control test.

2.6.5 Bacteria

The understanding of soil behaviour in the last 300 years has centred on mechanical principles, geological processes, mineralogy and colloidal chemistry. However, recent researches in biology and earth science have enabled significant advances in understanding the crucial involvement of microorganisms in the evolution of the earth, their participation in facilitating and mediating most geochemical reactions, their ubiquitous presence in near surface soils and rocks (Mitchell and Santamarina, 2005). Therefore, finally it also seems to be worth bearing in mind that soil bacteria may also be considered as a bonding agent.

Although not presently applied to bond soil particles together, the use of bacteria, particularly EPS (extracellular polymeric substance) producing ones, has been highlighted by Çabalar and Çanakcı (2005). In the light of the previous studies (e.g., Yang et al., 1994, Perkins et al., 2000, Ramachandran et al., 2001, Kim et al., 2004), showing various applications of bacterial products in civil engineering, they first have pointed out the applicability of biotechnologies in geotechnical engineering, and then focused on various aspect of xanthan gum, which is a microbial polysaccharide produced by *Xanthomonas campestris*. The experimental work they performed was directed mainly towards an investigation of influence of biopolymeric xanthan gum and Leighton Buzzard Sand used in combination as blending materials by weight on water demand. 1, 3 and 5 per cent xanthan gum solutions were studied by the variation of 7, 28 and 50 days time intervals using direct shear box testing apparatus. The mixtures have shown an overall increase in peak shear strength values at failure as xanthan gum percentage increases, regardless of time. On the other hand, as pointed out by Çabalar and Çanakcı (2005), Mitchell and Santamarina (2005), Çabalar (2006, 2007), and DeJong et al. (2006), collaborative studies between soil mechanics and microbiology may lead to the developments on ground improvement strategies. Accordingly, the requirements of a more detailed explained laboratory results and field studies on deformation behaviour of soils are clearly seen.

2.7 Stick-slip behaviour in granular materials

The mechanical behaviour of granular material may be attributed to the fabric and the associated pore spaces. When the arrangement of particles changes the bulk of the material responds differently. The stick-slip phenomenon occurs in granular materials when particles slide, slip with respect to each other. In some cases, sudden releases of stress followed by gradual increase may be observed during the deformation of granular materials (Alshibli and Roussel, 2006).

2.7.1 Physics considerations

Stick-slip motion in granular media is one of the problems in physics with a huge practical significance. In recent years, there has been a hightened interest in the stick-slip response of assembles of particles (e.g., Feder and Feder, 1991, Cates et al., 1998, Albert et al, 2000, 2001, Cain et al, 2001). These studies have been performed on various materials, including granular materials, boundary lubricants and foams. Surprisingly, all these materials have a number of common properties during stick-slip motion, including the shapes of the stick-slip friction spikes, pre- and postcursors to slip, and the existence of a critical velocity for the disappearance of stick-slip motion. A commonly accepted view is that shear induced 'fluidization' and pressure induced 'solidification' accompany the stick-slip mechanism in a discrete particles system (Figure 2.17). During sticking, the bulk of material behaves like a solid; the particles at the interface are closely packed and have a high shear strength. Periodically however, the material dilates and is like fluid, allowing the slider to slip (Cain et al., 2001).

Granular materials are composed of many solid particles that interact through contact forces. The propagation of stress in such a material is a complicated issue which was the subject of an extensive study. An applied load to granular material does not distribute uniformly through the contact points. An applied external load results in the development of an internal structure resisting the stress. An important amount of the load is supported by this chain developed. Above a certain threshold of stress, this system may jam depending on the magnitude and direction of the load (Cates et al., 1998). As an example, Figure 2.18 presents a matrix of hard particles under shear. The jammed state occurs when the particles form force chains along the load direction.

There are two basic modes of sliding between two surfaces; stable sliding at a nearly constant velocity, and unstable stick-slip sliding, in which the surface remains locked for a while and then moves suddenly. For example, a solid object being pulled slowly through a granular medium is resisted by local jamming, but if the applied force exceeds a certain threshold corresponding to the strength of jammed state, the solid object moves. These successive breakdowns result in fluctuations in the force value at low velocities, which gives insight the strength of successive jammed states (Cain et al., 2001).

Here in this study, it also would be useful to refer a typical experimental technique (Figure 2.19) used in tribology, which is described as a science of surfaces in relative motion. Based on the assumption of initially rested of the system, the spring has its natural length at that moment. When the base, which is connected to the spring, starts to move, the spring stretches, force increases until it reaches the threshold level corresponding to the static frictional force, the block starts to move. The slider accelerates to catch the base because of the inertia. If velocity of the base is small, force decreases down to the backward threshold force and the slider stops. Then the mechanism repeats so as to the stick-slip motion takes place (Figure 2.20). On the other hand, as the velocity of the base is large, smooth sliding occurs. A number of experiments show that the response depends on the k_{spring} and v_{spring}. The smooth sliding is observed as the spring is stiff and/or velocity is large, otherwise, the stick-slip motion is observed. During stick phase, elastic energy is stored into the system by driving; during slip phase, this elastic energy is converted to kinetic energy, which eventually must be released as heat (Braun and Naumovets, 2006).

2.7.2 Geotechnical considerations

Granular materials have an efficiency to simulate the earth behaviour in a realistic way. For instance, using microscopic modelling, researchers tried to reproduce the deformation in earth crust and the magnitude distribution that characterizes any seismic area (Bak et al., 1987). Earthquakes are an important part of the relaxation mechanism of the crust which is submitted to inhomogeneous increasing stresses accumulating at continental-plate borders (Sornette and Sornette, 1989). Here, it is also worth referring to the pioneering experimental study by Brace and Byerlee (1966) on geomaterial to mimic local mechanics in a fault with granite surfaces. They note that stick-slip instabilities in laboratory friction may simulate earthquake movement. Since the stick-slip motion on a natural fault is the source of earthquakes, stability of sliding is an important problem to study.

Burridge and Knopoff (1967) conducted a study presenting a laboratory work and a numerical modelling to explore the role of friction along a fault as a factor in the earthquake mechanism. Similar to the 'critical threshold' energy already mentioned, their study demonstrated that small shocks are necessary to the loading of potential energy into the focal structure; a large part, but not all, of the stored potential energy is later suddenly released in a major shock, at the end of a period of loading energy into the system. This approach accounts for all possible earthquake scenarios that could affect the site and results in hazard represented by ground motion parameters at reference ground conditions, such as peak ground acceleration and spectral accelerations (Erdik et al., 2004).

On the other hand, stick-slip behaviour is seldom observed in soil mechanics that uses uniaxial compression often. Nevertheless, it has been reported by Adjemian and Evesque (2004) that Kim (1995) and Duchesne (1998) have observed several drops in the q-ε curve with cohesionless glass beads, with rather strong dilation effect, using the same kind of glass spheres. Both have concluded that the motion is periodic and might be because of some periodic arrangement of the grains.

More recently, Alshibli and Roussel (2006) have presented the results of a series of experiments showing load oscillations in granular materials. They tested cylindrical specimens under axisymmetric triaxial loading condition at various confining pressures. They concluded that (i) the shape and gradation of particles is the paramount factor influencing the load oscillation (ii) load oscillation is attributed to the stick-slip behaviour between glass beads as they form chains of particles to support the applied load. In the light of the few test results given in the paper, the first conclusion seems to be fairly reasonable. Although, there is a considerable scatter among the data obtained, it has been realized that, since all the particles are smooth and spherical, a generalization appears misleading as to the real influences of the particle shape on the load oscillation. As an example, from the studies by Vermeulen (2001) and Theron (2004), it does not seem to be reasonable to expect a similar stick-slip mechanism for platy particles, like mica.

2.8 Particle characteristics

Particle size and shape of soils grains depend on composition, grain formation, transportation, and depositional environments (Santamarina and Cho, 2004). During the formation of soil from bedrock, which may be either physical (mechanical loading, mechanical unloading, thermal loading, wetting and drying, crystallization, pneumatic loading (Blyth and Freitas, 1984)) or chemical (solution, oxidation, reduction, hydration, hydrolysis, leaching, cation exchange (Blyth and Freitas, 1984)), the size and shape of particles change (see Figure 2.2). The resultant soil particles at the end of physical processes may have the same composition as that of the parent rock. However, the chemical process results in changes in the mineral form of the parent rock by the action of water, oxygen and carbon dioxide (Craig, 1983). For example clay is an aggregate of microscopic and submicroscopic particles originating from the chemical decomposition of rock, which consists of silica tetrahedra and alumina octahedra, and built up in layers, and are great importance of geotechnical engineering (Terzaghi et al., 1996). They contain large amount of water trapped between some of the silicate sheets types connecting to each other by layers of cations. The reason why these sheet structures produce very soft minerals can be explained by these weak bonds between cation layers (Harvey, 1982).

Saturated low permeability soil deposits, when loaded with a surcharge, may result in settlements taking place over a period of time. As an example, structures built on soft clay deposits may deform at a very slow rate. Terzaghi seems to have been one of the first engineers to make a detailed investigation to understand the mechanics controlling the compressibility of clays. Today, there is an enormous literature on consolidation (e.g., Terzaghi, 1925, Gilboy, 1928, Taylor, 1942, Bjerrum, 1967, Crawford, 1986) beginning with his initial work at Boğaziçi University (then Robert College). These pioneering contributions in the 20th Century have centred on mechanical principles and demonstrated the essential role of geology in identifying subsurface geomaterials and many aspect of their behaviour. Terzaghi (1925) postulated that the compressibility of cohesionless material is governed by the grain size, uniformity, volume of voids and mica content. Gilboy (1928) investigated the influence of mica content on the compressibility of sand, and concluded that an increase in mica content resulted in an increase in the void ratio of the uncompressed material as well as an increase in compressibility. Redlich et al. (1929) also concluded that compressibility of dry mixtures of sand and ground mica increases as mica content increases. Olson and Mesri (1970) investigated the importance of mechanical and physico-chemical effects on the compressibility of mica, kaolinite, illite and smectite.

It is now thought that some geomaterials may display characteristics of both sand and clay. The observations, first made by Gilboy (1928), that any system of analysis or classification of soil which neglects the presence and effect of the flat-grained constituents will be incomplete and erroneous. Gilboy (1928) investigated the influence of mica content on the compressibility of sand, and concluded that an increase in mica content resulted in an increase in the void ratio of the uncompressed material as well as an increase in compressibility. A recent experimental study by Theron (2004) was conducted on an artificial soil that consists of coarse rotund particles containing various amounts of platy fines. The two recent investigations (Vermeulen, 2001, Theron, 2004) into the behaviour of gold mine tailings have showed that the behaviour of some gold tailings is similar to clay (low strength, high compressibility and low permeability) despite being composed of sand size particles having no colloidally-active minerals.

In the light of these two studies, the study by Clayton et al. (2004) points to the importance of the influence of particle shape on the behaviour of such geomaterials using experimental data. This has significant implications for more understanding of both the compressibility and strength of the materials. The laboratory and field data for South African gold tailings show that the fine fraction (termed 'slimes') in tailings is much more compressible than the coarse fraction (the ‘sands’). They used scanning electron microscopy to demonstrate the linkage between particle size and particle shape. The authors show consistent relationships between particle size and particle shape using triaxial tests on mixtures of fine-grained mica and coarse sand.

The behaviour of micaceous sands was studied in connection with flow slides that occurred during construction of river training for the Jamura Bridge in Bangladesh (Hight et al., 1999), and Merriespruit gold tailings dam in South Africa which failed in such a catastrophic fashion in 1994 (Fourie and Papageorgiou, 2001, and Fourie, Blight and Papageorgiou, 2001). As an example, Figure 2.21 a gives a comparison between the behaviour of a loose clean sand and the same sand with 1% of mica by weight added, which was tested in simple shear apparatus. The clean sand shows a ductile behaviour having a tendency to dilate at large strains. Conversely, the sand with 1% mica is brittle and suppresses the tendency to dilate. Figure 2.21 b shows a series of experiments on samples having various mica contents. Figure 2.21 c demonstrates that the effect of mica on the behaviour of sand is not a reflection of its effect on void ratio. The two samples with 5% mica behave in a similar way, in spite of the differences in void ratio. The clean sand and sand with 5% mica at a similar void ratio behave in a quite different way (Hight et al., 2003).

2.9 Tests of sands with different viscosity pore fluids

Viscosity is one of the important physical properties of many fluid products, and can be described as a fluid’s resistance to flow. Fluids resist the relative motion of immersed objects in them as well as to the layers motion with different velocities within them. It is introduced by considering a gas or liquid flowing with a velocity which varies across the specimen, such as a liquid or gas flows through a tube with a velocity which is greatest in the axis of the tube and decreases at the wall. Viscosity varies with temperature. In general, most liquids become less viscous as the temperature increases. For example, honey and syrup can flow more easily when heated. As the temperature increases the speed of the molecules in a liquid increases, then the time they spend in contact with each other decreases. Viscosity is normally independent of pressure, although under extreme pressure the viscosity of liquids increases (Wright, 1977).

The author's review of tests of sands with different viscosity pore fluids in the soil mechanics is based on the readily available references that are cited. A critical review of the literature enables one to identify some important characteristics of the behavior of geomaterials with different viscosity pore fluids. The author therefore considers it is to be further noted that the studies from various approaches such as centrifuge tests (i.e., Zeng et al, 1998) and soil dynamics (Ellis et al, 2000) as well as fundamental soil behaviour (i.e., Ratnaweera and Meegoda, 2006).

In centrifuge model tests, it is common to use pore fluids having high viscosity to unify time-scaling factors for dynamic and consolidation events. The use of a pore fluid with higher viscosity reduces the permeability of a geomaterial, making it possible to achieve similar time scale. Zeng et al. (1998) presented a series of permeability test results on two types of sands. They found that using a glycerine-water mixture as the pore fluid has no significant effect on the strength and stress-strain relationship of Ottawa sand No. 40. Coefficients of permeability are inversely proportional to the viscosity. On the other hand, at small hydraulic gradients, it was observed that the highly viscous fluids can lead the clogging of flow through the pores.

Pore fluid viscosity is likely to make some contribution to the dynamic behaviour of sands, such as stiffness, damping, and liquefaction characteristics. As particles move with respect to each other, local viscous loss is likely to occur mainly close to the particle contacts (Ellis et al., 2000). Biot (1956a, b) modelled a mechanism of viscous damping associated with macroscopic fluid flow using linear elasticity and Darcy's law. In addition to the macroscopic viscous damping mechanism proposed by Biot, viscous losses due to local motion of liquid near the intergranular contacts need to be considered as another energy dissipation process. The local fluid flow occurs as the local excess fluid pressures created by particle movements attempt to equilibrate. This microscopic mechanism was described as 'squeeze film motion' by Stoll (1989). According to him, this mechanism is significant at frequencies lower than the critical frequency described by Biot's model. Wilson (1988) studied the effect of pore fluid viscosity on damping in sand, and showed that the change in damping between oil and water-saturated samples increased with applied shear strain. It was interpreted that pore fluid would be forced to flow around the moving soil particles by shear deformation.

Furthermore, Ratnaweera and Meegoda (2006) have recently published an interesting study presenting a series of unconfined compression tests on fine-grained soils contaminated with varying amounts of chemicals. Their observations, which showed a decrease in shear strength and stress-strain behaviour due to the presence of the additives, were attributed to changes in dielectric constant and pore fluid viscosity caused by the additives. Consolidated drained triaxial test results on a granular soil showed a similar behaviour. This is attributed to mechanical interactions at particle contacts, caused by the enhanced lubrication offered by the more viscous, compared to water, pore fluid. For fine-grained soils, the reduction in shear strength is attributed to physicochemical effects caused by a reduction in dielectric constant, and mechanical interactions caused by higher pore fluid viscosity.

Table 2.1 Soil-separate-size limits (Das, 1990)

Organization name	Grain size (mm)			
	Gravel	Sand	Silt	Clay
MIT[1]	>2	2 to 0.006	0.06 to 0.002	<0.002
USDA[2]	>2	2 to 0.05	0.05 to 0.002	<0.002
AASHTO[3]	76.2 to 2	2 to 0.075	0.075 to 0.002	<0.002
USCS[4]	76.2 to 4.75	4.75 to 0.075	Fines (i.e., silts and clays) < 0.075	

[1] Massachusetts Institute of Technology, [2] United State Department of Agriculture, [3] American Association of State Highway and Transportation Officials, [4] Unified Soil Classification System (U.S. Army Corps of Engineers, U.S. Bureau of Reclamation).

Table 2.2 Identification and description of soils (BS 5930)

Soil group	Density/compactness/strength: Term	Density/compactness/strength: Field test	Discontinuities		Bedding		Colour	Composite soil types (mixtures of basic soil types)		Particle shape	Particle size	PRINCIPAL SOIL TYPE
Very coarse soils	Loose	By inspection of voids and particle packing	Scale of spacing of discontinuities		Scale of bedding thickness			For mixtures involving very coarse soils, see 41.4.4.3		Angular	— 200	BOULDERS
	Dense		Term	Mean spacing mm	Term	Mean thickness mm	Red Orange	Term	Approx. %[c] secondary	Sub angular	— 60	COBBLES
Coarse soils (over about 65% sand and gravel sizes)	Borehole with SPT N-value		Very widely	Over 2 000	Very thickly bedded	Over 2 000	Yellow Brown Green	Slightly (sandy[d])	< 5	Sub rounded	Coarse — 20	GRAVEL
	Very loose	0 - 4	Widely	2 000 to 600	Thickly bedded	2 000 to 600	Blue White	(sandy[d])	5 to 20[b]	Rounded	Medium	
	Loose	4 - 10	Medium	600 to 200	Medium bedded	600 to 200	Cream			Flat	— 6	
	Medium dense	10 - 30	Closely	200 to 60	Thinly bedded	200 to 60	Grey Black			Tabular	Fine — 2	
	Dense	30 - 50	Very closely	60 to 20	Very thinly bedded	60 to 20	etc.	Very (sandy[d])	> 20[b]	Elongated	Coarse	SAND
	Very dense	> 50	Extremely closely	Under 20	Thickly laminated	20 to 6				Minor constituent type	— 0.6	
	Slightly cemented	Visual examination: pick removes soil in lumps which can be abraded	Fissured	Breaks into blocks along unpolished discontinuities	Thinly laminated	Under 6		SAND AND GRAVEL	about 50[b]	Calcareous, shelly, glauconitic, micaceous etc. using terms such as	Medium — 0.2 Fine — 0.06	
Fine soils (over about 35% silt and clay sizes)	Un-compact	Easily moulded or crushed in the fingers	Sheared	Breaks into blocks along polished discontinuities	Inter-bedded	Alternating layers of different types Prequalified by thickness term if in equal proportions. Otherwise thickness of and spacing between subordinate layers defined	Light Dark Mottled	Term	Approx. %[c] secondary	Slightly calcareous, calcareous, very calcareous.	Coarse — 0.02 Medium	SILT
	Compact	Can be moulded or crushed by strong pressure in the fingers	Spacing terms also used for distance between partings, isolated beds or laminae, dessication cracks, rootlets etc.					Slightly (sandy[e])	< 35		— 0.006 Fine — 0.002	CLAY/ SILT
	Very soft 0 - 20	Finger easily pushed in up to 25 mm			Inter-laminated					% defined on a site or material specific basis or subjective		
	Soft 20 - 40	Finger pushed in up to 10 mm										
	Firm 40 - 75	Thumb makes impression easily						(sandy[e])	35 to 65[e]			CLAY
	Stiff 75 - 150	Can be indented slightly by thumb										
	Very stiff 150 - 300	Can be indented by thumb nail						Very (sandy[f])	> 65[e]			
	Hard (or very weak mudstone) Cu > 300 kPa	Can be scratched by thumbnail see 41.2.2										

Organic soils	Firm	Fibres already compressed together	Fibrous	Plant remains recognizable and retains some strength	Transported mixtures	Colour	
					Slightly organic clay or silt Slightly organic sand	Grey as mineral	Contains finely divided or discrete particles of organic matter, often with distinctive smell, may oxidize rapidly. Describe as for inorganic soils using terminology above.
	Spongy	Very compressible and open structure	Pseudo-fibrous	Plant remains recognizable, strength lost	Organic clay or silt Organic sand	Dark grey Dark grey	
					Very organic clay or silt Very organic sand	Black Black	
					Accumulated in situ	Predominantly plant remains, usually dark brown or black in colour, distinctive smell, low bulk density. Can contain disseminated or discrete mineral soils	
	Plastic	Can be moulded in hand and smears fingers	Amorphous	Recognizable plant remains absent	Peat		

Table 2.2 Identification and description of soils (continued)

PRINCIPAL SOIL TYPE	Visual identification	Minor constituents	Stratum name	Example descriptions
BOULDERS	Only seen complete in pits or exposures	Shell fragments, pockets of peat, gypsum crystals, flint gravel, fragments of brick, rootlets, plastic bags etc. using terms such as; with rare with occasional with abundant/frequent/ numerous % defined on a site or material specific basis or subjective	RECENT DEPOSITS, ALLUVIUM, WEATHERED BRACKLESHAM CLAY, LIAS CLAY, EMBANKMENT FILL TOPSOIL, MADE GROUND OR GLACIAL DEPOSITS ? etc.	Loose brown very sandy sub-angular fine to coarse flint GRAVEL with small pockets (up to 30 mm) of clay. (TERRACE GRAVELS) Medium dense light brown gravelly clayey fine SAND. Gravel is fine (GLACIAL DEPOSITS) Stiff very closely sheared orange mottled brown slightly gravelly CLAY. Gravel is fine and medium of rounded quartzite. (REWORKED WEATHERED LONDON CLAY) Firm thinly laminated grey CLAY with closely spaced thick laminae of sand. (ALLUVIUM) Plastic brown clayey amorphous PEAT. (RECENT DEPOSITS)
COBBLES	Often difficult to recover whole from boreholes			
GRAVEL	Easily visible to naked eye; particle shape can be described; grading can be described.			
SAND	Visible to naked eye; no cohesion when dry; grading can be described.			
SILT	Only coarse silt visible with hand lens; exhibits little plasticity and marked dilatancy; slightly granular or silky to the touch; disintegrates in water; lumps dry quickly; possesses cohesion but can be powdered easily between fingers			
CLAY/ SILT	Intermediate in behaviour between clay and silt. Slightly dilatant			
CLAY	Dry lumps can be broken but not powdered between the fingers; they also disintegrate under water but more slowly than silt; smooth to the touch; exhibits plasticity but no dilatancy; sticks to the fingers and dries slowly; shrinks appreciably on drying usually showing cracks.			

NOTES

[a] Or described as coarse soil depending on mass behaviour

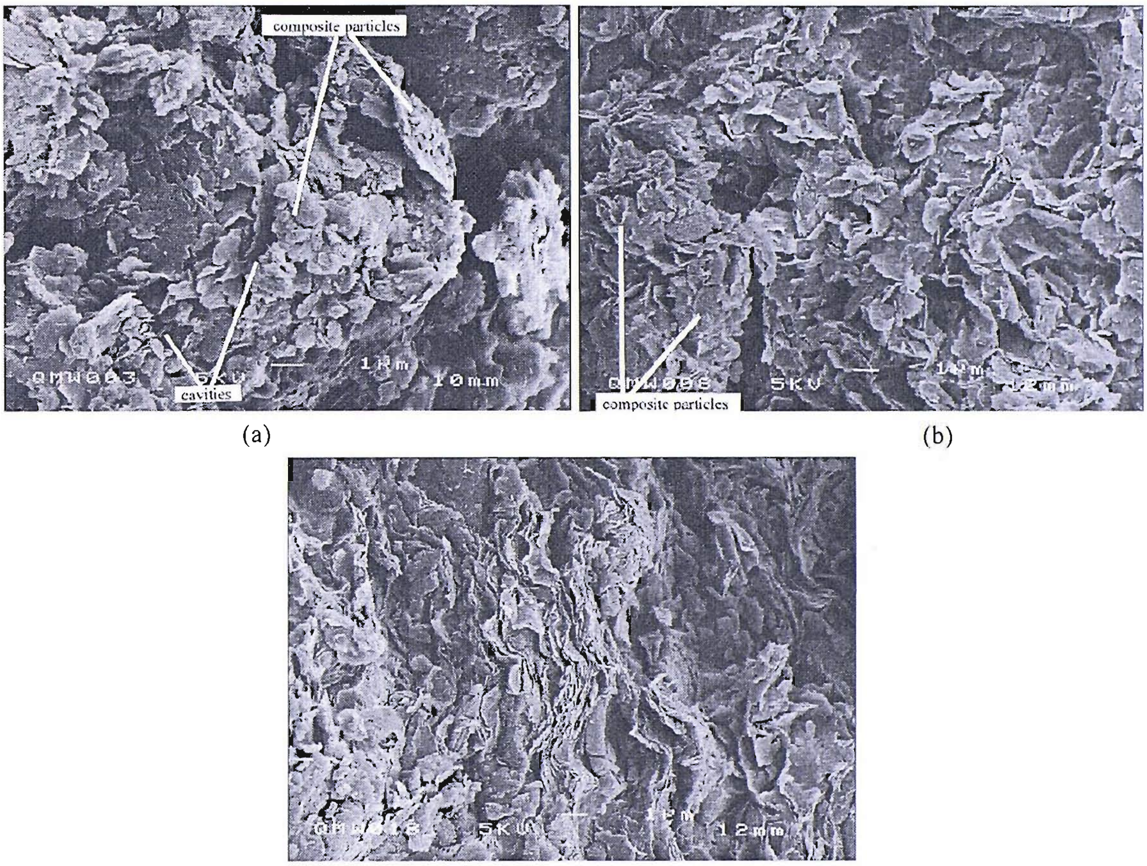
[b] Or described as fine soil depending on mass behaviour

[c] % coarse or fine soil type assessed excluding cobbles and boulders

[d] Gravelly or sandy and/or silty or clayey

[e] Gravelly and/or sandy

[f] Gravelly or sandy



(a)

(b)

(c)

Figure 2.1 Scanning electron micrograph of the fine-grained clay matrix of the (a) natural soil, (b) reconstituted soil, (c) minced soil (Fearon and Coop, 2000).

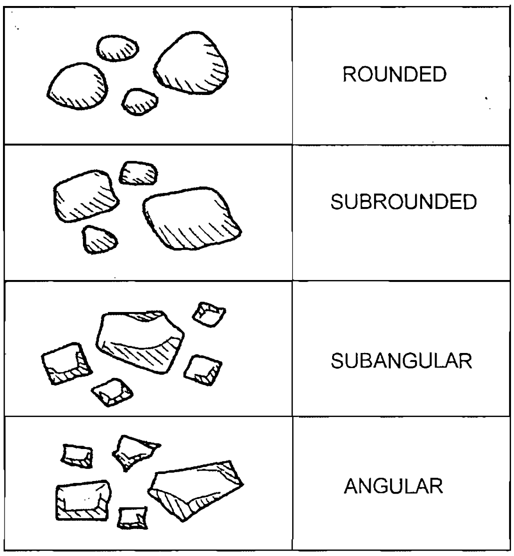


Figure 2.2 Angularity of coarse soil particles (Clayton et al., 1995)

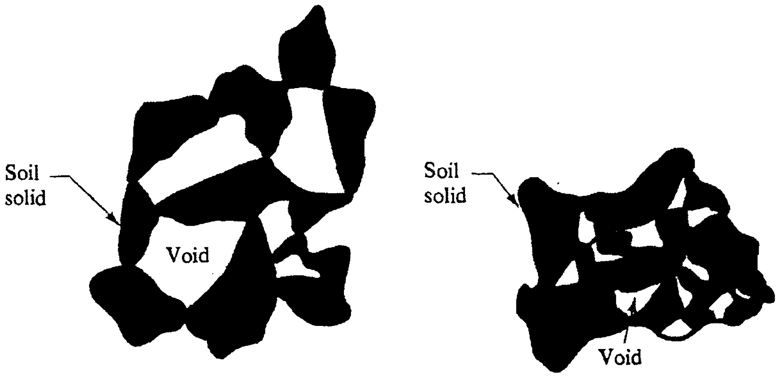


Figure 2.3 Singlegrain structures (a) loose (b) dense (Das, 1990)

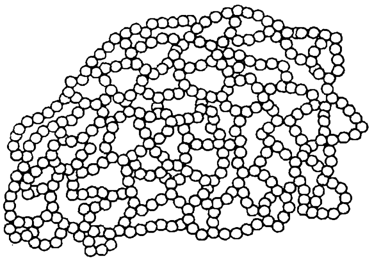


Figure 2.4 Multigrain (honeycomb) structure (Mitchell, 1976)

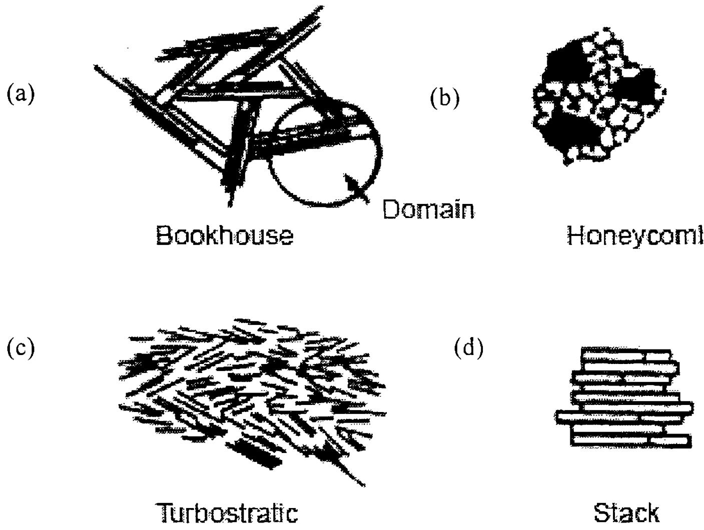


Figure 2.5 Idealized clay fabrics (Sides and Barden, 1970, as presented by Cotecchia and Chandler, 1997)

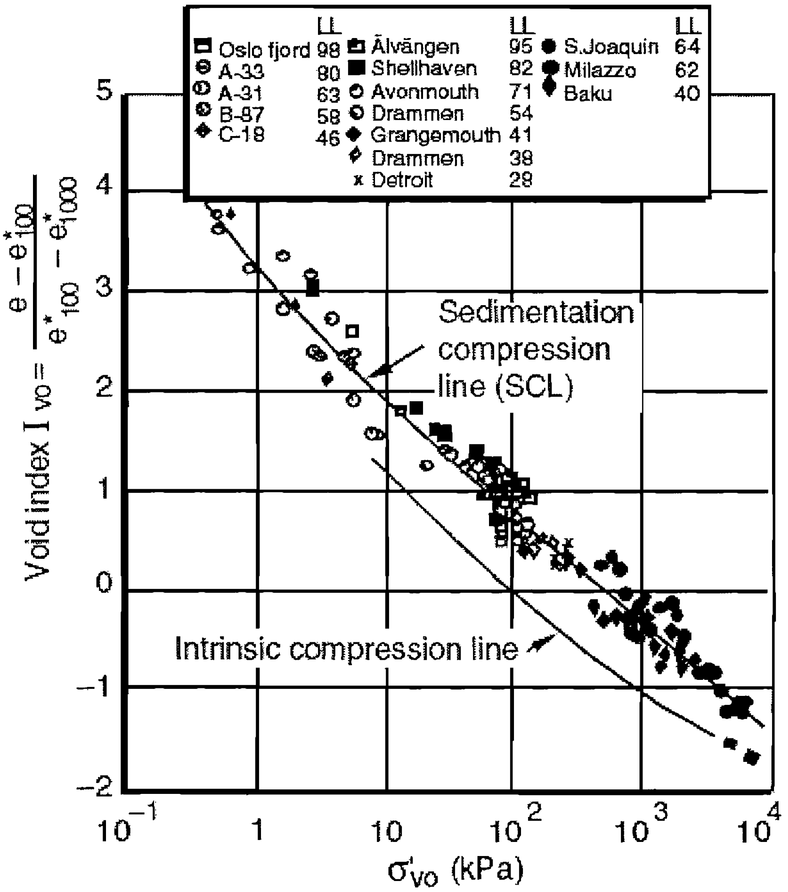


Figure 2.6 Relationship between the normalized one-dimensional compression curve of the natural and reconstituted specimens for many normally consolidated clays (Burland, 1990).

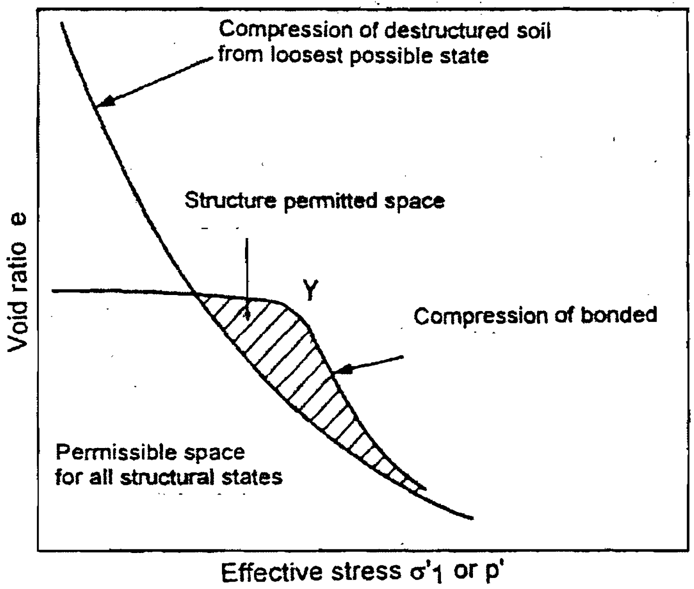


Figure 2.7 Structured and destructured soils in oedometer (Leroueil and Vaughan, 1990).

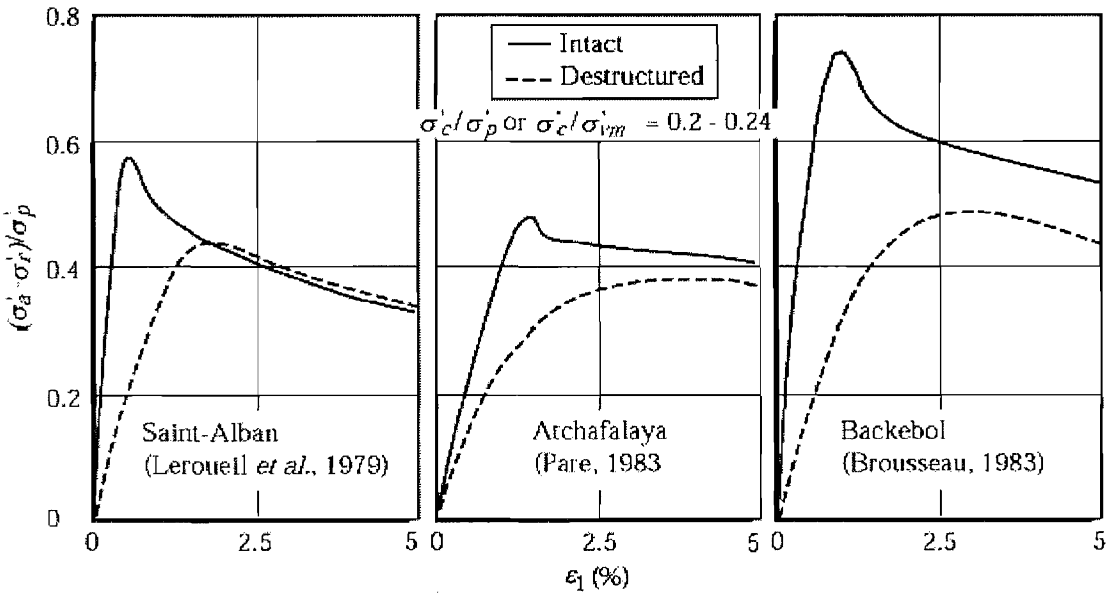


Figure 2.8 Stress-strain relationship obtained on undrained triaxial test on intact and destructed clays (Tavenas and Leroueil, 1985 as presented by Leroueil and Vaughan, 1990).

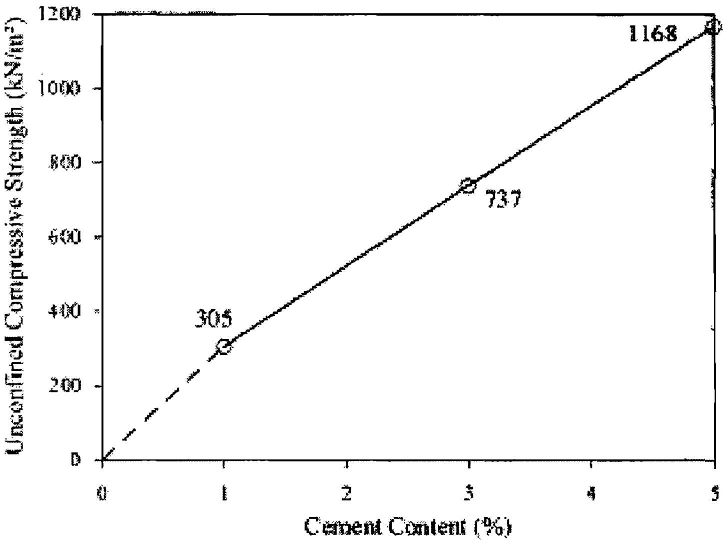


Figure 2.9 Effect of cement content on unconfined compressive strength (Schnaid et al., 2001)

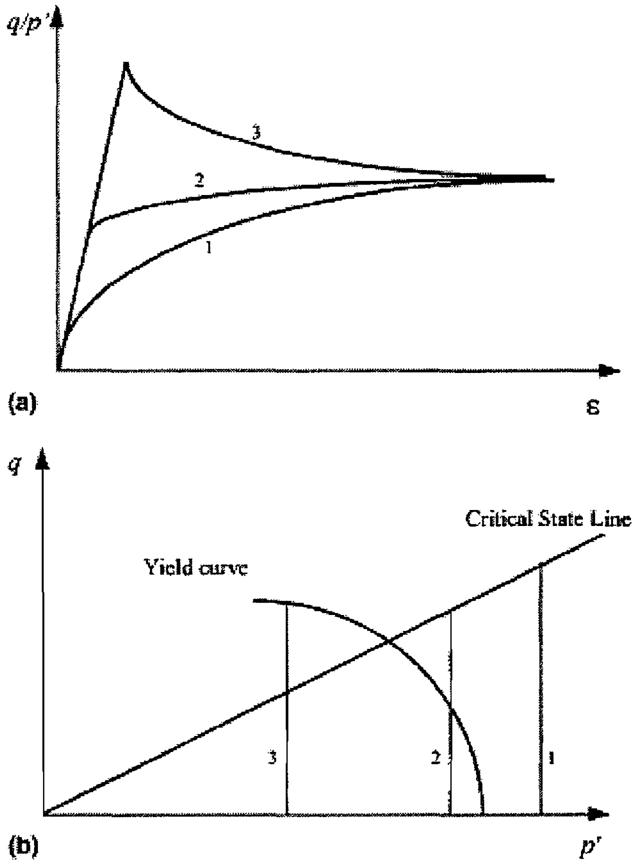


Figure 2.10 Idealized behaviour of cemented soils; (a) stress-strain behaviors (b) stress-paths (Coop and Atkinson, 1990).

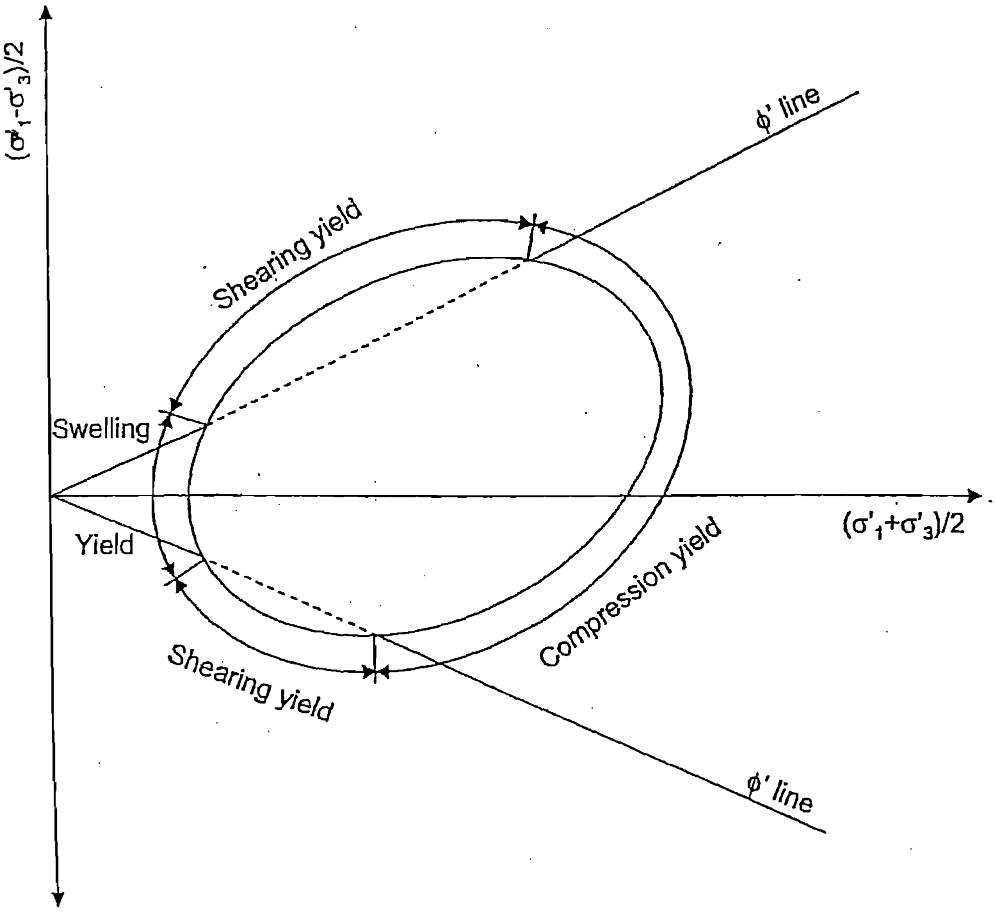


Figure 2.11 Different types of yielding (Leroueil and Vaughan, 1990)

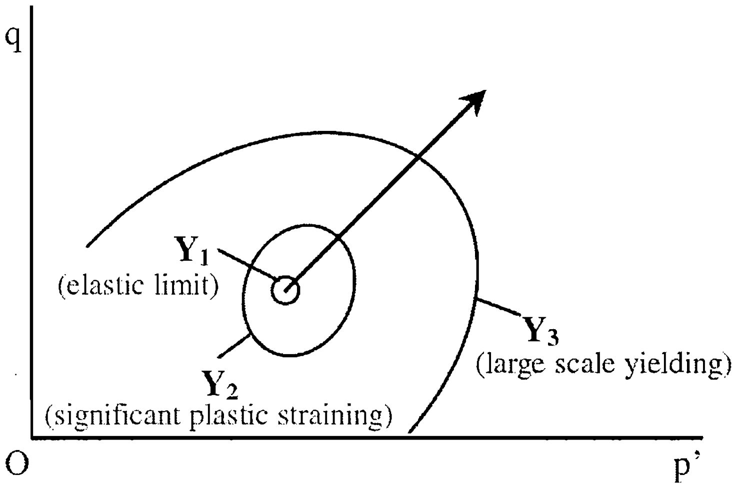


Figure 2.12 Multiple yield surfaces (Jardine, 1992).

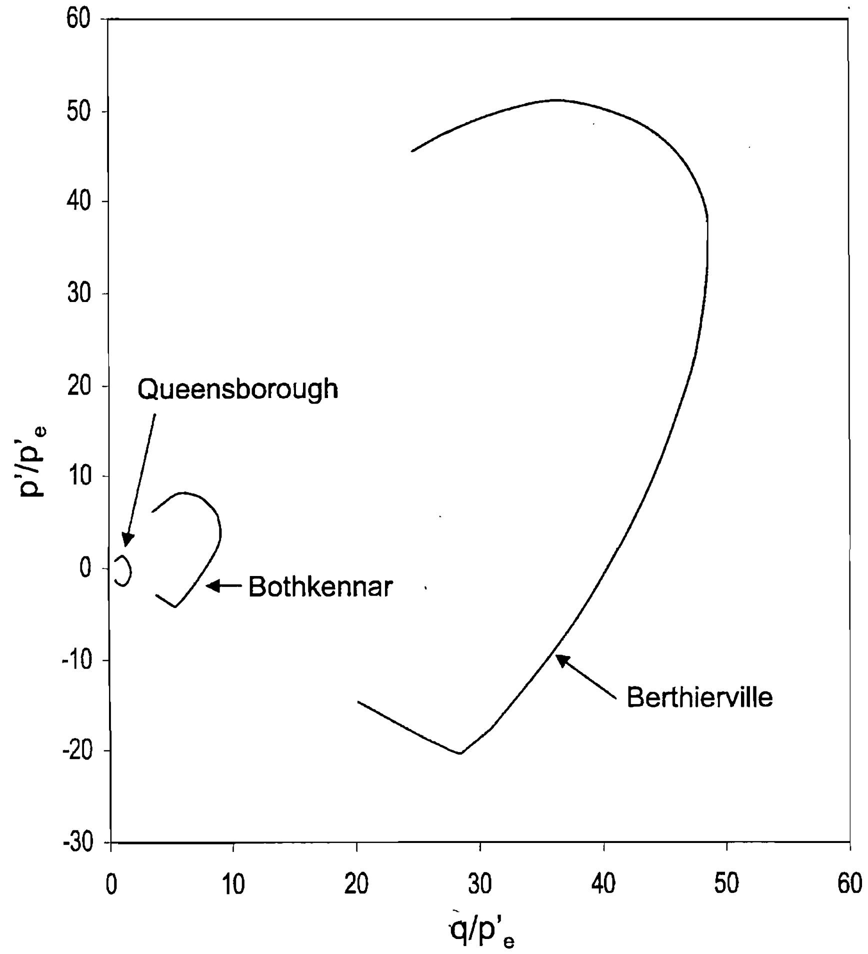


Figure 2.13 Comparison of State Boundary Surfaces for the natural Queensborough, Bothkennar and Berthierville clays (Smith, 1992)

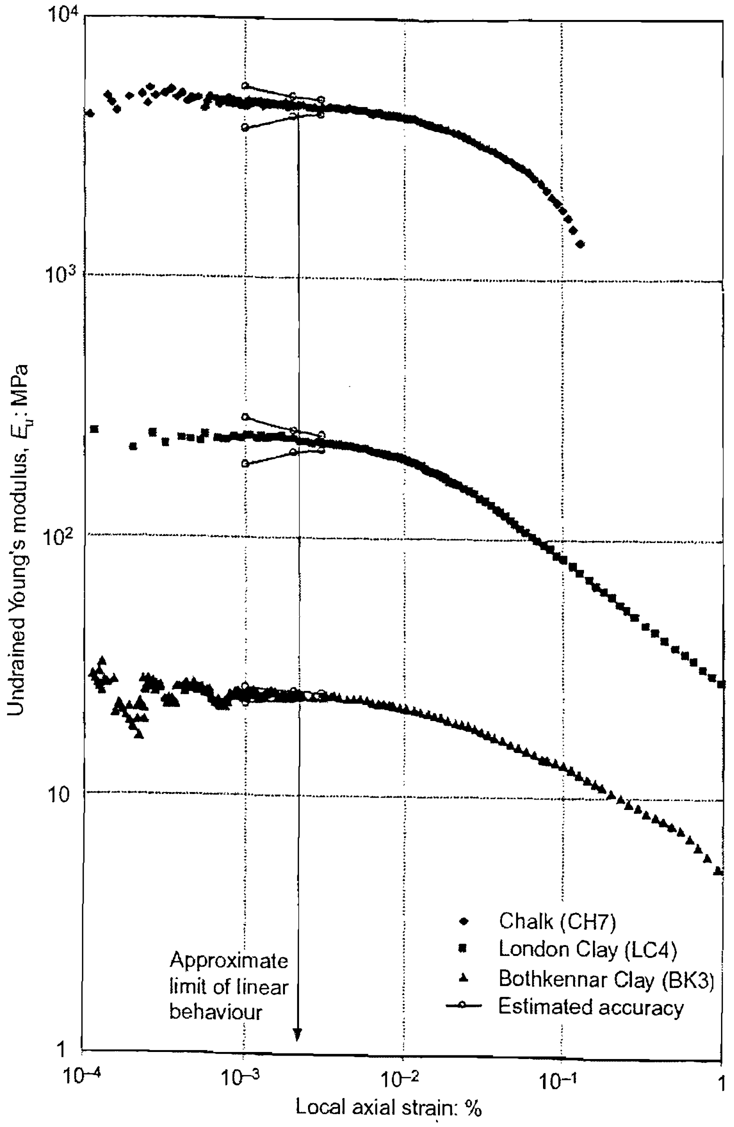


Figure 2.14 Comparison of the stiffness of the three materials used in the study by Clayton and Heymann (2001).

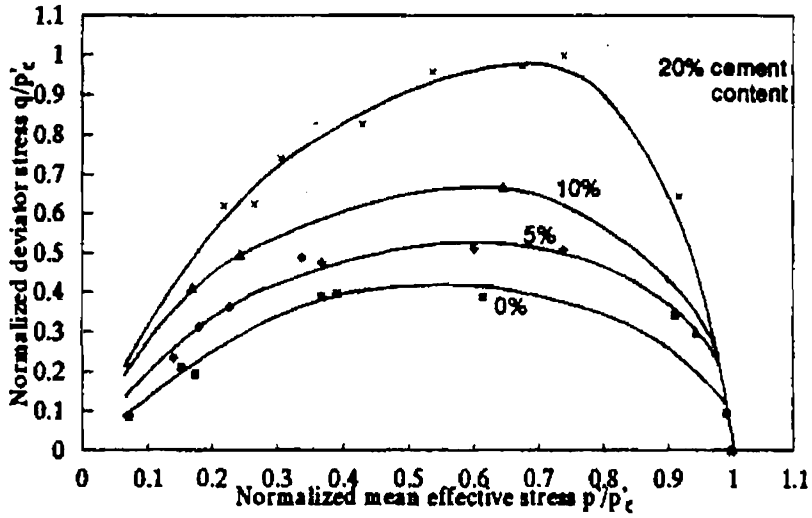


Figure 2.15 Effects of cement content on the shape of the yield locus (Huang and Airey, 1993).

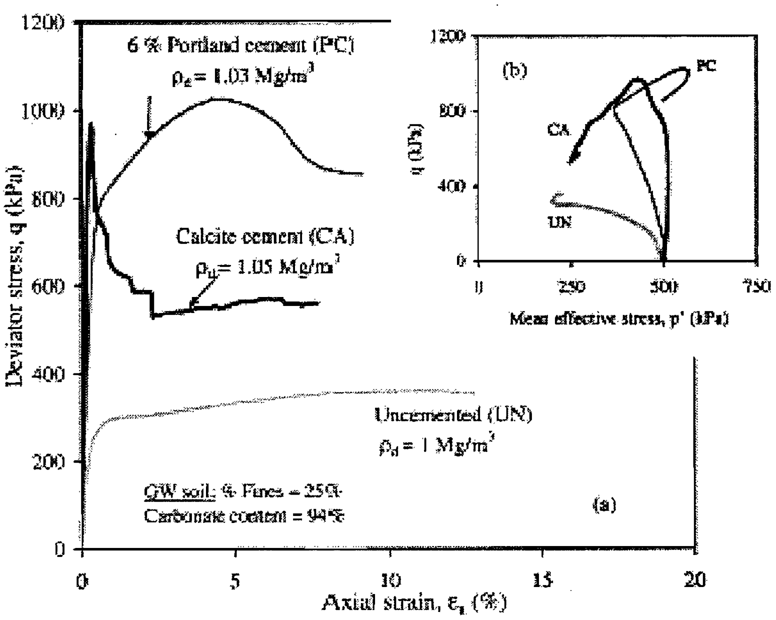


Figure 2.16 Response of Goodwy (GW) soil in CIU for different cementing agents (a) stress-strain curves (b) stress paths (Ismail et al, 2002b).

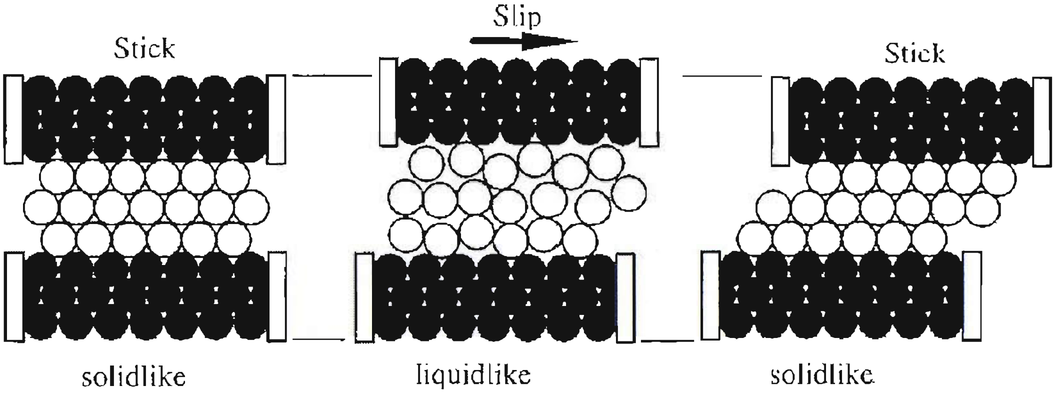


Figure 2.17 The transition from static to dynamic states. In the static state, the material in the shear zone is dense and solidlike. To allow slip this material must dilate, becoming more liquidlike (Cain et al., 2001).

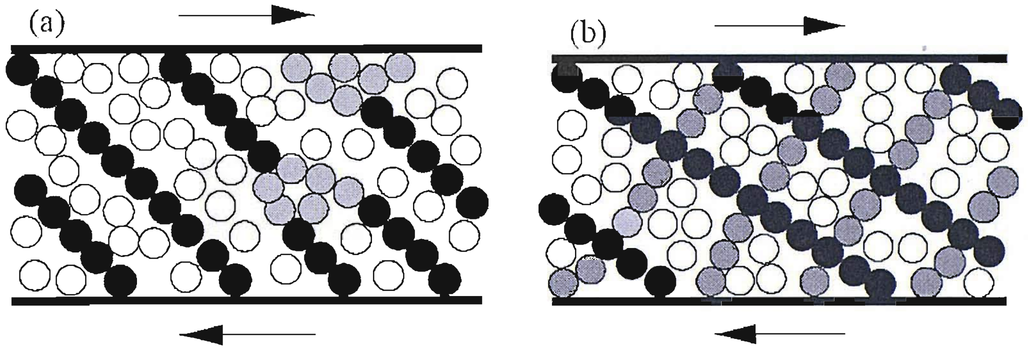


Figure 2.18 (a) A jammed colloid. Black: force chains; grey: other force-bearing particles; white: spectators, (b) Idealized rectangular network of force chains (Cates et al., 1998).

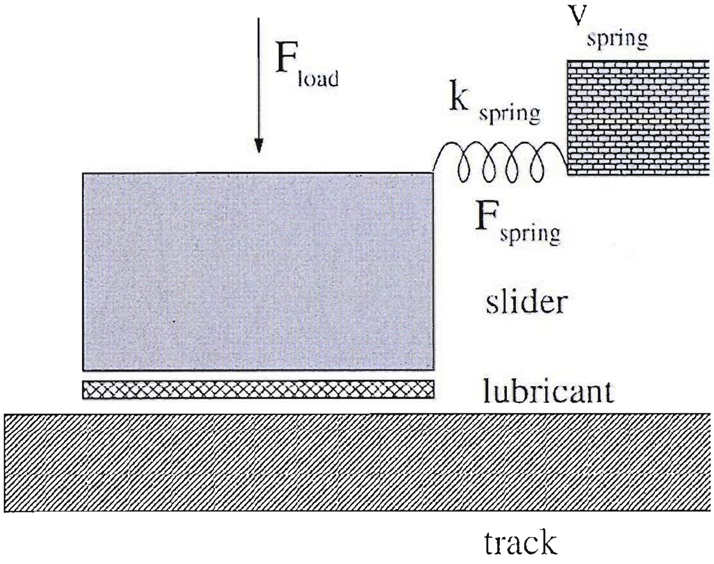


Figure 2.19 A typical experimental device for tribology studies. A top solid substrate (slider) is put on the bottom substrate (track). The slider and track may be separated by a thin lubricant film and compressed together by a loading force (F_{load}). The slider is connected with the base moving with a constant velocity v_{spring} through a spring of the elastic constant k_{spring}. The output (measured) parameter is the spring force F_{spring}. (Braun and Naumovets, 2006).

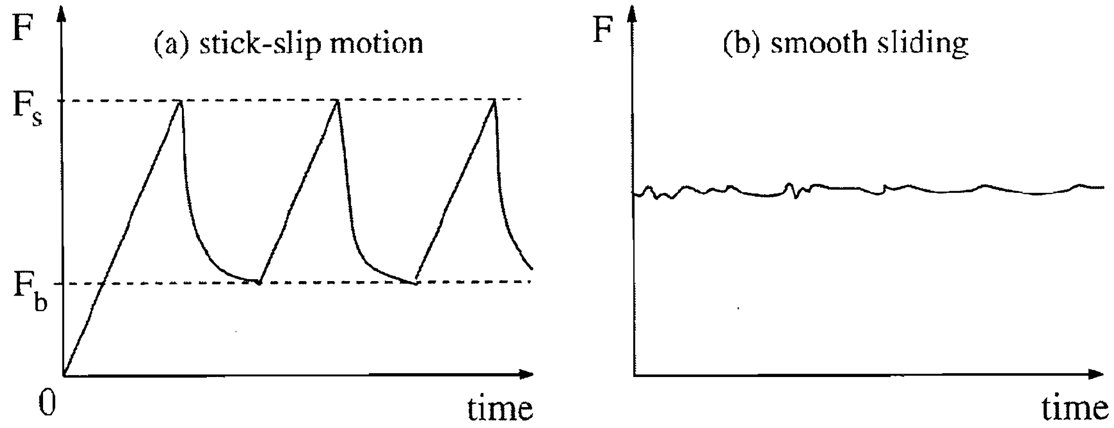


Figure 2.20 Friction force as a function of time in the **(a)** stick-slip, **(b)** smooth sliding regimes (schematically) (Braun and Naumovets, 2006).

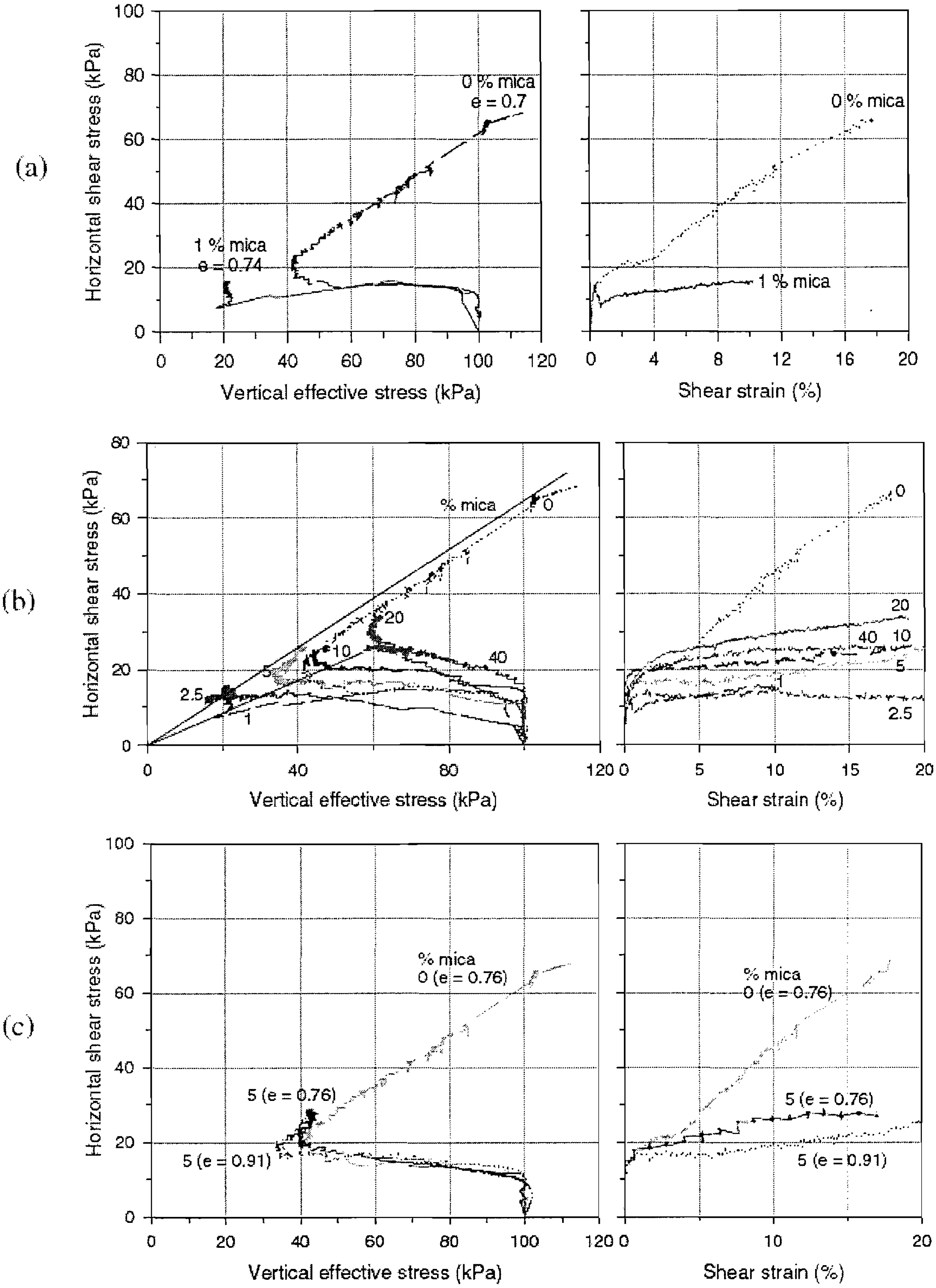


Figure 2.21 Effect of mica content (Hight and Leroueil, 2003).

CHAPTER 3

EXPERIMENTAL TECHNIQUES

This chapter will present the basic design and modification of a 100 mm diameter conventional triaxial testing apparatus that allows tests to be conducted under controlled temperatures ranging between 15 ^{o}C and 60 ^{o}C. Following the description of the apparatus, the reader is introduced to the instrumentation. The chapter reports the experiments carried out to determine the effects of various temperatures on the stress-strain and the stiffness of a synthetic geomaterial under monotonic loading. The chapter explores the mechanical behaviour of geomaterials with various pore fluids under a number of temperatures applied. A series of experiments conducted for investigating the influence of particle shape on compressibility, stiffness and stress- strain behaviour is also presented. Also described is the growth of sucrose crystals with the time, viscosity tube measurements of various fluids.

3.1 Temperature controlled triaxial testing apparatus

The triaxial apparatus is one of the most commonly used tests in soil mechanics. A conventional cell consists of three principal components, (i) base, which is the machined from corrosion-resistant metal, forms the pedestal on which the specimen rests and incorporates the various connections, (ii) top cap and the removable cell body enclosing the fluid pressure to be applied, (iii) the loading ram to apply deviatoric stress to the specimen. However, in this study to generate more understanding of the influence of structure, particular attention has been given to the development of a temperature controlled cell having a coil to provide heat transfer in addition to the main components briefly described above.

A conventional 100-mm-diameter Wykeham Farrance compression triaxial machine having a 50-kN load capacity was modified to include a temperature control system. Figure 3.1 shows a schematic representation of working mechanism of the set up. As can be seen from the figure, the set-up consists of load cell, displacement transducers, pressure transducers, pressure controllers, coil in the cell, thermistors, pump, chiller, water bath, and data acquisition system.

The temperature controlled system is based on a coil in the cell, which circulates water at various temperatures. The temperature difference between the coil and the other components of the apparatus is the driving force for the heat to be exchanged. The wider temperature difference, the greater amount of heat needs to be transferred between the coil and the other components. Accordingly, it was realized that the thermal conductivity of the material and the physical attributes of the coil, including tube size and the distance between the cords, affect the amount of heat to be transferred between the two media. Considering the availability of material, a copper tube having 10 mm outside and 0.7 mm wall thickness, as one of the most effective heat transfer material, was selected to build the coil. Figure 3.2 presents some of the details as well as the position of a sample in the coil.

During the testing of a specimen in the apparatus, a peristaltic pump (Watson Marlow 603S-1), a chiller (Tricity 455x455x845) and a heater (Grant type, having 0-80 ^{o}C temperature capacity) were used to ensure that the temperature remained constant in the cell. Great care was taken to manually control the water suppliers (i.e., pump, chiller, heater), which was crucial to maintain the temperature constant in the cell.

3.2 Data acquisition system

During the experimental investigation, GDSLAB v 2.0.8 software package was used to log the output of the instrumentation. GDSLAB, produced by GDS Instruments Limited, is a program for the control and/or data acquisition of geotechnical laboratory tests.

3.3 Instrumentation

For this study, it was realized that local measurements of axial strain were required in order to make more accurate estimates of the soil stiffness. Two submersible linear variable differential transformers (LVDT) were employed to measure the axial displacement in the middle third of the specimen in diametrically opposite positions. They were used over a 27 mm gauge height (clear distance). LVDTs produced by R.D.P. Electronics Ltd. were calibrated against a Mitutoyo micrometer. Two LVDTs with a ±1mm linear range was kept in place by means of a plastic screw on brackets, and then they were held in position on the specimen (Figure 3.2). As can be seen from Figures 3.3 and 3.4, maximum error was 1.5 μm for the LVDTs (S/N 25706 and S/N 25708) over a 1 mm calibration range.

The axial strain was also measured externally using a linear displacement sensor (LDS) fixed to the ram of load cell (see Figure 3.2). This allowed measurements of large strains in the specimen. The LDS had a 25 mm range and was produced by Measurement Group Ltd. Figure 3.5 shows the calibration results for the LDS. It can be seen from the Figure 3.5 that maximum error is around 20 μm over the 24 mm calibration range.

A 5-kN Wykeham Farrance STALC 4958 type internal load cell was calibrated using Budenberg dead-weight tester Model 580 L. The calibration graph for the load cell is given in Figure 3.6. The figure shows approximately 4 N maximum error.

In order for the cell and the back pressures to be measured, two separate pressure transducers were used, one connected to the back pressure line and the other to the cell pressure line. The Budenberg dead-weight tester was used for calibration of the pressure transducers. The pressure transducers had a 10 bar pressure range and were type PDCR 810 produced by Druck Limited. Figures 3.8 and 3.9 present the calibration results for the PDCR 810 type cell and back pressure transducers. The Figures 3.7 and 3.8 show a 0.4 kPa maximum error for PDCR 810 S/N 1912137, and similarly around 0.4 kPa maximum errors for the PDCR 810 S/N 2024472 over a calibration range of 900 kPa pressures.

3.4 Material properties

The experimental work has been directed mainly towards an investigation of (i) the effects of sucrose ($C_6(H_2O)_{11}$) solutions at various concentrations, and (ii) the influence of different proportions of platy fines on the behaviour of coarse rotund sand.

Leighton Buzzard Sand representing equidimensional coarse rotund particles was used, which is a standard material referred in BS 1881-131:1998. The Leighton Buzzard Sand used in the experiments was fraction B, having minimum and maximum dry densities of 1.48 g/cm^3 and 1.74 g/cm^3 respectively. The procedure defined by Cresswell et al. (1999) was used to obtain the maximum dry density by pluviation and that of BS1377 (1990) was used to find the minimum dry density. As it can be seen from the Figure 3.9, more than 90% of the coarse sand particles, which are rounded (Figures 3.10) and quartz, are between (approximately) 0.6 mm and 1.1 mm. It can also be clearly seen from the figure that D_{10}, D_{30} and D_{60} sizes are around 0.68, 0.78 and 0.92 respectively. Thus, the coefficient of uniformity (C_u) and the coefficient of curvature (C_c) have been calculated as 1.35 and 0.97 respectively.

For the experimental works aiming to have a greater understanding of the influence of different proportions of platy fines on the behaviour of coarse rotund sand, 52 mm to 105 mm mica powder and icing type of sucrose were separately mixed with the Leighton Buzzard Sand. Mica supplied by Dean and Tranter Ltd. can be simply described as a group of silicate minerals that split easily into thin flakes along lines of weakness in their cleavage structure (Figure 3.11). Mica has a structure of aluminium silicate sheets weakly bonded together by potassium ions layers. These ion layers lead a cleavage of mica particles. Despite mica particles have an easy cleavage; the cleavage sheets are fairly durable and found in sands without destroying. They have a pearly lustre, glossy, and they are found in many metamorphic rocks and igneous. Their good thermal and electrical insulation qualities make them valuable in industry. It was realised that determination of minimum and maximum dry densities was not as easy as that of the Leighton Buzzard Sand, because of their shape, since the particle's orientation could easily change as they're falling. Therefore, here it is preferred to refer the Table (3.1) that shows the basic properties of mica used in the experimental study as stated by the supplier company.

In addition to the Leighton Buzzard Sand and mica used in the experimental study, sucrose solutions at various concentrations were selected as pore fluid because of simplicity and availability. Carbohydrates including sucrose consist of the elements oxygen (O), hydrogen (H) and carbon (C) with a ratio of hydrogen twice that of oxygen and carbon. Sucrose solutions employed during the experimental study were prepared by using two different types of sucrose solids $C_6(H_2O)_{11}$, namely; (i) castor, and (ii) icing sugars. Sucrose is freely available in the form of fine table sugar. The particle size distribution of the castor sucrose varies between 0.1 mm to 0.5 mm (Figure 3.9). Under a microscope, one can see that the castor sugar crystals are oblong and slanted at both ends (Figure 3.12); however icing sugar particles can be to be platy (Figure 3.13). Furthermore, Figures 3.14 and 3.15 show comparisons between castor sugar and mica particles, and icing sugar and mica particles respectively. As can be seen from the production nutritional information shown in Table 3.2, castor and icing sucrose crystals have the same type of nutritional values, differing in the quantity is available. The only difference in ingredients of icing sucrose from the castor sucrose is tricalcium phosphate.

When one adds sugar to water, the sugar crystals dissolve and the sucrose goes into solution. But one cannot dissolve an infinite amount of sucrose into a given volume of water. When as much sucrose have been dissolved into a solution as possible, the solution is said to be saturated. The saturation level is a function of temperature. The higher temperature, the more sucrose that is able to be held in solution (Figure 3.16). But when the solution begins to cool, there will be more sugar in solution than is normally possible at the former temperature. Then the solution is said to be supersaturated with sucrose, and crystallization may take place.

Depending on the concentration of sucrose in the solution and the temperature, sucrose crystals may grow. Crystal form when the concentration of a solution is higher than the solubility of the sucrose. There are two simple methods that can be employed to produce sucrose crystals; the slow cooling method and the evaporation method. These two methods assume that one begins with a saturated solution of water and sucrose particles. If one doesn't use a saturated solution, then the crystals won't quickly grow, if one adds too much sucrose, new crystals can start to grow on the unsolved sucrose particles. Figure 3.17 shows some representative examples of sucrose crystal growing over the time. For this relatively simple observation, 207 g castor sugar was dissolved per 100 g water, and then poured on to 5 mm-diameter steel spheres in a 15 mm depth plastic plate. Care was taken to keep the solution's level just above the steel spheres' top surface. Then, the plastic plate was closed with a plastic cover and left to a temperature controlled laboratory environment. The developing of the crystal growth was observed by a microscope day by day, until no change was observed. Thus, the Figure 3.17 shows the crystal growing at the end of 2, 4, 6, 8, 10, and 11th day.

3.4.1 Viscosity measurements for various liquids

When a fluid deformed due to applied external forces and flow, frictional effects are seen by the motion of molecules with respect to each other. The effects are encountered in all fluids are because of their viscosity. Therefore, the viscosity may be described as a resistance of a fluid to a change in movement of portions next to each other, or change in shape. For example, water flows from a tilted bottle more quickly and easily than oil does. Oil is more viscous than water, so although gravity creates nearly the same stresses in oil and water, the more viscous fluid flows more slowly.

Here in this thesis, a viscometer which is relatively easy to use was selected to determine the viscosity values of the fluids (i.e., water, sucrose solutions at various concentrations, and oil) used in the experimental study. All the measurements were carried out in a temperature controlled room at 23 ^{o}C using the VHB-320-070F type viscometer produced by Poulten Selfe & Lee Ltd. The water used in the measurements was de-aired water. Viscosity of the solutions with castor sucrose were measured at the ratios of 0.5g sucrose/100g water, 5g sucrose/100g water, 25g sucrose/ 100g water, 50g sucrose/100g water, 100g sucrose/100g water, and 207g sucrose/100g water. In addition, viscosity of the Dow Corning (200/50) type silicon fluid produced Dow Corning Corporation was measured using the same testing apparatus. To understand the effect of viscosity on the behaviour of specimens tested, determined viscosity measurements of the liquids used as pore fluid in the experimental study are illustrated in Table 3.3. The apparatus used for the viscosity measurement is also shown in Figure 3.18.

3.5 Sample preparation

The research mainly focuses on the mechanical effects of sucrose solutions at various concentration and oil as well as water as pore fluid within porous medium, its aim being to illustrate the coupling between the bonding of particles and the overall behaviour of geomaterials. It is also aimed to re-investigate the particle effects on the soil behaviour.

Adding sucrose through the sand particles could potentially lead to both solution and precipitation by either slowly heating or cooling the pore fluid. The use of the slow cooling method where one produces a hot saturated solution and cools it down to precipitates crystals, was selected for a procedure to be followed inside a triaxial cell under controlled effective stress. The testing technique described by Theron (2004) was employed to investigate the effect of particle shape on the soil behaviour.

To study the mechanical properties of specimens tested, it is crucial to maintain consistency between the specimens prepared for the testing. Therefore, great care was taken to have reasonable repeatability during the preparing of the specimens to be tested. The sample preparation technique was the same for all specimens. During testing of the specimens, eleven different preparation methods were employed for four different mixtures. For all the experiments, Leighton Buzzard Sand fraction B was tested in compression and under undrained conditions.

(1a) Specimens tested at 60°C with sucrose solution pore fluid. These specimens were prepared and tested approximately at the same temperature level, which was 60°C.

(1b) Specimens tested at room temperature with sucrose solution pore fluid. These specimens were prepared and tested at the room temperature.

(1c) Specimens tested at the end of gradual decrease from 60°C to the room temperature with sucrose solution pore fluid. In this method, the specimens were prepared using a heater mat at the beginning of specimen preparation, and then the temperature was decreased step by step. The procedure adopted was aimed at observing the growth of crystal.

(1d) Specimens tested at the end of gradual decrease to room temperature followed by sharp increase to 60°C with sucrose solutions. This method was employed to make a comparative study to understand the possible difference between specimen prepared with heater mat and specimens prepared without heater mat.

(2a) Specimen tested in loose form at room temperature with de-aired water. This test was done as a reference experiment.

(2b) Specimens tested in dense form at room temperature with de-aired water.

(3) Specimen tested at room temperature with silicon oil as pore fluid. This specimen was tested to understand the influence of Dow Corning silicon oil on the behaviour of geomaterial.

(4a) Specimens mixed with 5% mica tested at room temperature with de-aired water. These series of experiments were done to understand the effects of particle shape.

(4b) Specimens tested at room temperature with 10% mica.

(4c) Specimens tested at room temperature with 15% mica.

(4d) Specimens tested at room temperature with sucrose.

A description of the technique **(1a)** employed for preparing the loose coarse rotund sand specimens with 'sucrose solutions' to be tested at 60°C is given below;

1. The required amount of dry sand was weighed and placed in a conventional oven at a minimum of 70 °C. It was then kept in the oven at least 24 hours. The required amount of sucrose and de-aired water were also weighed just before preparing of the specimen on the pedestal. Sucrose and water quantities were determined based on the desired temperature value from the solubility chart for the sucrose grains. In this investigation, the mix ratios of sucrose to water were 207/100 g/g, 287/100 g/g to be tested at 60°C.
2. The sucrose grains were mixed with pre-heated water at a minimum of 70 °C. The mixture was then stirred by a wire, and kept on a hot plate until the sugar dissolved entirely.
3. A membrane was attached to the pedestal using an o-ring, and a three-part split mould was placed around the endplate.
4. The mould was then covered with a heater mat that provides an even distribution of heat to the mould. The heater mat is a simple and effective method of applying heat to the specimen on the pedestal. The heater mat has a 12 V supply voltage, 80 W power rating, 200 mm length and 400 mm width (RS 245-578).
5. The heater mat was connected to a battery, and around 1-cm-height oil was poured on to the pedestal (to avoid any crystal deposition in the pedestal). The membrane was then filled with the sucrose solution in layers of approximately 67 mm (1/3 height of the mould). Figure 3.19.a illustrates the process followed until this point. Region (I) shown in Figure 3.20 gives the typical change in temperature and pressure in both the specimen's itself and the triaxial cell. It can be seen that the temperature in the specimen exhibits a sharp increase to 85 °C. due to the pouring of sucrose solution into the mould. However, the temperature measurement in the cell was almost constant at approximately room temperature.

6. The sand was removed from the oven and gently spooned in to the mould in thin layers without vibration or any type of method that could lead to compaction. This procedure was repeated until the mould was filled.
7. When the mould was completely filled, the top of the specimen was flattened by gently sliding the sand particles from the centre of the specimen to its side using the tip of a small screwdriver. Excess sand was removed and the weight of the remaining sand was recorded for density measurement.
8. The o-ring stretcher with two o-rings was slipped over the top plate which was placed on top of the specimen. The membrane was slipped over the top plate, ensuring that no sand particles were trapped between the two before using the o-rings.
9. Approximately 20 kPa of vacuum was applied to the specimen. The vacuum was maintained by specifying a target pressure of -20 kPa to the GDS pressure controller, connected to the pedestal. The region (II) shown in Figure 3.20 illustrates the process from the end of the region (I) to this point. It can be seen that the temperature in the specimen gradually decreases to just below 70 ^{o}C. The thermistor outside the specimen shows values more than room temperature, because the heat of the specimen affects the surrounding devices including the thermistor.
10. Once the pore pressure measured through the pedestal stabilized at this pressure value, the heater mat was removed, and then the three-part-mould was carefully split to prevent disturbance to the specimen.
11. The dimensions of the specimen were measured in mm to two decimal places. The LVDT brackets were glued to the membrane using super glue at the middle third of the specimen. The LVDTs were then inserted in the place with screw on the side of the bracket. The gauge length of the LVDTs was adjusted to ensure the maximum linear range. This was done by adjusting the screw. (If necessary) Elastic bands were placed around the brackets and cables for additional support. Note that this step needed to be carried out as quickly as possible, also as careful as possible, because any delay in the process could lead to a decrease in specimen temperature, which might cause a change in the properties of specimen

to be tested. As shown by region (III) in Figure 3.20, the temperature in the specimen decreased around 20 °C in a short time. Figure 3.19.b shows the specimen at this stage.

12. The coil was assembled (Figure 3.19.c), and then the cell was closed and filled with pre-heated water. At the same time, circulation of heated water in the coil was started using a peristaltic pump.
13. The vacuum inside the specimen was reduced while gradually increasing the confining water pressure in small steps until the desired value (400 kPa) was achieved. The process from the end of region (III) to this point is represented by region (IV) in the Figure 3.20. The specimen under a constant temperature was left overnight to ensure that the heat evenly distributes through the specimen, and that any air trapped in the specimen would dissolve into the pore fluid, which was 400 kPa cell pressure and 300 kPa back pressure. Figure 3.19.d and the region (V) in Figure 3.20 represent the details of this step.
14. The dry weight of the specimen was calculated by subtracting the dry weight of the sand remaining from the original dry weight of the sand weighed before the sample preparation.
15. Next day, the test was commenced.

A description of the technique **(1b)** employed for preparing the loose coarse rotund sand specimens with 'sucrose solutions' to be tested at room temperature is as follow;

1. A required amount of sand, sucrose, and water were weighed at room temperature. Sucrose and water quantities were determined based from the solubility chart for the sucrose grains shown in Figure 3.16. The mix ratios of sucrose to water were 5/100 g/g, 25/100 g/g/, 100/100 g/g, 150/100 g/g to be tested at room temperature.
2. The sucrose grains were mixed with water. The mixture was then stirred by a wire until it dissolved entirely. It was not necessary to use hot plate for these mixtures. However, warm water was preferred to prepare the sucrose solution mixtures at these ratios.

3. The three-part split mould was placed around the pedestal following the attaching of membrane.
4. A three-part mould was used to retain the shape of the specimen during the preparing and to support it until an effective pressure was applied. The top of the membrane was folded back over the mould to fit the membrane and mould. The membrane was then filled with the sucrose solution in layers of approximately 67 mm (1/3 height of the mould).
5. The sand was gently spooned in to the mould in thin layers without vibration or any type of method that could lead to compaction. This procedure was repeated until the mould was filled.
6. When the mould was completely filled, the top of the specimen was flattened by gently sliding the sand particles from the centre of the specimen to its side using the tip of a small screwdriver. Excess sand was removed and the weight of the remaining sand was recorded for density measurement.
7. The o-ring stretcher with two o-rings was slipped over the top plate which was placed on top of the specimen.
8. Approximately 20 kPa of vacuum was applied to the specimen. The vacuum was maintained by specifying a target pressure of -20 kPa to the GDS pressure controller, connected to the pedestal.
9. Following the pore pressure stabilized at the -20 kPa pressure value. The three-part-mould was carefully split to prevent disturbance to the specimen.
10. The dimensions of the specimen were measured to determine the density values. Following the glue of the LVDT brackets to the membrane at the middle third of the specimen, the LVDTs were then inserted in the place with screw on the side of the bracket.
11. Following the assembling of the coil, the cell was closed up and filled with water at room temperature.
12. The pressure inside the specimen was increased while gradually increasing the confining water pressure until the desired value was achieved. The specimen was left overnight to ensure that the heat evenly distributes through the specimen

(cell pressure was 400 kPa, and the back pressure was 300 kPa), and that any air trapped in the specimen would dissolve into the pore fluid.

13. Next day, the test was commenced.

A description of the technique **(1c)** employed for preparing the loose coarse rotund sand specimens with 'sucrose solutions' to be tested at room temperature followed by a gradual decrease from 60°C can be explained as below;

1. A certain amount of sand was kept in a conventional oven at a minimum of 70 °C at least 24 hours. Sucrose solution as a pore fluid was prepared at the mix ratios of sucrose to water were 207/100 g/g, 287/100 g/g to be tested.
2. The sucrose solution was prepared using pre-heated water at a minimum of 70 °C, and then kept on a hot plate until it dissolves completely.
3. The three-part split mould covering a membrane was placed around the pedestal.
4. The heater mat was placed around the mould, and then connected to a battery.
5. To avoid any crystal growing in the pedestal first oil was poured, and then the specimen was started to prepare by spooning the in to the mould in thin layers without vibration or any type of method that could be reason of a compaction.
6. Completing the sample preparation, the o-ring stretcher with two o-rings was slipped over the top plate which was placed on top of the specimen, and then a 20 kPa of suction was applied to the inside of the specimen.
7. Once the pore pressure stabilized, the heater mat was removed, and then the three-part-mould was carefully split to prevent disturbance to the specimen.
8. The LVDT holders were glued to the membrane, and then the LVDTs were placed with screw on the side of the holder.
9. The coil was assembled; the cell was closed and then filled with pre-heated water.
10. A desired confining pressure and pore pressure were employed. The specimen under a constant temperature was left overnight to ensure that the even temperature distribution through the specimen, and that any air trapped in the specimen would dissolve into the pore fluid.

11. The temperature of the specimen was reduced from approximately 60°C to room temperature, even (if needs) below room temperature using a chiller. The decrease in the temperature was employed at every (approximately) 10°C in 24 hours time intervals. It was postulated that this approach leads to a precipitation in the porous medium tested and that could modify the mechanical properties of the solid. A typical figure showing the gradual decrease in the temperature from 60°C to the 17°C as described above is presented in Figure 3.21.
12. At the end of 5th day, the loading was commenced.

The technique **(1d)** employed for preparing the loose coarse rotund sand specimens with 'sucrose solutions' to be tested at room temperature is described below. In this technique, a specimen was simply prepared at room temperature, then heated up to 60°C in one day and cooled down gradually to the room temperature or below it using a chiller.

1. The procedure followed steps 1 to 10 for the preparation of the loose rotund sand specimens in the technique 1c.
2. The temperature of the specimen was sharply increased to approximately 60°C and then gradually reduced to room temperature. The decrease in temperature employed was (approximately) 10°C every 24 hours, while the increase was employed in a couple of hours. It was postulated that this approach leads to a precipitation in the porous medium tested and that could modify the mechanical properties of the solid. A typical figure showing the gradual decrease in the temperature from 60°C to the 17°C as described above is presented in Figure 3.21.
3. At the end of a desired time (not less than 2 days), the loading was commenced.

Preparation of the loose Leighton Buzzard with 'de-aired water' **(2a)** specimens to be tested at room temperature was as follow;

1. A sufficient amount of coarse rotund sand particles were submerged in de-aired water and stirred by a spoon.
2. The circumference of the pedestal was coated with a thin layer of grease to ensure a watertight connection between the, membrane and o-rings. The membrane was then attached to the pedestal and two o-rings were pushed into place using an o-ring stretcher.
3. A three-part split mould was placed around the pedestal to retain the shape of the specimen during construction and to support the specimen until a suction was applied.
4. The sand-water mix was gently spooned in to the mould in thin layers and levelled without vibration or any type of method that can lead to compaction. Great care was taken to ensure that the sand remained submerged in de-aired water at all times. This procedure was repeated until the mould was filled.
5. When the mould was completely filled, the top of the specimen was flattened by gently sliding the sand particles from the centre of the specimen to its side using the tip of a small screwdriver.
6. The o-ring stretcher with two o-rings was slipped over the top plate which was placed on top of the specimen. The membrane was slipped over the top plate, ensuring that no sand particle was trapped between the two before using the o-rings.
7. An approximately 20 kPa vacuum was applied to the inside of the specimen. The vacuum was maintained by specifying a target pressure of -20 kPa to the GDS pressure controller, connected to the pedestal.
8. Once the pore pressure measured through the pedestal stabilized at this pressure value, the three-part-mould was carefully split to prevent disturbance to the specimen.
9. The dimensions of the specimen were measured in mm to two decimal places. The LVDT brackets were glued to the membrane using super glue at the middle third of the specimen. The LVDTs were then inserted in the place with screw on the side of the bracket. The gauge length of the LVDTs was adjusted to ensure

the maximum linear range. This was done either by adjusting the screw. Elastic bands were placed around the brackets and cables for additional support.

10. The coil was assembled, and then the cell was closed and filled with water at room temperature.
11. The vacuum inside the specimen was reduced while gradually increasing the confining water pressure until the desired value was achieved. The specimen under a constant temperature was left overnight to ensure that any air trapped in the specimen would dissolve into the pore fluid.
12. The loading was commenced next day.
13. Completing the test, specimen was removed from the pedestal and weighed for bulk density. The specimen was then took to an oven, and weighed next day for the dry density measurement.

A description of the technique **(2b)** follows for preparing the dense coarse rotund sand specimens with 'de-aired water' can be described as follow;

1. The procedure followed steps 1 to 3 for the preparation of the loose rotund sand specimens in the technique 2a.
2. The sand-water mix was spooned in to the mould in thin layers and levelled. After each consecutive layer, the mould was tapped to increase the density of the specimen prepared. Great care was taken to ensure that the sand remained submerged in de-aired water at all times. This procedure was repeated until the mould was filled.
3. Steps 5 to 13 of the procedure for preparing loose rotund sand specimens in the technique 2a were then repeated.

Similarly, a description of the technique **(3)** follows for preparing the coarse rotund sand specimens with 'silicon oil' as pore fluid at room temperature.

1. The procedure followed steps 1 to 3 for the preparation of the loose rotund sand specimens in the technique 2a.

2. The membrane was then filled with Silicon oil (Dow Corning 200/50) in layers of approximately 67 mm (1/3 height of the mould). Then the sand mix was gently spooned in to the mould in thin layers and levelled without vibration or any type of method that can lead to compaction. This procedure was repeated until the mould was filled.
3. Steps 5 to 13 of the procedure for preparing loose rotund sand specimens in the technique 2a were then repeated.

For the preparation of the 'mica-coarse rotund sand mixture' specimens **(4)**, the technique described as below was followed;

1. Required amount of rotund and platy sand/sucrose particles were weighed (5%, 10% or 15% by weight). The specimen was mixed and then submerged in de-aired water and stirred with a spoon to obtain a uniform mixture.
2. The circumference of the pedestal was coated with a thin layer of silicon grease to ensure a watertight connection between the o-rings and membrane. The membrane was then attached to the pedestal with two o-rings.
3. The three-part split mould was placed around the pedestal to retain the shape of the specimen during construction and to support the specimen until enough suction was applied. The top of the membrane was then folded back over the mould.
4. The mica and coarse rotund sand mix was as gently as possible spooned in to the mould in thin layers without vibration or any type of method that could lead to compaction. Great care was taken to ensure that the mix remained submerged in de-aired water at all times. This procedure was repeated until the mould was filled.
5. When the mould was completely filled, the top of the specimen was flattened by gently sliding the sand particles from the centre of the specimen to its side using the tip of a small screwdriver.
6. An o-ring stretcher with two o-rings was slipped over the top plate which was placed on top of the specimen. The membrane was slipped over the top plate,

ensuring that no sand particle was trapped between the two before using the o-rings.

7. Approximately 15 kPa of vacuum was applied to the inside of the specimen. The vacuum was maintained by specifying a target pressure of -15 kPa to the GDS pressure controller, connected to the pedestal.
8. Once the pore pressure measured through the pedestal stabilized at this pressure value, the three-part-mould was carefully split to prevent disturbance to the specimen.
9. The dimensions of the specimen were measured in mm to two decimal places. The LVDT holders were glued to the membrane using super glue at the middle third of the specimen. The LVDTs were then placed with screw on the side of the holders. The gauge length of the LVDTs was adjusted to ensure the maximum linear range. This was done either by adjusting the screw. (if necessary) Elastic bands were placed around the brackets and cables for additional support.
10. The coil was assembled, and then the cell was closed up and filled with water at room temperature.
11. The vacuum inside the specimen was reduced while gradually increasing the confining water pressure until the desired value was achieved. The specimen under a constant temperature was left overnight to ensure that any air trapped in the specimen would dissolve into the pore fluid.
12. The loading was applied next day.

In addition to the preparation of the specimens to be tested in the triaxial apparatus, a series of sacrificial specimens were also made for relative density measurements of the specimens, as well as to judge the repeatability obtained from the followed technique. During the experiments, three different types of sucrose were used. Namely; Silver Spoon Castor Sugar, Royal Icing Sugar, Silver Spoon Icing Sugar. The specimens were constructed by following the technique corresponding the (1d). In that technique, a specimen was simply prepared at room temperature, then heated up to 60°C in one day and cooled down gradually to the room temperature, or below it using the chiller. However, here in the tests for sacrificial specimens, no loading, apart from 20 kPa vacuum, was employed; change in temperature was not applied, loading was not applied. The results are given in Table 3.4. The table shows the description of the materials used, method followed, dimensions of the specimens at the end of the test, mass (wet, dry) of the specimens at the end of the test and the various densities. The sample preparation method, including for the sacrificial specimens, and the test procedure followed during all tests reported here was as follows;

- The required amount of dry sand, sucrose, and water were weighed.
- The sucrose grains and sand were mixed with water at room temperature. The mixture was then stirred by a spoon to obtain a uniform mixture.
- A membrane was attached to the pedestal using o-ring, and a three-part split mould was placed around the endplate.
- A three-part split mould was placed around the pedestal to retain the shape of the specimen during construction and to support the specimen until suction was applied. The top of the membrane was then folded back over the mould.
- The mixture was gently spooned in to the mould in thin layers without vibration or any type of method that can lead to compaction. Great care was taken to ensure that the mix remained submerged in de-aired water at all times. This procedure was repeated until the mould was filled.
- When the mould was completely filled, the top of the specimen was flattened by gently sliding the sand particles from the centre of the specimen to its side using the tip of a small screwdriver.
- The o-ring stretcher with two o-rings was slipped over the top plate which was placed on top of the specimen. The membrane was slipped over the top plate, ensuring that no sand particles were trapped between the two before using the o-rings.
- Approximately 20 kPa of vacuum was applied to the inside of the specimen. The vacuum was maintained by specifying a target pressure of -20 kPa on the GDS pressure controller connected to the pedestal.

- Once the pore pressure measured through the pedestal stabilized at this pressure value, the three-part-mould was carefully split to prevent disturbance to the specimen.
- The dimensions of the specimen were measured in mm to two decimal places.
- The dry weight of the specimen was calculated by subtracting the dry weight of the sand remaining from the original dry weight of the sand weighed before the sample preparation.

3.6 Test procedure and experimental results

Isotropically consolidated undrained triaxial compression tests were conducted on specimens prepared according to the procedures outlined in the preceding section to determine the shear behaviour of the mixtures. Tests in this investigation were applied to fully saturated samples sheared in consolidated-undrained triaxial compression. During the consolidation process, the pore-pressure, cell pressure, volume, strain measurements as well as the temperature reading were closely examined and recorded. Tests were carried out at 100 kPa effective consolidation stress (400 kPa cell pressure and 300 kPa back pressure). Care was taken to ensure that the effective consolidation pressures were achieved by raising the cell pressure in coordination with the back pressure. The load ram is finally brought into just above with the sample using hand and zero of the strain dial is set. Following the consolidation at various temperature values and these last processes, the drainage valve to the specimen were closed, and then compressive load was applied using the load frame. The rate of loading of 0.015 mm/min at which the tests were run was governed by 10 second-logging rates. The test was usually terminated when the maximum deviatoric stress has clearly been reached.

Examples of typical test results obtained in the experimental study are shown in Figures 3.22-3.25. The selection of the test results being presented here have been divided into categories mainly depending on the specimen preparation techniques described in the previous section. Figures 3.22 and 3.23 illustrate the development of deviatoric stress and excess pore pressure with local axial strain for a specimen with sucrose (castor) solution tested at 60°C, and a specimen with sucrose (icing) solution tested at room temperature, respectively. The test result shown in Figure 3.22 was obtained by following method 1a, in which a specimen was prepared and sheared at 60 °C at the end of an isotropic consolidation procedure. In the test result shown in Figure 3.23, sufficient icing sugar, water and coarse rotund sand particles were mixed in a laboratory environment and placed on to the pedestal to be tested by mainly following method 1d, in differing any change in temperature. Figure 3.24 presents the effect of icing sugar and de-aired water as pore fluid on stress paths of coarse rotund sand specimens isotropically consolidated at 100 kPa effective consolidation pressure. The undrained secant Young's modulus of the specimens was also derived from local strain and effective pressure measurements. Figure 3.25 shows the relationship between undrained secant Young's modulus and axial strain of loose coarse rotund sand (Leighton Buzzard) with silicon oil and de-aired water specimens isotropically consolidated to 100 kPa effective consolidation pressure.

3.7 Oedometer Tests

An investigation of oedometer (one-dimensional-consolidation) tests was also conducted aiming to provide a general understanding of influences of mica on the compressibility of a Leighton Buzzard Sand. The experimental work described here was directed mainly towards an investigation of different percentage of mica (i.e., 5%, 10%, 15%, and 20%) and Leighton Buzzard Sand, fraction B, used in combination as blending materials by weight on water demand. Clean Leighton Buzzard Sand specimens at various densities were also tested.

The consolidation test method carried out during the investigation is the standard method of measuring consolidation properties, which involves the incremental loading of soil specimens. Incremental loading is to apply daily increments of vertical load to a submerged container in a rigid ring, with drainage permitted through porous stones at the bottom and top. The load is doubled each day, that is, the ratio of load increment to existing load is usually 1.

Table 3.1 Properties of mica used in the experimental study (Theron, 2004).

Supplier	Dean and Tranter Ltd.
Particle size	100 Mesh (52- 10 μm)
Description	Dry ground muscovite mica powder
Geological classification	Muscovite
Particle shape	Flat, platy
Sphericity	Flaky
Roundness	Low
Colour	White
Specific gravity	2.9
Maximum dry density (kg/m^3)	916
Minimum dry density (kg/m^3)	728

Table 3.2 Nutrition information of the sucrose solids used in the tests (per 100 g)

	Castor	**Icing**
Energy	1700 kj 400 kcal	1683 kj 396 kcal
Protein	0 g	0 g
Carbohydrates	100 g	99.4 g
Fat	0 g	0 g
Tricalcium phosphate	not available	available

Table 3.3 Some properties of the pore fluids used during the experimental study.

Substance	**Viscosity (mm^2/s)**	**Density (g/ml)**	**Approximate temperature at the tests conducted (^oC)**
Water (de-aired)	0.942	0.9738	23
0.5 g sucrose / 100 g water	0.997	0.9737	23
5 g sucrose / 100 g water	1.075	0.9927	23
25 g sucrose / 100 g water	1.580	1.0564	23
50 g sucrose / 100 g water	2.897	1.1146	23
100 g sucrose / 100 g water	9.345	1.1896	23
150 g sucrose / 100 g water	37.120	1.2573	23
207 g sucrose / 100 g water	134.151	1.2924	23
Silicon oil (Dow Corning 200/50)	50.983	0.9165	23

Table 3.4 A brief explanation showing the sacrificial test results.

Name of test	**Material**	**Amount of sugar g/100g water**	**Method followed**	**Dimensions of the specimens (mm)**	**Mass (g) wet, dry**	**Bulk density (g/cm^3)**	**Dry density (g/cm^3)**	**Relative density of sand (%)**
SS¯1	LB	207	At constant temperature	99.13x207.93	3134.45, 2500.10	1.954	1.559	34.7
SS2	LB	207	At constant temperature	99.1x206.74	3122.66, 2488.00	1.959	1.561	34.6
SS3	LB	207	At constant temperature	98.98x206.62	3098.98, 2468.40	1.950	1.553	31.7
SS4	LB, SSIS£	207	At constant temperature	99.98x201.28	3318.23, 2536.5	2.100	1.606	52.5
SS5	LB, SSIS	207	At constant temperature	99.11x210.17	3370.33, 2592.7	2.079	1.599	50.5
SS6	LB, SSIS	207	At constant temperature	99.2x210.90	3415.21, 2616.6	2.103	1.612	55.4
SS7	LB, SSCS	207	At constant temperature	99.03x197.23	3150.74, 2394.5	2.075	1.577	41.6
SS8	LB, SSCS	207	At constant temperature	99.14x204.74	3248.34, 2477.2	2.056	1.568	37.6

†**LB**: Leighton Buzzard Sand, fraction B, °**SSCS**: Silver Spoon Caster Sugar, +**RIS**: Royal Icing Sugar, ¯**SS**: Sacrificial Specimen, £**SSIS:** Silver Spoon Icing Sugar.

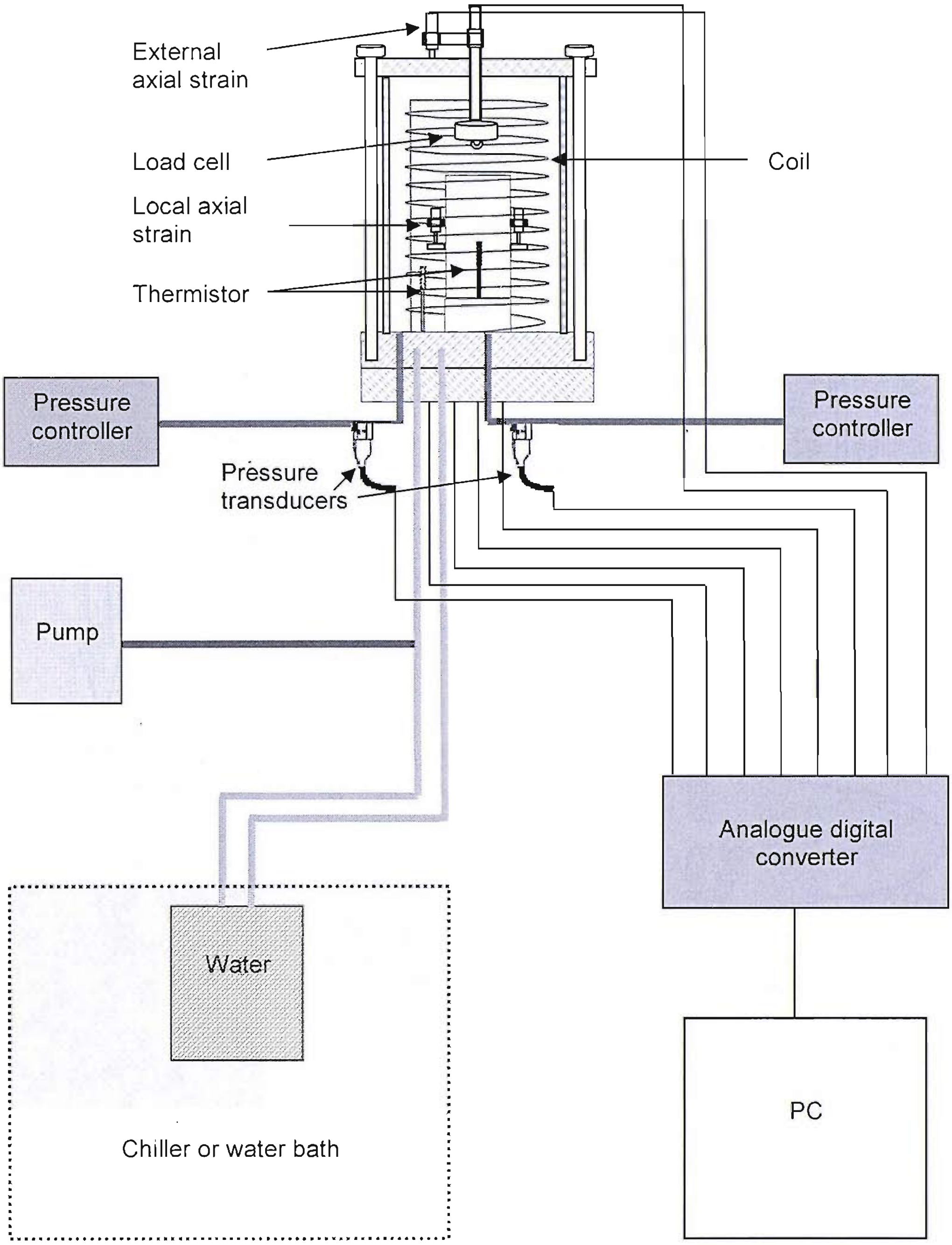
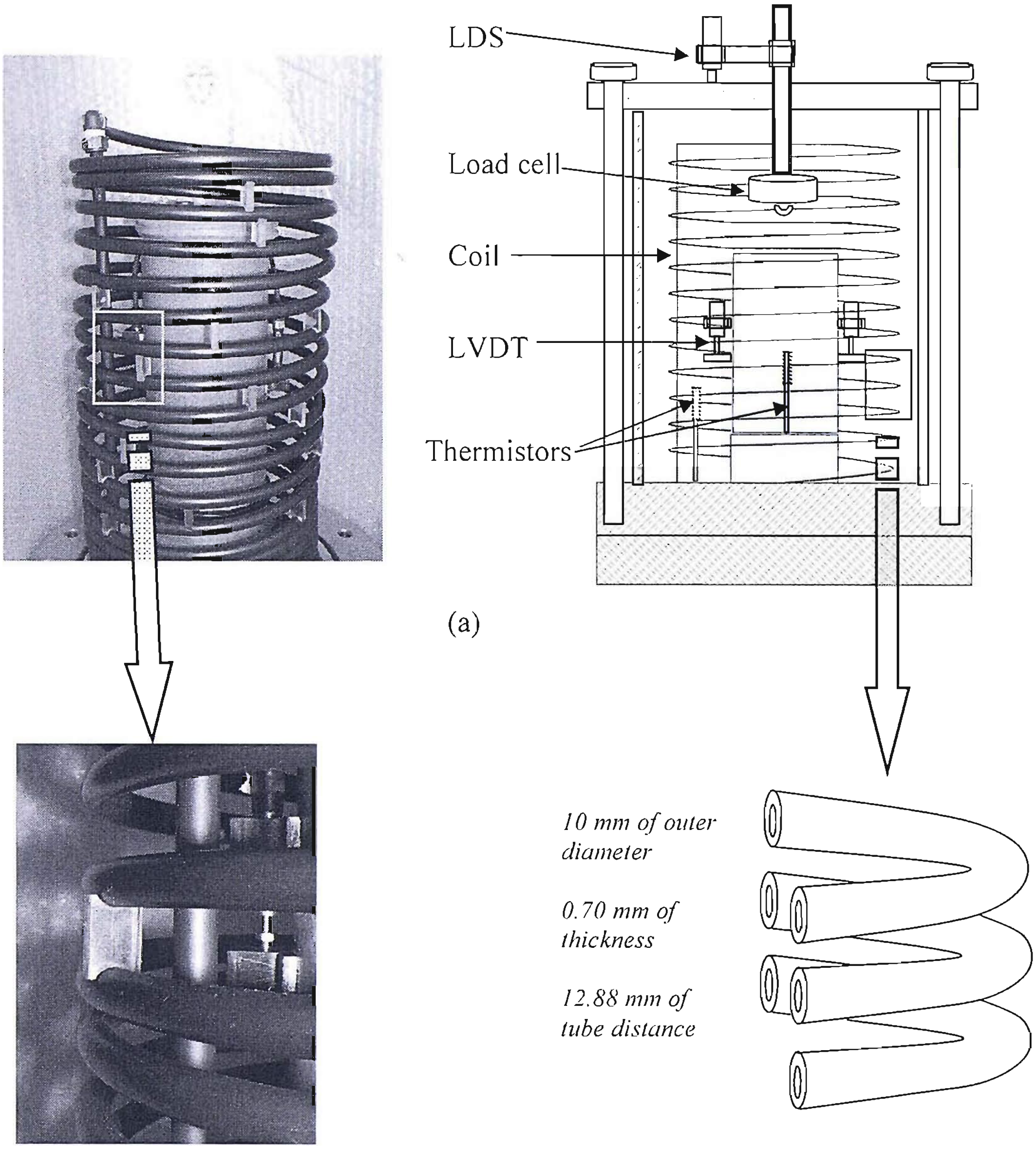


Figure 3.1 Schematic diagram of the set-up used during the experimental study.



(a)

(b)

Figure 3.2 (a) A specimen to be tested in triaxial compression test with temperature control system, (b) details of the coil made of copper.

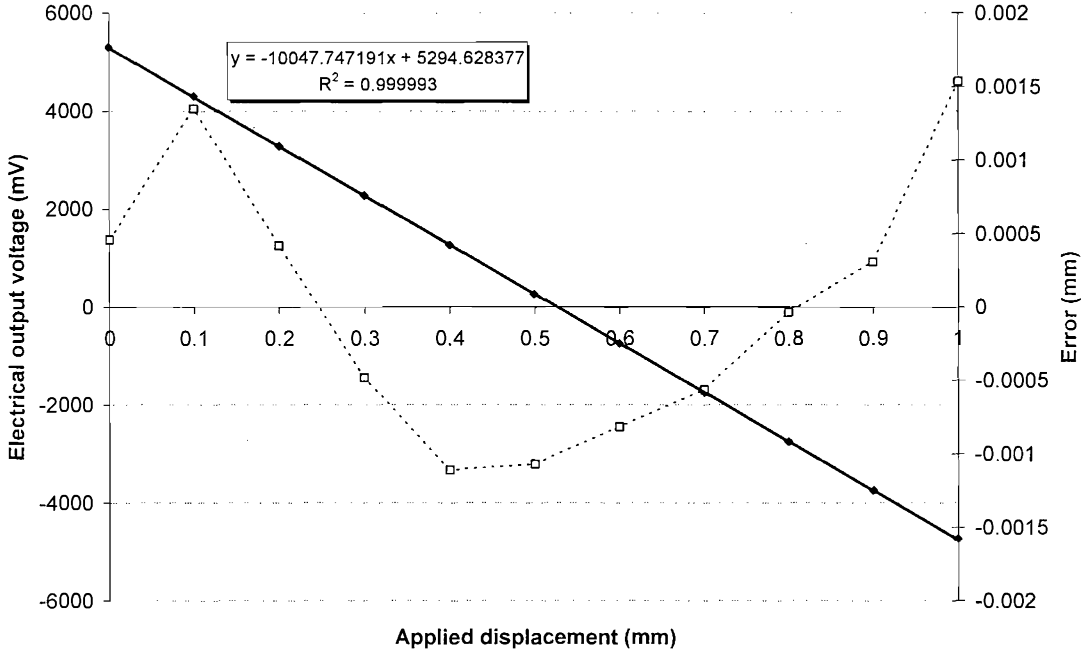


Figure 3.3 Calibration graph for the LVDT, RDP D5/40W/745 S/N 25706

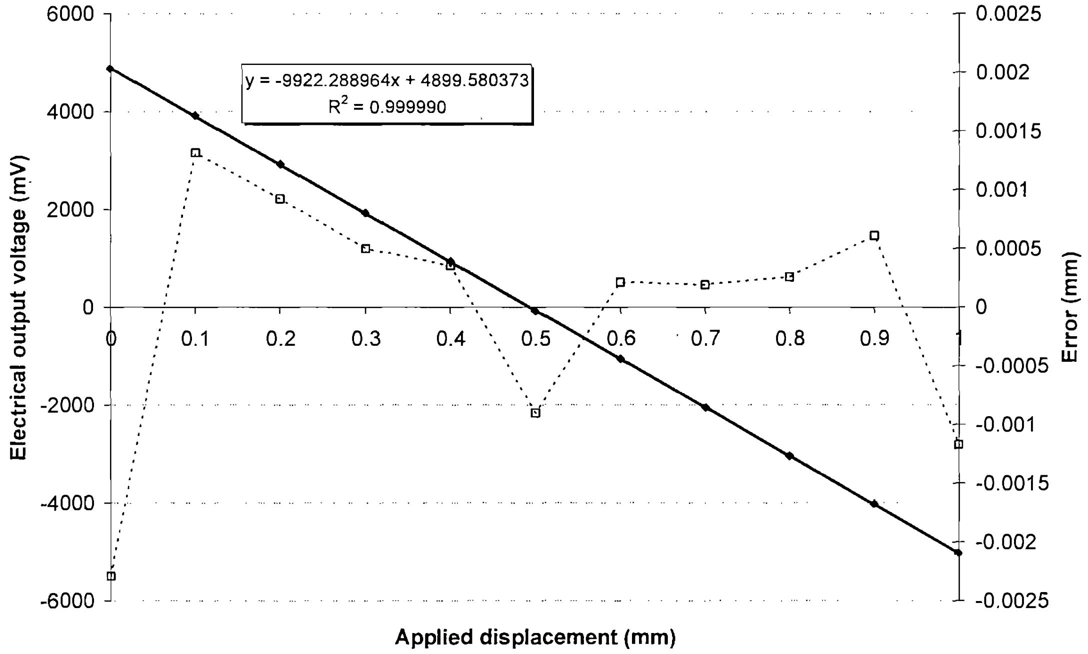


Figure 3.4 Calibration graph for the LVDT, RDP D5/40W/745 S/N 25708

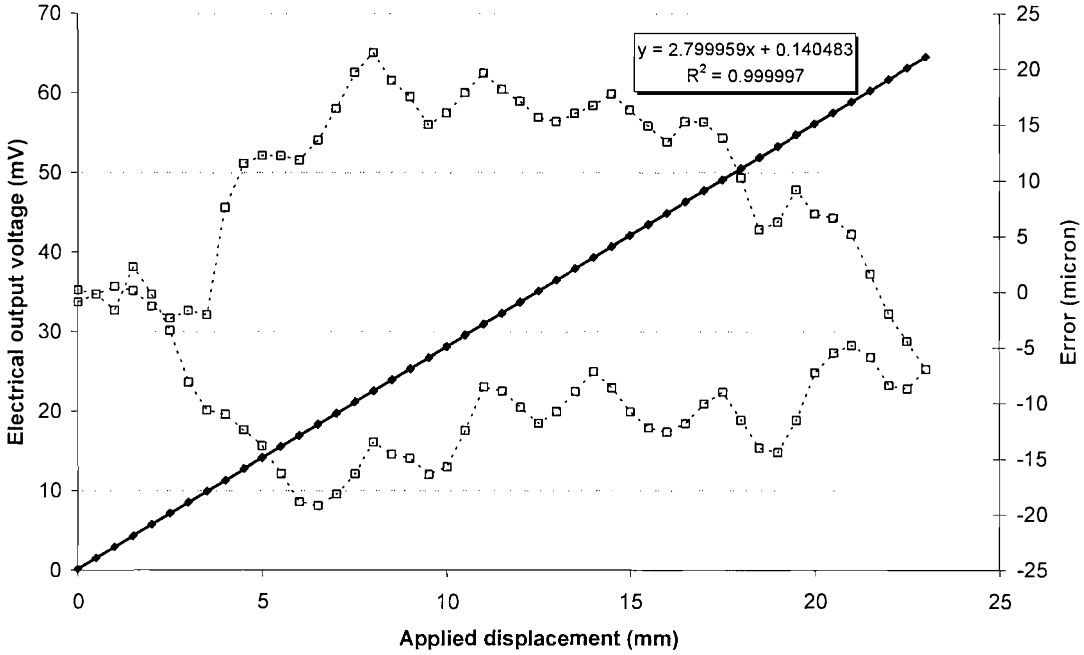


Figure 3.5 Calibration graph for the linear displacement sensor (LDS), HS25 MG2004

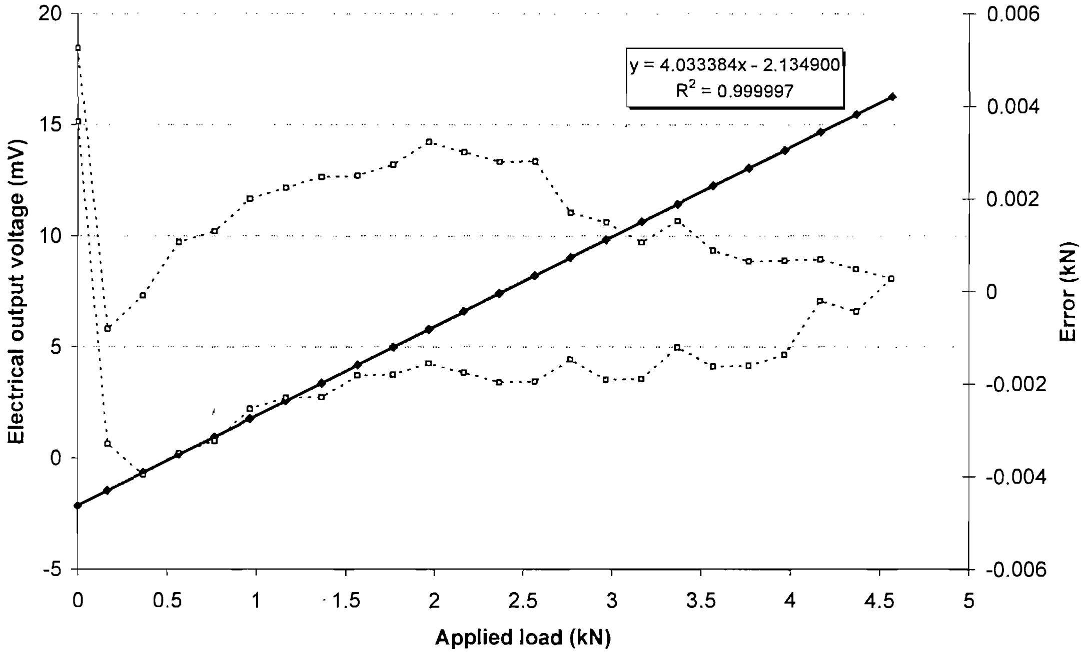


Figure 3.6 Calibration graph for the 5-kN load cell, 4958 Type S/N 8408858

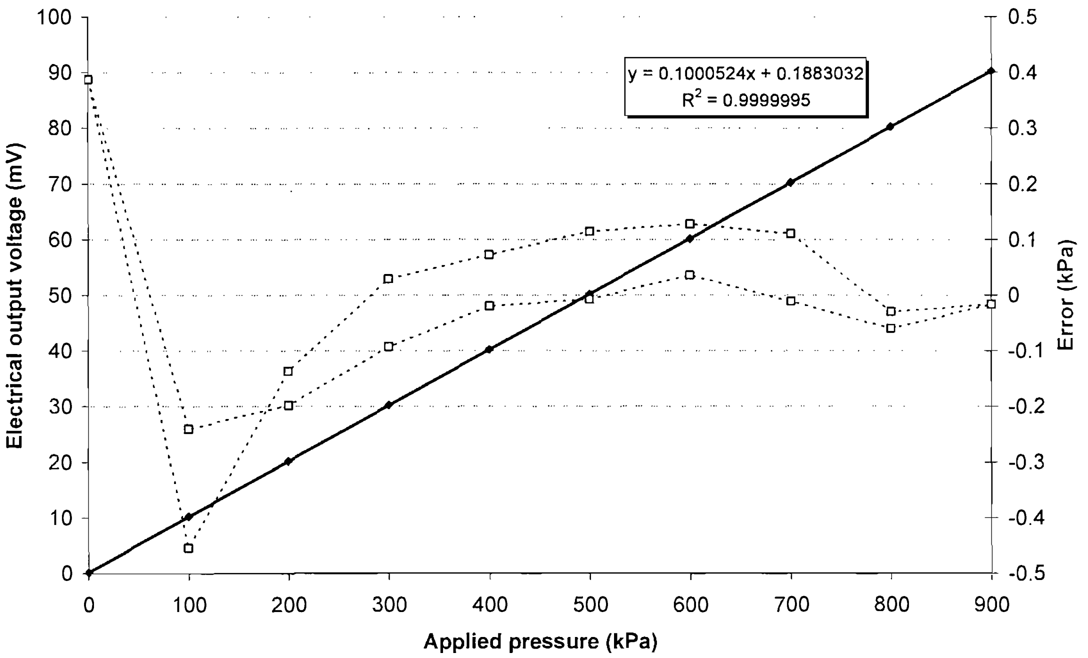


Figure 3.7 Calibration graph for the cell pressure transducer, PDCR 810 S/N 1912137

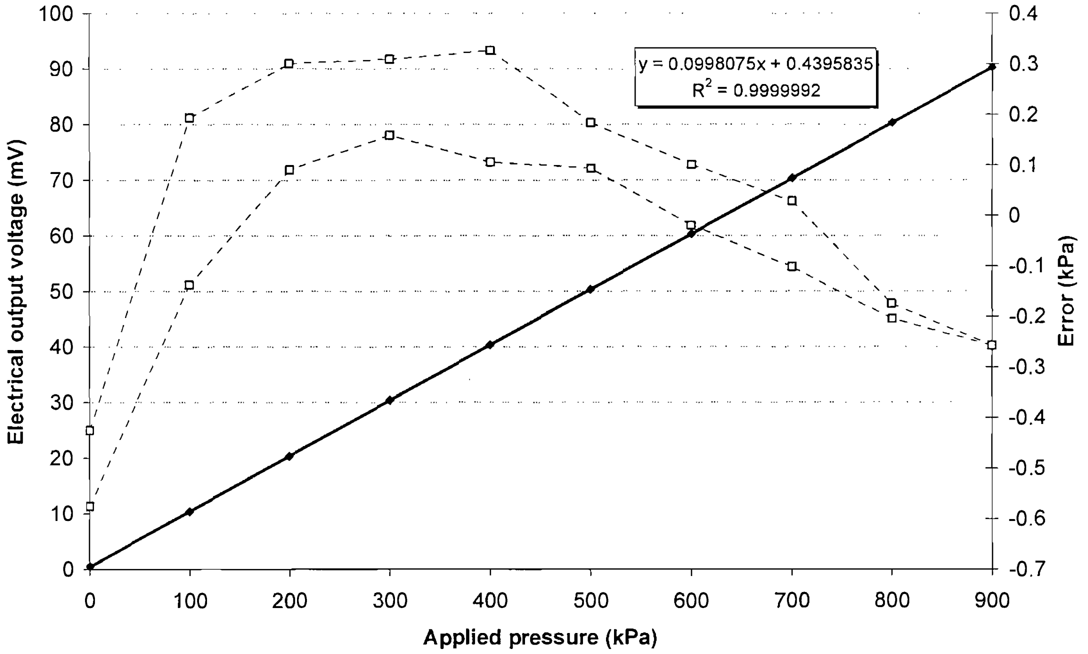


Figure 3.8 Calibration graph for the pore pressure transducer, PDCR 810 S/N 2024475

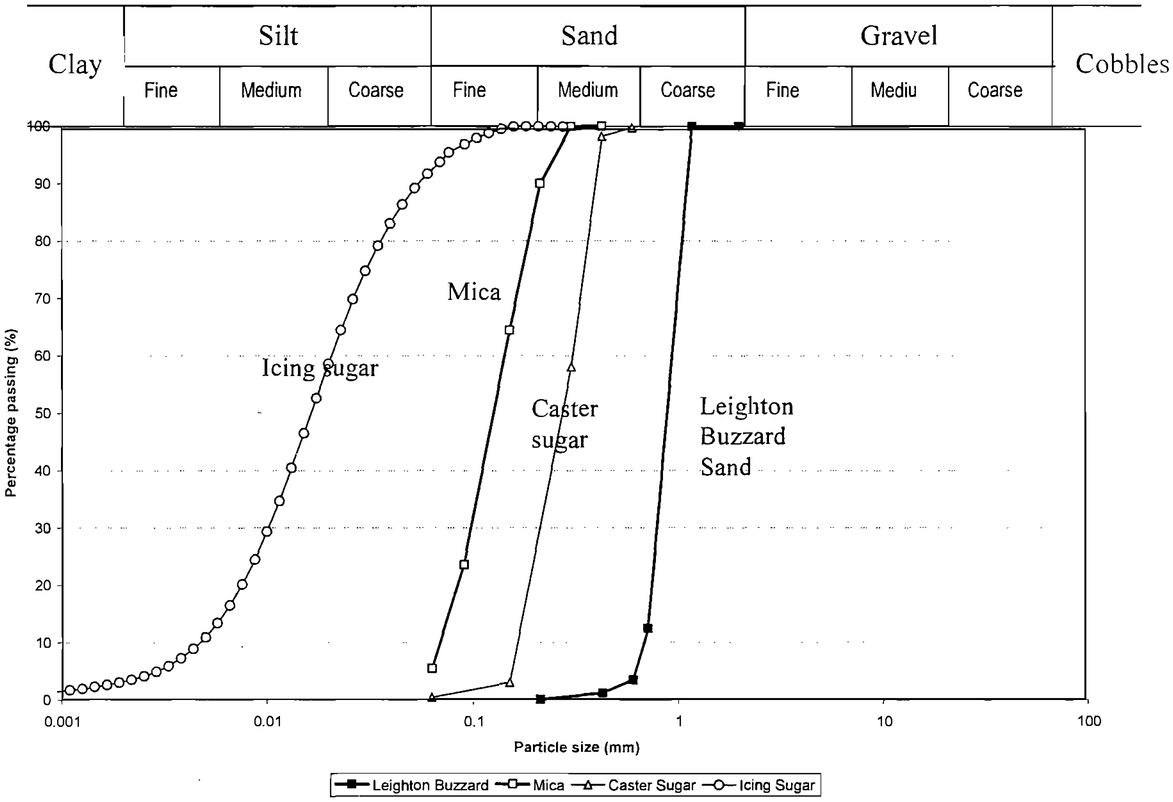


Figure 3.9 Particle size distributions for Icing Sugar, Mica, Caster Sugar, and Leighton Buzzard used during the test.

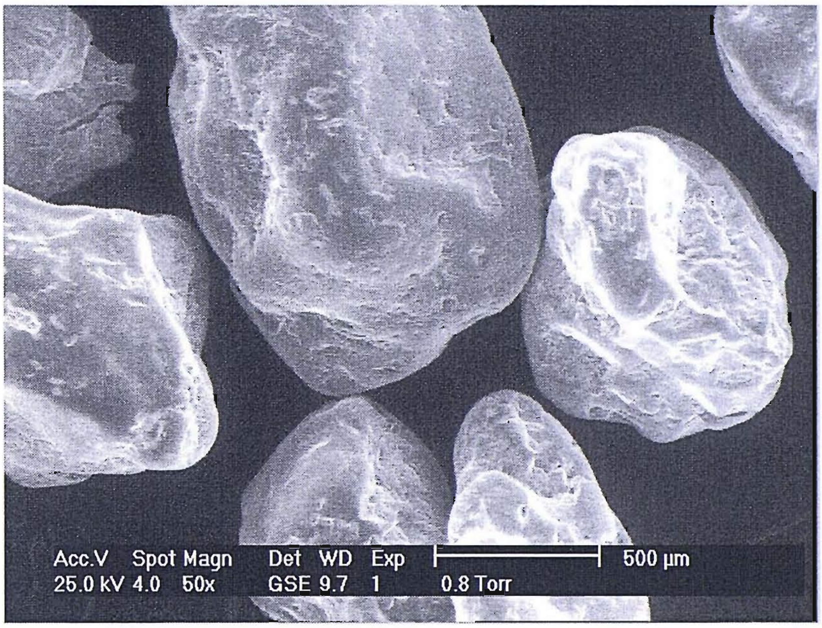


Figure 3.10 Scanning electron micrograph of Leighton Buzzard Sand B.

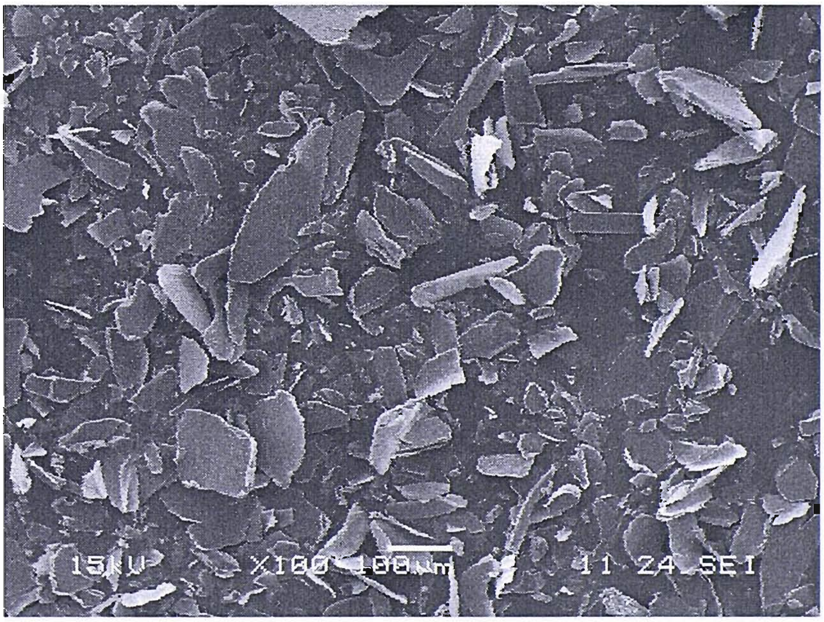


Figure 3.11 Scanning electron micrograph of the mica particles supplied by Dean and Tranter Ltd.

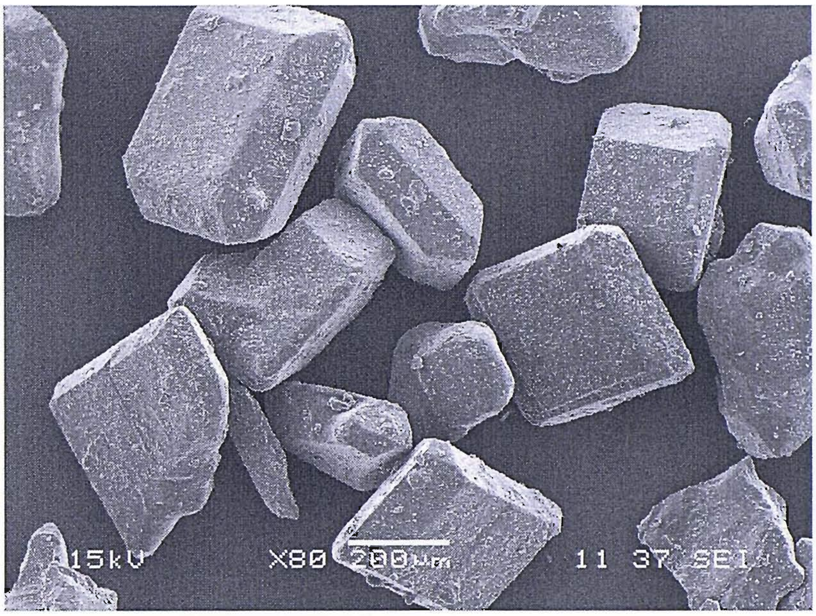


Figure 3.12 Scanning electron micrograph of castor sugar particles.

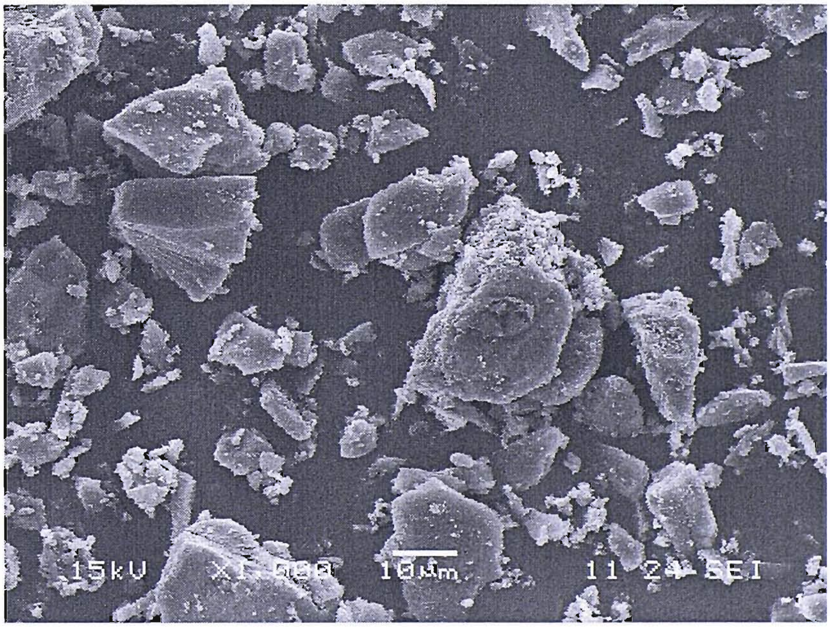


Figure 3.13 Scanning electron micrograph of icing sugar particles.

Figure 3.14. Scanning electron micrograph of castor sugar and mica particles

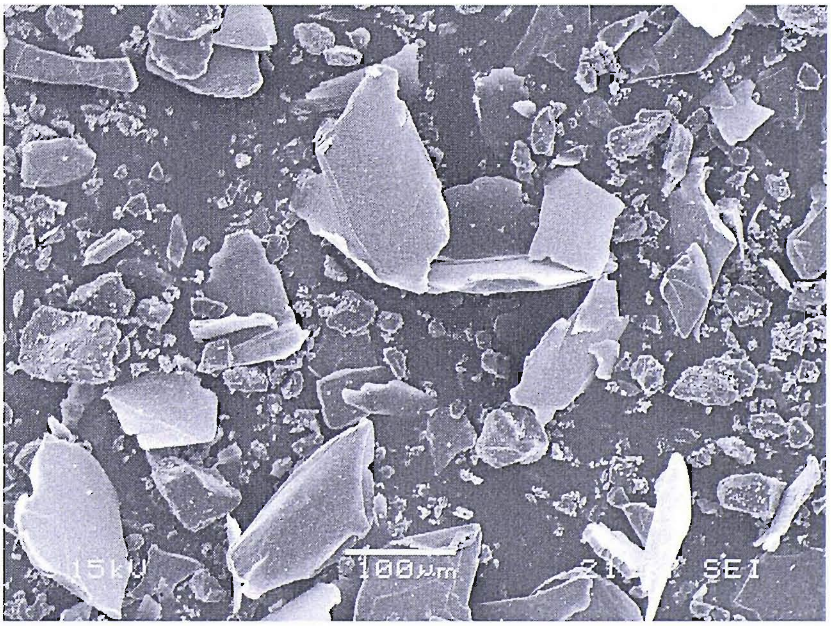


Figure 3.15 Scanning electron micrograph of icing sugar and mica particles

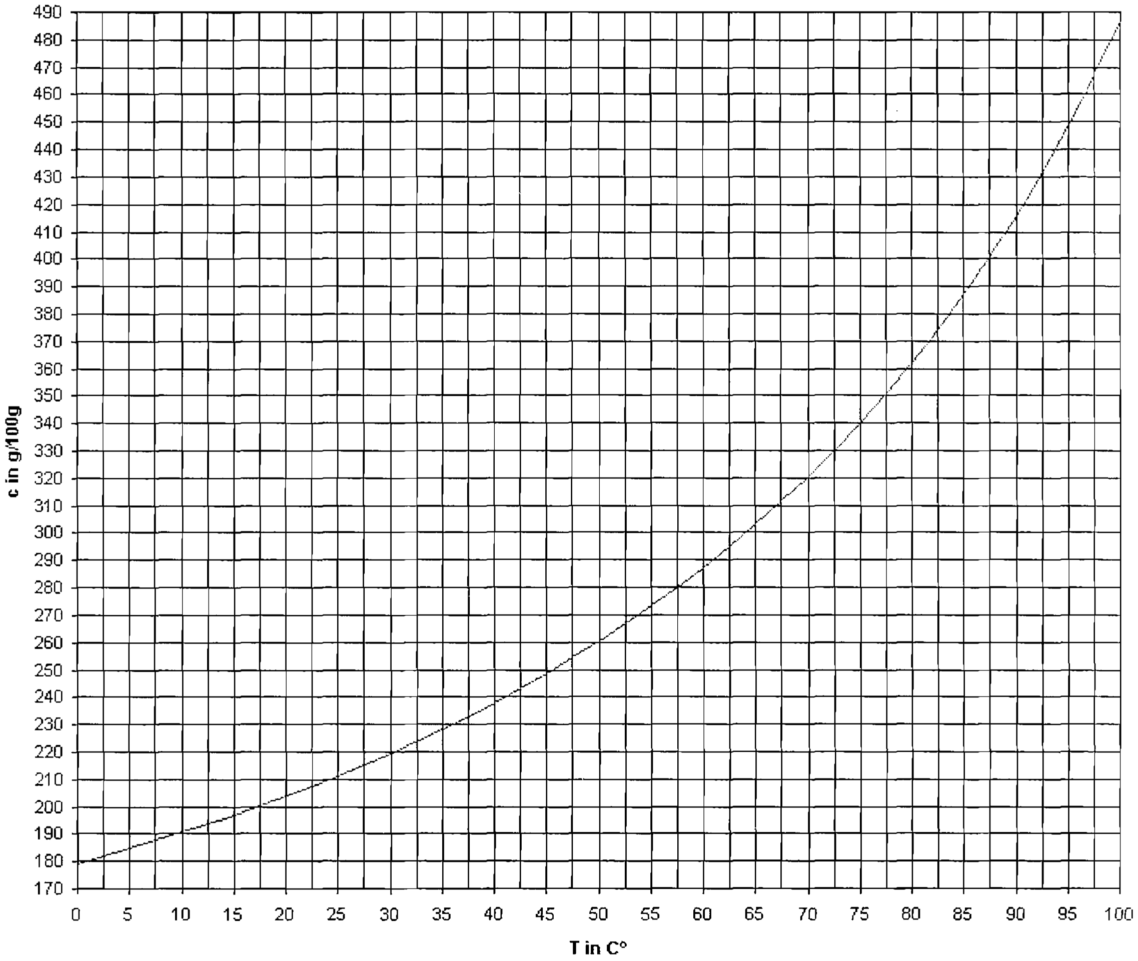
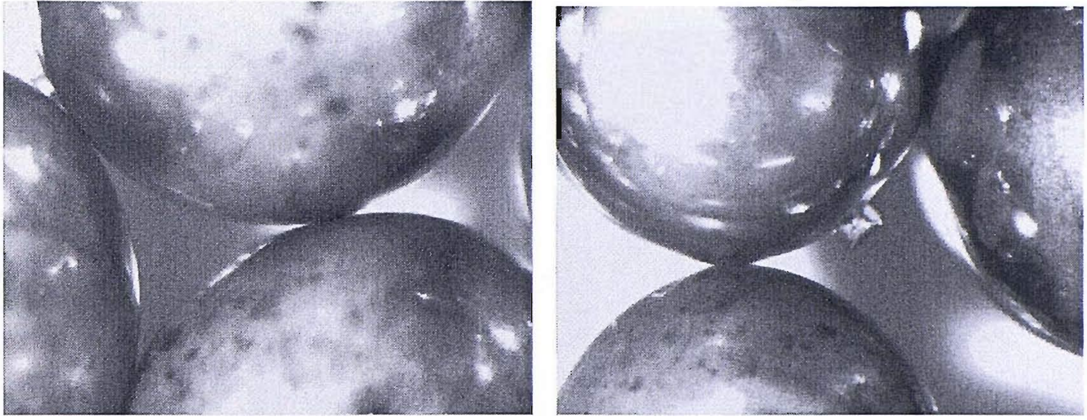
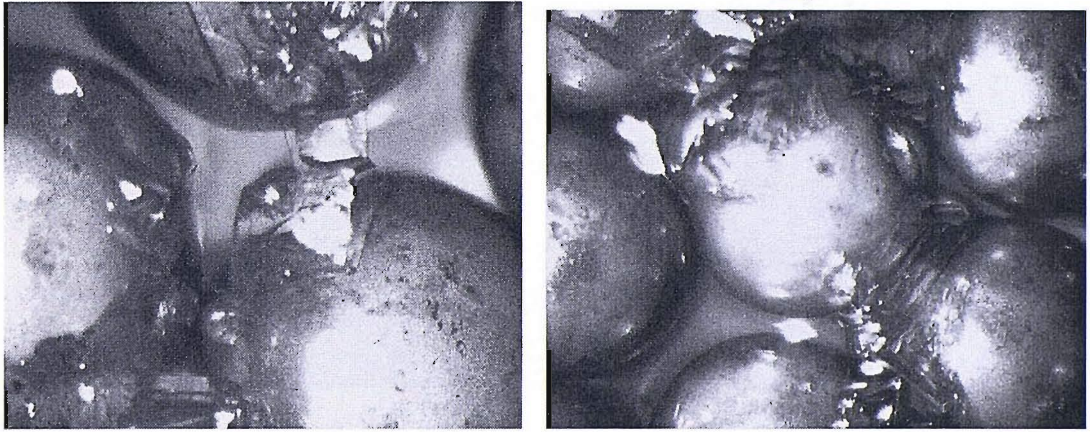


Figure 3.16 Solubility chart of sucrose (Behner, 1998).

Day 2 Day 4



Day 6 Day 8



Day 10 Day 11

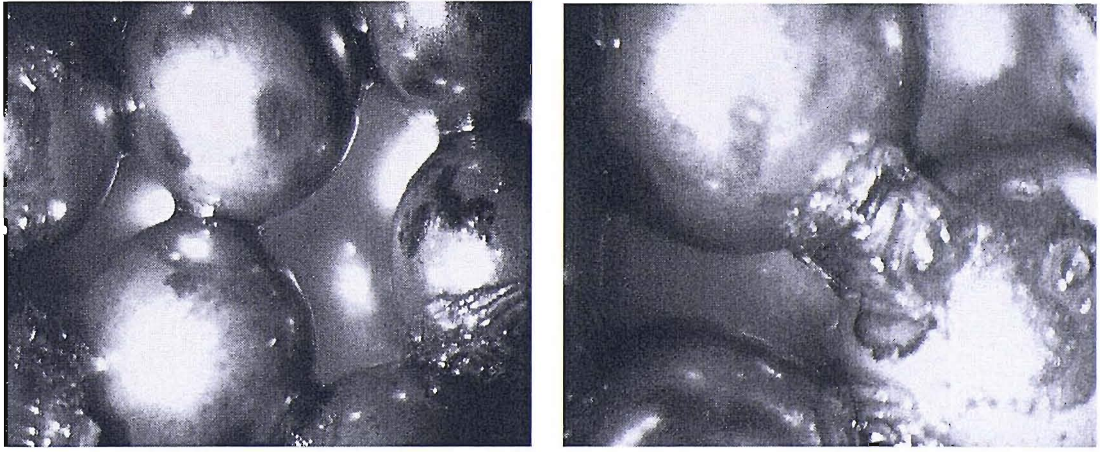


Figure 3.17 Typical examples of the crystal-growing process among the particles

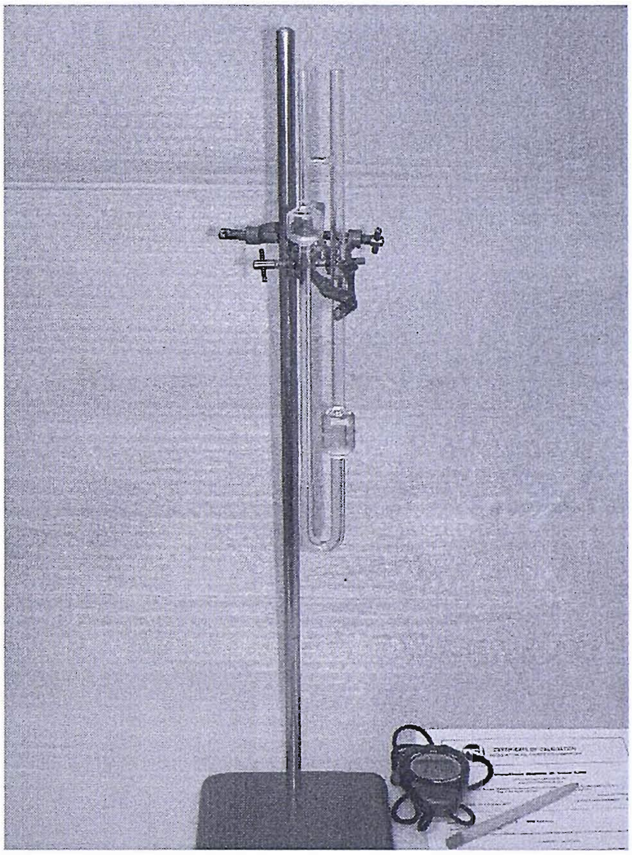
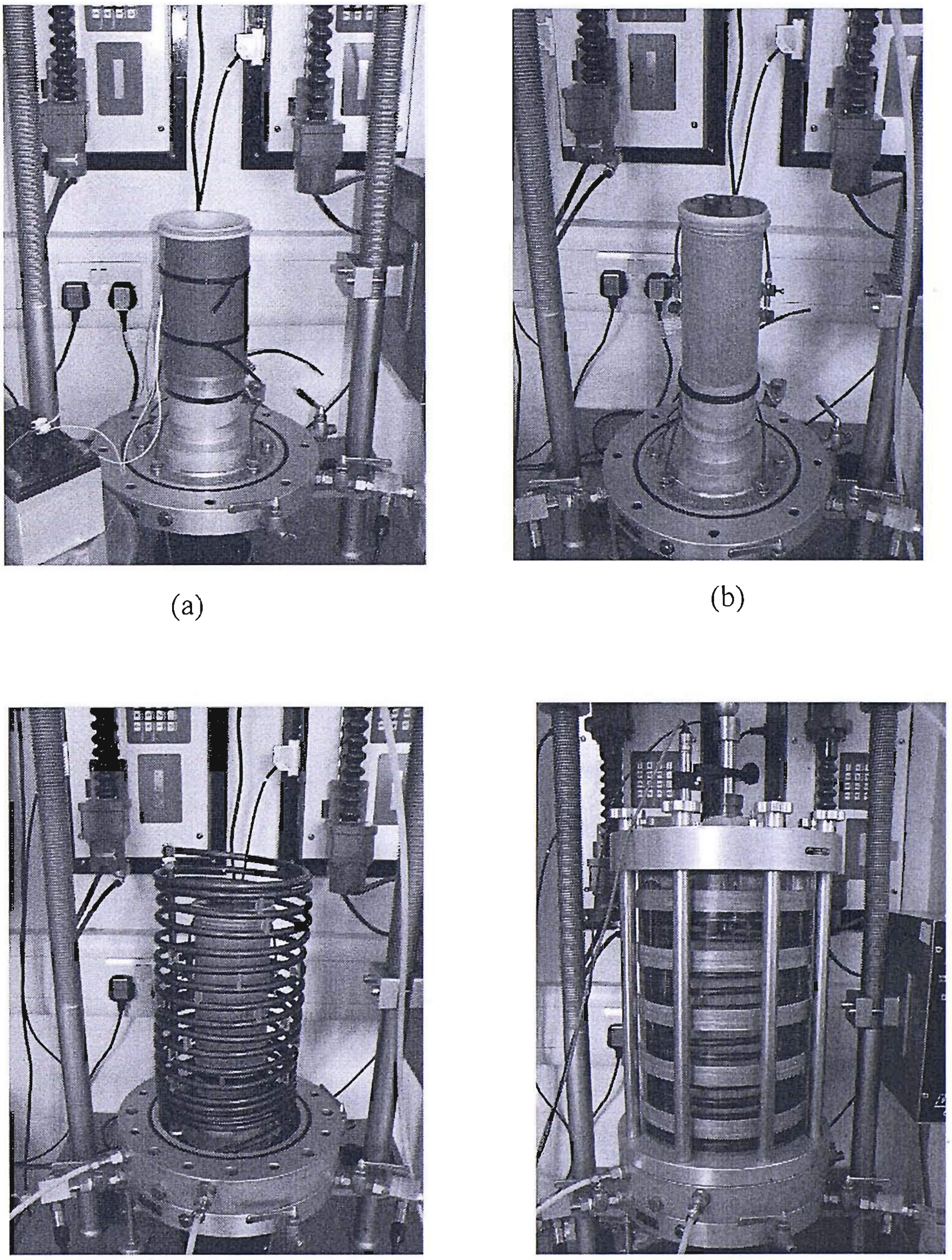


Figure 3.18 The viscometer (PPSL BS/U B VHB-320-070F produced by Poulten Selfe & Lee Ltd) used for the measurements.



(a) (b)

(c) (d)

Figure 3.19 Four-step-process for specimen preparation.

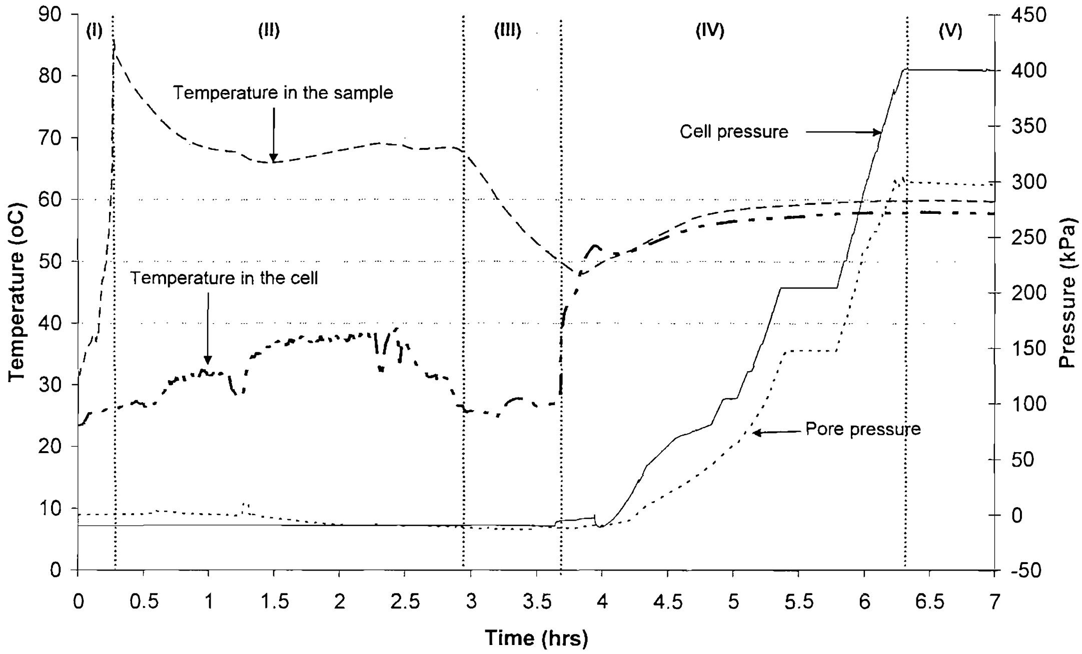


Figure 3.20 A typical plot showing the changes in temperature and pressure with time before testing a specimen in temperature controlled triaxial apparatus.

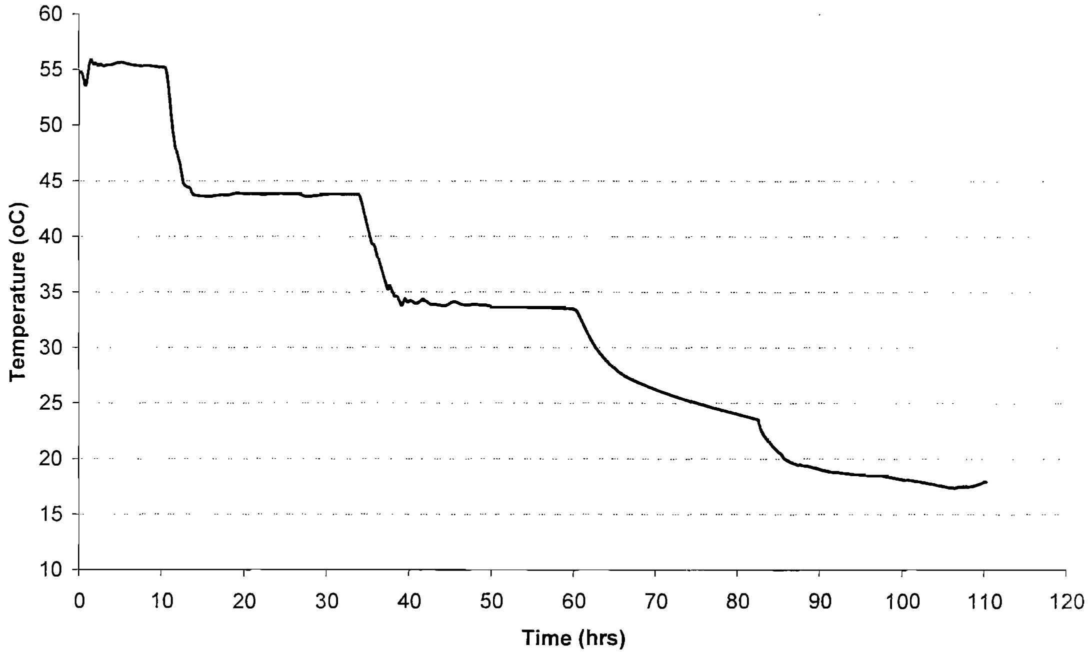
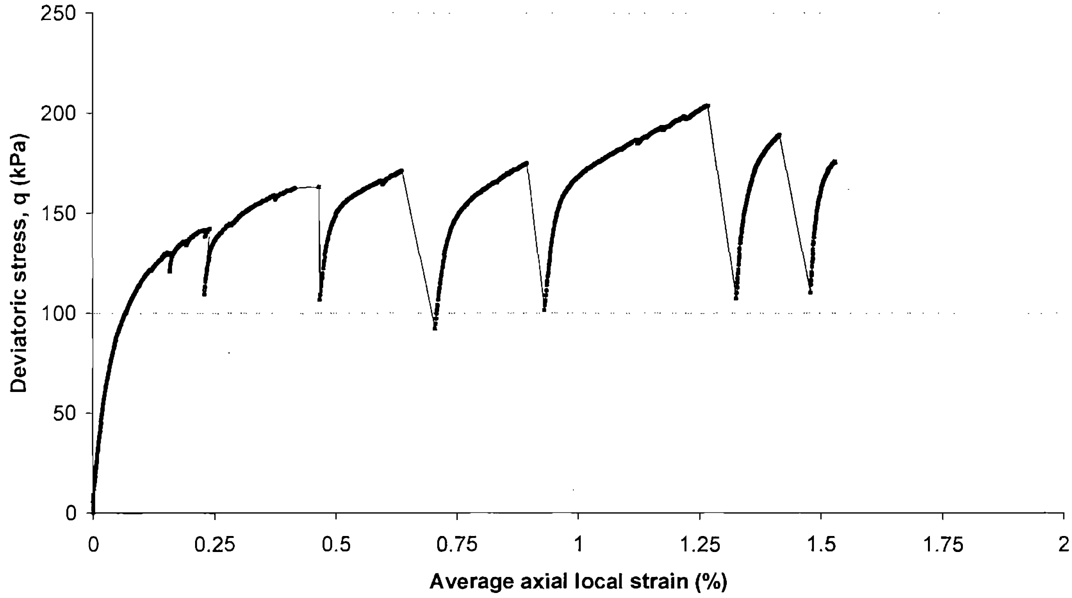
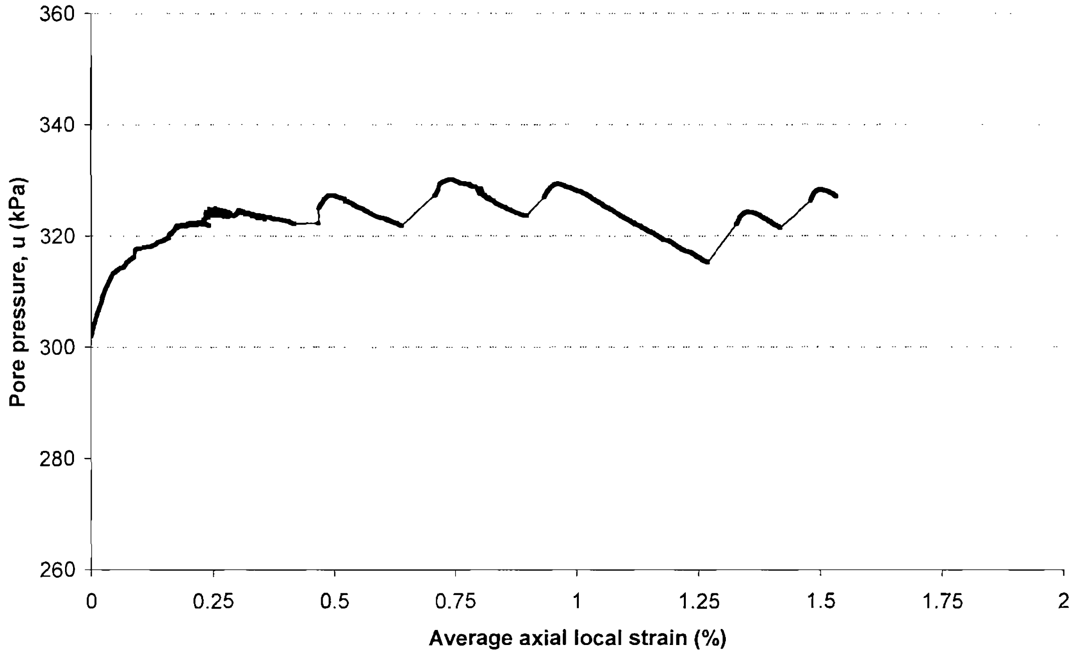


Figure 3.21 Change in temperature with time before testing a specimen.

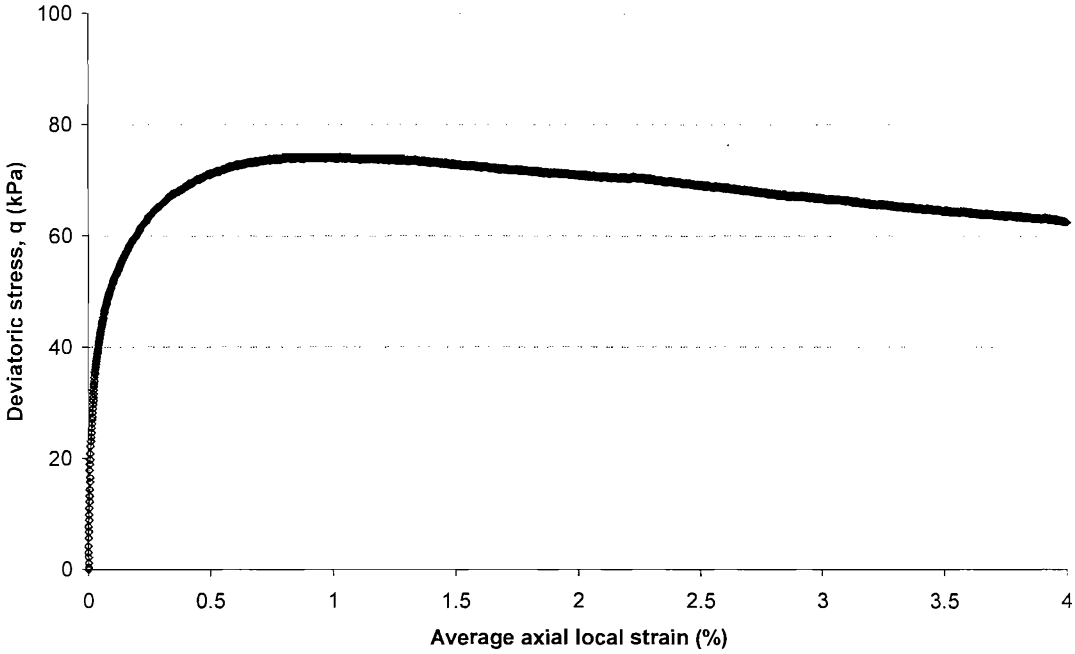


(a)

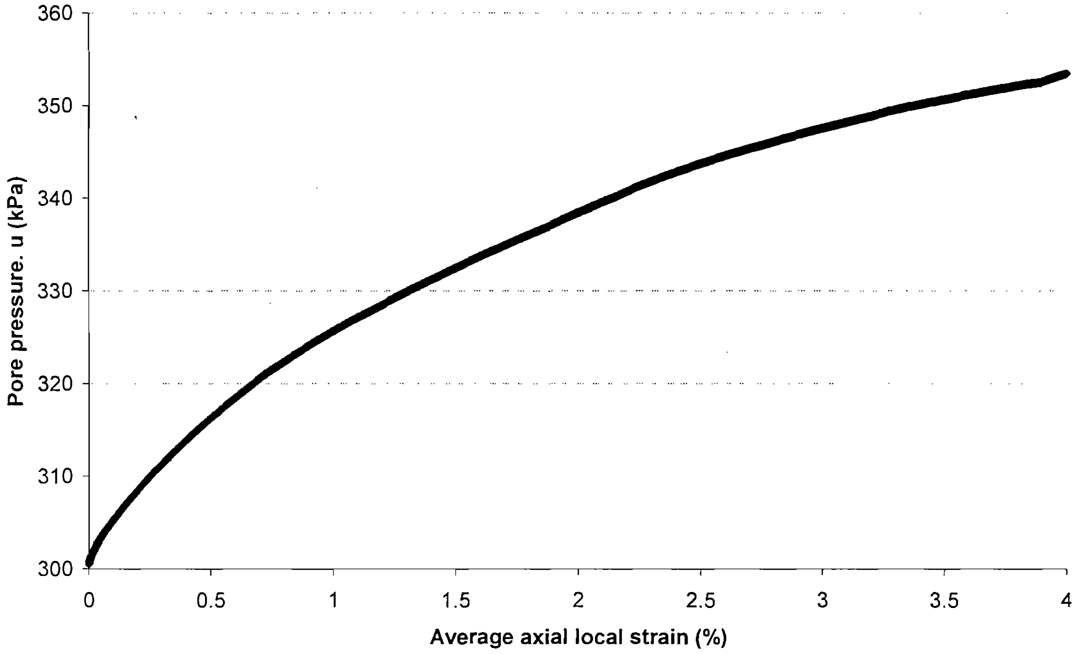


(b)

Figure 3.22 A typical example of (a) stress vs. strain, and (b) pore water generation plots for a Leighton Buzzard specimen with castor sucrose solution (mix ratio is 207g sucrose/100 g water) tested at 60°C.

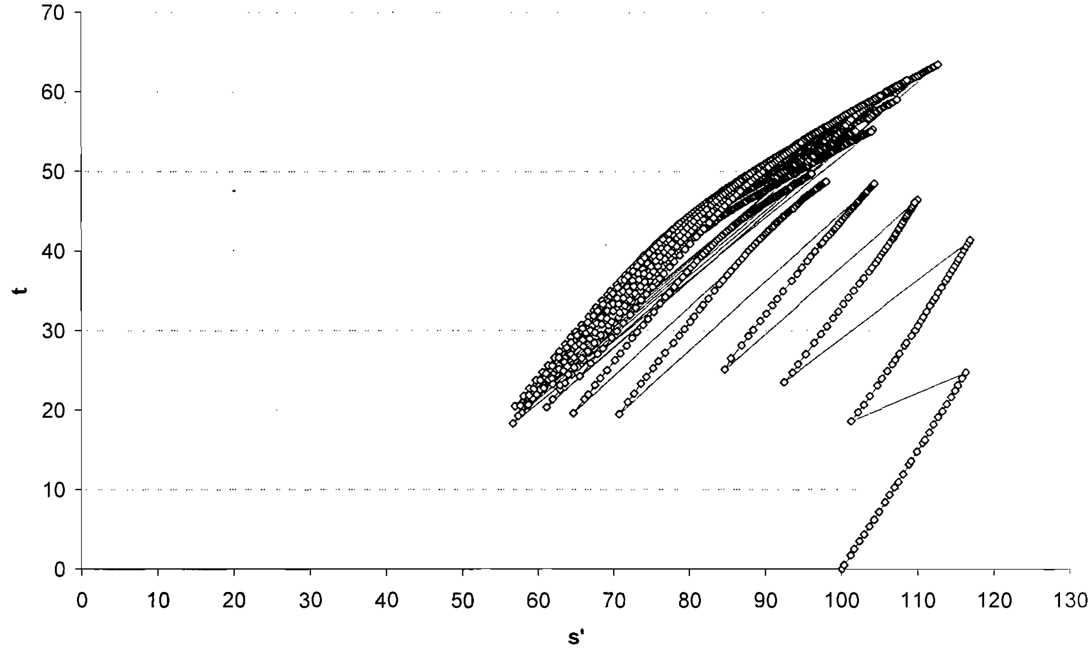


(a)

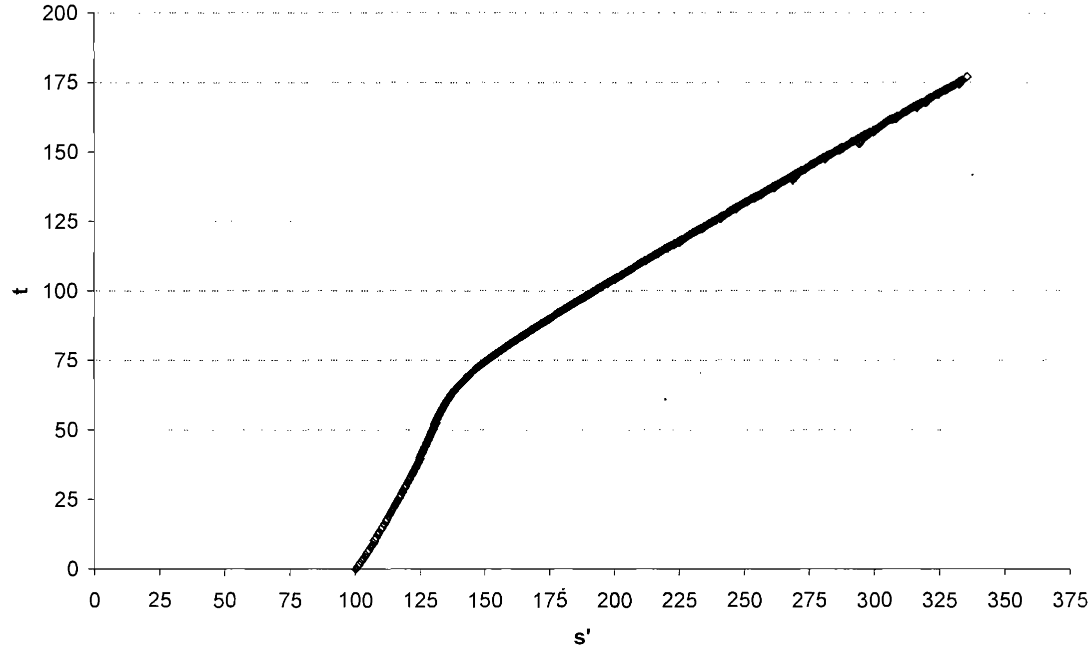


(b)

Figure 3.23 An example of (a) stress vs. strain, and (b) pore water generation plots for a Leighton Buzzard specimen with icing sugar prepared and tested at room temperature, without making any change in temperature.

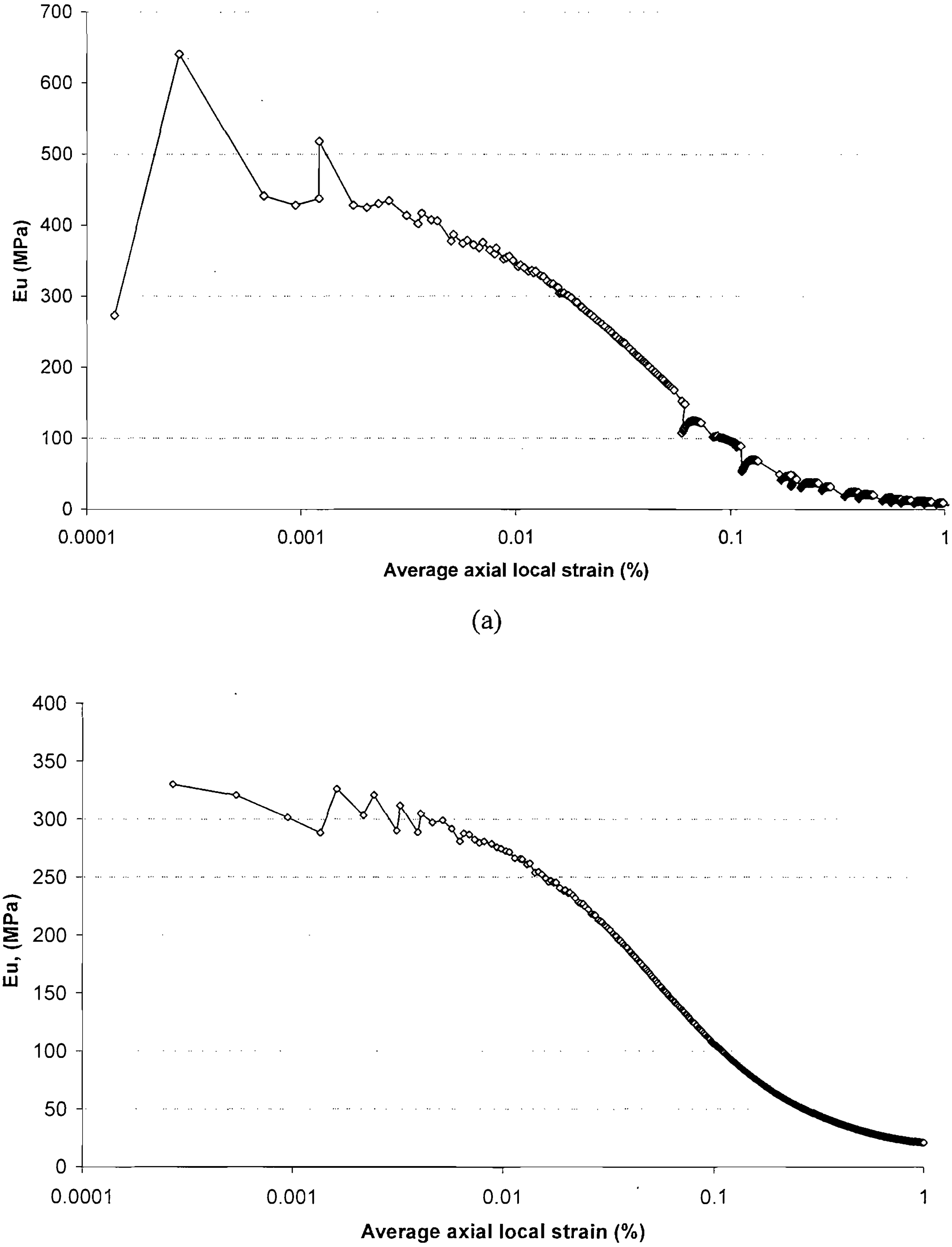


(a)



(b)

Figure 3.24 Example of stress space paths using MIT method (a) Leighton Buzzard and icing sugar induced specimen with the ratio of 207g sucrose/100 g water, (b) loose Leighton Buzzard Sand.



(a)

(b)

Figure 3.25 Undrained secant Young's modulus as a function of local axial strain of Leighton Buzzard Sand with (a) Dow Corning (200/50) silicon oil, (b) de-aired water as pore fluid, isotropically consolidated to 100 kPa effective consolidation pressure.

CHAPTER 4

DISCUSSION

A literature review has been performed (i) to develop a framework for the mechanical behaviour of sand, (ii) to investigate the influences of platy fines, (iii) to review the influences of bonding on the geomaterials, and (iv) to study the stick-slip instabilities in granular samples (Chapter 2). A series of experiments have been performed on various sucrose-Leighton Buzzard Sand mixtures using the temperature controlled triaxial apparatus that has been modified with a motivation by the lack of a thorough experimental investigation to generate more understanding of the influence of structural behaviour of synthetic geomaterials. In this section, data of the preceding sections are analysed and discussed.

4.1 Observed effects of sucrose solutions as pore fluid on the behaviour of Leighton Buzzard Sand

The effect of pore fluid on the stiffness, strength, and pore pressure characteristics of a sand was investigated to assess the potential contributions to microstructural behaviour for soils with various pore fluids. The study primarily investigates the influence of interparticle attractions by creating artificial bonding through sand particles under controlled pressure and temperature conditions using sucrose ($C_{12}H_{22}O_{11}$).

The influence of sucrose solutions on the mechanical behaviour of coarse rotund particles was one of the research subjects, which was assessed by studying effects of the change in amount of sucrose (i.e., castor sugar) on the behaviour of Leighton Buzzard Sand specimens. The experimental study presented here in this section primarily aims to provide a basis for the analysis of viscosity effects that change with the amount of sugar. Table 4.1 summarizes the bulk density, dry density, relative density, and void ratio of the specimens tested at an effective consolidation pressure of 100 kPa as well as the viscosity measurement values. Maximum and minimum dry densities used in the calculations were 1.74, and 1.48 Mg/m^3 respectively. The table also shows the viscosity tube measurements and density measurements of the pore fluids used in these series of tests, which are sucrose solutions at various concentrations and de-aired water. Bulk densities were calculated using the measured specimen weight and dimensions. The relative densities given in this table refer to the Leighton Buzzard Sand only. Comparing the relative density of the specimen prepared in de-aired water with those proposed by Terzaghi and Peck (1948) as shown in Table 4.2, it can be seen that the materials tested could almost exclusively be classified as medium dense.

The addition of 5 g sucrose (i.e., castor sugar) per 100 g water slightly increased the bulk density. This was because of the fact that the sucrose dissolved in water fill the pores between the sand particles without changing the volume of the specimen. The addition of sucrose dissolved in the water increases the relative density as well as bulk density and dry density. Leighton Buzzard Sand with sucrose solution is in a relatively denser state comparing to the loose Leighton Buzzard Sand specimen with de-aired water. This increase could be mainly attributed to the influence of sucrose solution that acts as a lubricant between soil particles, and so substantially increase the bulk density, dry density and relative density of the specimens tested.

As can be seen from the Table 4.1, an increase in sucrose quantity at every ratio resulted in an overall increase in bulk density, dry density and relative density, and decrease in void ratio. Adding of sucrose may affect the structure of specimens. In order to establish how this is brought about, the change in fabric of the specimens tested was investigated.

The addition of sucrose may change the way energy is used during deposition of the specimen. To understand this behaviour one should understand the internal structure of the specimens tested. The addition of a heavy pore fluid between the Leighton Buzzard Sand particles increases the bulk density of the specimens without changing their volume. The void ratio decreases while the bulk density and relative density increase. All the specimens tested using different concentrations satisfy this condition.

The deviatoric stress vs. strain, and pore pressure generation vs. strain behaviours of the specimens with sucrose solutions changes with the amount of sucrose (i.e., castor sugar) added to the solution. The Leighton Buzzard Sand particles are assumed to be in clean contact with each other in the specimen tested with de-aired water, and hence the mechanical behaviour of this specimen was governed by the Leighton Buzzard Sand particles only. This experiment result presents a good agreement with Theron (2004). As the amount of sucrose increases, the behaviour of the specimens moves away from the Leighton Buzzard Sand with de-aired water.

Figure 4.1 shows deviatoric stress vs. local axial strain, and pore pressure generation with local axial strain for the Leighton Buzzard Sand with de-aired water and that with different concentration sucrose solutions at 100 kPa effective consolidation pressure. Figure 4.1 presents result of six different experiments, which are the Leighton Buzzard Sand specimens with (i) de-aired water, (ii) 5g sucrose per 100g water, (iii) 25g sucrose per 100g water, (iv) 100g sucrose per 100g water, (v) 150g sucrose per 100g water, and (vi) 287g sucrose per 100g water. All the tests in this series were performed using castor sugar. The specimen with de-aired water, which was prepared by following the method 2a as explained in Chapter 3, and the specimens with the mix ratios of 5g/100g, 25g/100g, 100g/100g, and 150g/100g, which were prepared by following the method 1b, were prepared and sheared at room temperature. However, the specimen with sucrose solution at the ratio of 287g/100g was prepared at 60°C and sheared at room temperature because of the solubility of sucrose for this ratio.

As can be seen from Figure 4.1.a, all the specimens tested with sucrose solutions exhibit similar behaviour that consists of a series of successive jumps; rises in deviatoric stress to a local maximum level are followed by sharp drops. The strain levels, corresponding stress, and pore pressure data points where the fluctuations happen are tabulated in Tables 1, 2, 3, 4, and 5 in Appendix A. The fluctuations take place earlier and more frequently as the amount of sucrose increases. In addition, the bigger amplitude in deviatoric stress drops observed at larger strain levels. The Gutenberg-Richter (1956) law has been adapted to illustrate this relationship between the frequency and the deviatoric stress amplitude, which is shown in Figure 4.2. For example, the plot for the specimen with 5g sucrose in 100g water is the most similar one to the Leighton Buzzard Sand with de-aired water. In this specimen, there is only one sudden decrease followed by gradual increase in deviatoric stress. A close similarity was also observed between the pore water vs. strain plot for Leighton Buzzard Sand with de-aired water and that for Leighton Buzzard Sand with sucrose solution, which is given in Figure 4.1.b. It can also be seen that sudden change in pore pressure generation in the specimen with sucrose solution at 5g/100g takes place at approximately 2.5% strain level. From an overall view to the Figure 4.1, it can be seen that an increase in sucrose content triggers an increase in the number of fluctuations starting earlier within the same strain level. Examining all the plots in these two Figures (Figures 4.1.a, b) further, it is observed that the initial amount of sucrose may give an insight to predict the characteristics of the remaining test.

It is postulated that the mechanism observed in the specimens tested could be because of two main reasons; (i) this successive formation may be due to the tested specimens' own material properties (ii) the mechanism observed in the specimens could be because of any compliance in the apparatus. In the light of the study by Gajo (2004), it has been seen that the influence of system compliance on collapse of sand samples needs to be taken into account. Gajo revealed that the compliance of the loading apparatus can deeply affect the onset of dynamic instability. One of the reasons of the fluctuations observed could be the stiffness of the apparatus itself. However, it seems to be beyond the scope of this study to provide a detailed discussion on the apparatus compliance, since the apparatus used during the testing of specimens with sucrose solutions and with de-aired water was the same apparatus. Here in this study therefore, it will be more specifically focus on the material properties of the specimens tested.

The fluctuation mechanism observed at all the specimens tested with sucrose solutions may be attributed to the structure of the specimens. It is postulated that the mechanism observed in the specimens tested could be because of (i) higher number of contact points, (ii) a number of open fabric structure in the specimens, and (iii) stick-slip behaviour between soil particles because of the characteristics of sucrose solutions. The higher number of contact points may be due to the higher relative density values in the specimens with sucrose solutions (Table 4.1). It seems to be possible that the grains in the specimen may be held in by interlocking asperities in a denser sand packing leading to a mechanism having higher internal friction between the soil particles. Therefore, it is postulated that the movement (rotation and axial) of each particle in the specimens with sucrose solutions occurs suddenly and much later than movement in the specimens with water. In other words, as the movement of a particle in the sample with water develops gradually in time domain, the movement of the particles in a sample with sucrose solution develops dis-continuously where it may take more time to overcome the grain interlocking based on the particle roughness leading a kind of threshold energy.

In the light of the study by Luding (2003), the author believes that, during the increase in deviatoric stress, the particles in the shear band of the specimens with sucrose solutions do not move as easy as those of the specimens with water. Whereas, during the sudden decrease in the deviatoric stress, the motion of the particles is allowed more easily, because it is assumed that the applied force exceeds a critical threshold force. However, although it is seen that the Leighton Buzzard Sand with de-aired water in dense form, which was prepared and tested by following the method 2b, has also a higher density, this specimen doesn't exhibit any fluctuation in neither deviatoric stress nor pore water generation among testing of the specimen. So, it seems to be arguable to propose an approach based on density only.

One of the reasons that may lead to the fluctuations observed in the specimens with sucrose might be related to a fabric structure. Although a series of open fabric structure (i.e., honeycomb) seems to be able to give a series of fluctuations in a deviatoric stress vs. strain and corresponding pore water generation plots, it is expected that the fluctuations would be in a descending trend in deviatoric stress amplitude as well as a decrease in frequency. However, it doesn't correspond to the fluctuation characteristics observed at the end of experimental study here in this investigation. The fluctuation mechanism observed in all specimens tested with sucrose solutions may be attributed to the stick-slip behaviour happening between soil particles because of the characteristics of sucrose solutions.

4.1.1 Stick-slip mechanism

The successive formation (rises in deviatoric stress to a local maximum level and the sharp stress drop) observed in the Leighton Buzzard Sand with sucrose solutions at various concentrations might give a stick-slip behaviour nature to the fluctuations. Stick-slip is a complicated phenomenon that can be identified mainly by the stress drop amplitude (Δq) and the deformation ($\Delta\varepsilon$) (or time, Δt) between two successive stress drops. Deviatoric stress oscillations were observed for each sucrose-coarse rotund sand mix ratio, differing in the deviatoric stress amplitude and the strain intervals corresponding to each of the stress drops. The deviatoric stress fluctuations may be attributed to the stick-slip mechanism between the sand grains as they form force chains to support the applied load. Jamming occurs because the sand particles form the chains (primarily) along the compressional direction. During the sticking, the sand grains are more closely packed and exhibit a gradual increase in deviatoric stress, however; when the force chain becomes relatively unstable, some grains slide out of the column resulting in the deviatoric stress to sharply drop. The deviatoric stress subsequently builds up (self organize) again to form a new chain of columns so as to support an applied stress.

Figures 4.3 and 4.4 present two idealizations for stick-slip behaviour in a deviatoric stress vs. strain and pore water pressure vs. strain plots respectively. It can be seen from the Figures that a drop in deviatoric stress leads to a sudden increase in pore pressure values. A drop in deviatoric stress and corresponding increase in pore pressure is shown by letters ‘A’ for the first point, and ‘B’ for the second one at each side of this line. Because of the time interval selected in data recorder at the beginning of experimental work, the time difference between these two points is 10 seconds. However, as can be seen from the Figure 4.5, while the data recording interval is constant, a few strain values observed later are somehow smaller than the strain values observed earlier. In Figure 4.5.b, four different fluctuations up until the 0.15% strain level were focused upon, which are numbered from 1 to 4. Each region was constrained by two solid lines as A and B, time recordings were also presented at each line. It is believed that the reason of this may be movement of the sand particles from inside towards the specimen circumference. Figure 4.6 presents the relation between change in displacement and time. It can be clearly seen from the Figure 4.6 that the LVDTs move in opposite directions, although it is a very minor amount. However, this mechanism does not seem to show a good harmony to make an overall judgement on the difference in direction and amount of the movements.

Stick slip behaviour in granular materials has been investigated by other researchers in different disciplines, such as; Thompson and Grest, 1991, Feder and Feder, 1991, Demirel and Granick, 1996, Miller, O’Hern and Behringer, 1996, Nasuno et al. 1997, Albert et al. 2000, Cain et al. 2001, Gourdon and Israelachvili, 2003. These investigations however, were limited to spherical particles. An increased understanding of this phenomenon could provide insight into many different systems and disciplines, including tribology (friction), material science (fracture, failure), geophysical processes (seismic motion, earthquakes), and the flow of granular materials (avalanches) as well as man made phenomena such as traffic flow.

Materials in granular form are composed of many solid particles that interact through contact forces. The reason of this jamming noted in the specimens tested with sucrose solutions might lie in the fact that the forces may not propagate uniformly through the specimen but are localized along the force chains, and the jammed state is dependent on the properties of the network of these force chains. A similar behaviour was also noted by Alshibli and Roussel (2006). Cates et al. (1998) showed that the strain resulting in a simple granular pile from grain weight combines with the randomness in their packing to limit the motion of individual grains, which leads to a jammed state. An applied external stress leads to an internal structure resisting the stress; therefore, a jammed state is dependent on the direction and the magnitude of the stress. Based on observations of force chains on photoelastic disks it was hypothesized that particles formed stress chains preferentially in the direction of the slope. Vanel et al. (1999) and Al Hattamleh et al. (2005) described that these force chains form arches that shield the centre from some of the weight, thereby forming the jamming.

From the experimental investigation, comparing the test results obtained using Leighton Buzzard Sand with de-aired water and the Leighton Buzzard Sand with sucrose solutions (i.e., castor sugar) at various concentrations; it is thought that the difference between these experimental results may be attributed to the force chain mechanism in the specimens tested. It is believed that the potential energy of the particles in sucrose solutions indicates a relatively higher inhomogeneity of the forces and stresses. From one particle in a chain to the next one out of the chain in a specimen with sucrose solution, the stress during the shearing process can change significantly. A deviatoric stress increase in a specimen tested with sucrose solution causes a stress distribution among the matrix that develops relatively unstable comparing to the other specimens tested with water where the stress distribution occurs gradually. During the loading, stress or force chains distribution in the specimens with sucrose solutions are formed slower than those in the specimens with water, and destroyed suddenly. This takes place mainly in the direction against the applied load. Jensen et al. (1999) modeled soil by discrete element model and showed that in two dimensions, each particle brings three degrees of freedom to the model. In that study, each interparticle contact was modeled with a normal-direction spring and dashpot, and a spring-dashpot-slider assembly in the tangential direction. From the investigation by Jensen et al. (1999), it seems to be possible that the stress chain in Leighton Buzzard Sand with sucrose may be stronger than that in the Leighton Buzzard Sand with de-aired water in all the aspects in two dimensions.

4.1.2 Small strain stiffness

The influence of sucrose solutions at different concentrations on the small strain stiffness of Leighton Buzzard Sand was also investigated by considering secant Young's modulus. The small strain Young's modulus of the specimens was measured using linear variable differential transformers (LVDTs). Degradation of the undrained secant Young's modulus of the specimens tested was measured at effective consolidation pressures of 100 kPa (Figure 4.7.a). In order to have an understanding if the Young's modulus of the specimens was also a function of sucrose solutions, the results were compared at the range of 0.001% to 1% strain levels.

The secant Young's modulus of all the specimens increased with adding sucrose, with an exception of the sucrose solution prepared at a concentration of 150g sugar per 100 g water, which is seen just below the Leighton Buzzard Sand specimen with de-aired water. As can be seen from the Figure 4.7.a, the stiffness values for all the specimens tested using various pore fluids are fairly close to each other at a strain level after the 0.01% strain level. However, the secant Young's modulus result of the specimens with sucrose solutions differed from the small strain stiffness result obtained from the specimen with de-aired water at larger strains, in particular; after the 0.01% strain level, characteristics of the plots for the specimens with sucrose solutions considerably change. It is observed that the use of sucrose solution at various concentrations led to a sawtooth like behaviour at larger strains. This was not surprising as similar fluctuations were present in the deviatoric stress vs. strain and the pore water pressure vs. strain plots for the same specimens, which were given in Figure 4.1.

A comparison of the stress space results for the Leighton Buzzard Sand with sucrose solutions at various concentrations and the Leighton Buzzard Sand with de-aired water under an effective consolidation pressure of 100 kPa is also given in Figure 4.7.b. The Figure presents only three of the results to show clearly the difference between them, which are the Leighton Buzzard Sand with (i) 287g sucrose per 100g water, (ii) 100g sucrose per 100g water, and (iii) de-aired water. As can be seen from the Figure 4.7.b, the test result on the Leighton Buzzard Sand with 100g sucrose per 100g water may be described as a transition between the test results on the Leighton Buzzard Sand with de-aired water and that with the 287g sucrose per 100g water. The data on the secant Young's modulus as well as the data on the deviatoric stress and pore pressure already gives an insight to such a gradual change in the behaviour of the specimens tested.

4.2 Observed effects of silicon oil as pore fluid on the behaviour of Leighton Buzzard Sand

From the pore pressure generation among the test results given in the section 4.1, the pore fluid viscosity seems to be significant. Here in this section, it was aimed to test whether the observed behaviour is a result of differences in viscosity only. To test this hypothesis, Leighton Buzzard Sand was tested using silicon oil as a pore fluid, which has a similar viscosity range to the sucrose solutions employed in the previous section, but different chemical composition. Analysis, interpretation and discussion of the results will be now followed to understand the change in the behaviour of Leighton Buzzard Sand by the silicon oil.

A triaxial compression test was conducted on the Leighton Buzzard Sand with Dow Corning Silicon oil at room temperature. The tests were performed at effective consolidation pressures of 100 kPa. Although, the Leighton Buzzard Sand with oil and that with de-aired water were prepared by following similar methods, described as methods 3 and 2a respectively in Chapter 3, the obtained bulk densities, dry densities and relative densities for the Leighton Buzzard Sand were different from each other. Bulk, dry, and relative densities for the Leighton Buzzard Sand are; 1.91g/cm^3, 1.56g/cm^3, and 34.5% for the specimen with de-aired water; 1.94g/cm^3, 1.59g/cm^3, and 50.8% for the specimen with silicon oil.

Figure 4.8 makes a comparison between the stress-strain and pore pressure-strain results of the Leighton Buzzard Sand with de-aired water, and the Leighton Buzzard Sand with silicon oil. Similar to the specimens tested with sucrose solutions, the Leighton Buzzard Sand with oil also gives a series of fluctuations that could be interpreted due to the stick-slip behaviour among the particles. The deviatoric stress values, where the fluctuations take place, change between approximately 90 kPa and 60 kPa. Table 6 in Appendix shows all the drop and building up points (stress, strain, and pore pressure) recorded during the testing of the specimen with oil.

Comparing the small strain secant Young's modulus behaviour of the Leighton Buzzard Sand with silicon oil and that with de-aired water at 100 kPa effective consolidation pressure (Figure 4.9.a), it is noted that somewhat higher secant Young's modulus values were obtained for the Leighton Buzzard Sand prepared with Dow Corning silicon oil up until approximately 0.06% strain level, where the first sudden drop took place. After this strain level, the Leighton Buzzard Sand with de-aired water exhibited higher values of secant Young's modulus. In Figure 4.9.b, a significant difference can also be seen between the stress path plots obtained from the two results. While the Leighton Buzzard Sand with silicon oil gives a structure permitted area, the specimen tested by using de-aired water gives a curve.

4.3 Effects of temperature on the behaviour of Leighton Buzzard Sand

An experimental programme was also followed to establish whether changes in temperature influence the behaviour of Leighton Buzzard Sand with sucrose solution. To investigate the effects of temperature for the purpose of crystal growing at different temperatures, a program of triaxial compression testing was conducted on Leighton Buzzard Sand with sucrose solution at elevated temperature and room temperature separately. The tests were performed at an effective consolidation pressure of 100 kPa. Materials used in the tests were Leighton Buzzard Sand with 207g sucrose (i.e., icing sugar, castor sugar) per 100g water. Each material was tested at both 60°C and at room temperature (approximately 23°C).

4.3.1 Sucrose solutions using castor sugar

207g sucrose (i.e., castor sugar) was weighed and mixed with Leighton Buzzard Sand. The mixture was then submerged in de-aired water and stirred with a spoon to obtain a uniform paste, which was explained in details as method 4 in chapter 3. The observed behaviour of the Leighton Buzzard Sands with sucrose solutions having the same amount of castor sugar but at different temperatures were compared. Considering the deviatoric stress vs. strain plot in Figure 4.10.a, it can be seen that the deviatoric stress values in Leighton Buzzard Sand with sucrose solution tested at 60°C are higher than the Leighton Buzzard Sand with sucrose solution at room temperature. The temperature difference produced an approximately 50kPa increase in peak deviatoric stress. The deviatoric stress values of the Leighton Buzzard Sand with sucrose solution tested at 60 °C is also higher than the Leighton Buzzard Sand tested with de-aired water up to 0.63% strain level. However, after this point Leighton Buzzard Sand with de-aired water gives the highest stress values among the three specimens analysed in this investigation. Following the above, it can be concluded that based on the tests where the same materials and apparatus were used, a change in temperature seems to increase the deviatoric stress. Further examining the Figure 4.10.a, it can be seen that the first fluctuation in the Leighton Buzzard Sand with sucrose solution tested at room temperature took place at 0.021% strain level, which is much earlier than the first fluctuation in the Leighton Buzzard Sand with sucrose solution tested at 60°C, which occurs at 0.23% strain level. It could be described that the specimen tested at 60°C gives an insight to a ductile behaviour as the specimen tested at room temperature behaves relatively brittle.

Figure 4.10.b presents the pore water pressure generation corresponding to the tests described in Figure 4.10.a. The test on Leighton Buzzard Sand with de-aired water exhibits a higher pore water generation up to the 0.45% strain level. Although a clear dilation can be observed in the specimen tested using Leighton Buzzard Sand with de-aired water, the tests on the Leighton Buzzard with sucrose solutions at both

temperatures do not show dilation within the measured strain range. However, it may not mean that there is no dilation in a local scale between every sudden drop followed by gradual increase in the plots.

Considering the undrained secant Young's modulus of the Leighton Buzzard Sand tested with de-aired water at room temperature, the Leighton Buzzard Sand tested with sucrose solution at room temperature, and the Leighton Buzzard Sand tested with sucrose solution at 60°C, it can be seen from Figure 4.11.a that the undrained secant Young's modulus is relatively higher for the specimen with sucrose solution tested at 60°C while the remaining specimens relatively low. The difference in behaviour is confirmed by the relatively high deviatoric stress values. On the other hand, the values of the undrained secant Young's modulus between 0.001% and 0.01% are in approximately same range as those for the Leighton Buzzard Sand with de-aired water and those with the sucrose solution tested at room temperature. In spite of the similarity in stiffness at smaller strain levels, as it was expected from the stress-strain plots of the specimens, Leighton Buzzard Sand with sucrose solution exhibits fluctuations which first start at the 0.022% strain level with a considerable change happening suddenly. This sharp decrease in the secant Young's modulus starts to be built up again from the 0.048% strain level. This mechanism continues up until the end of testing. It can be seen that the fluctuations in the specimen prepared by using Leighton Buzzard Sand with sucrose solution and tested at 60°C take place much later than that in Leighton Buzzard Sand with sucrose solution tested at room temperature.

In addition, Figure 4.11.b presents the stress path results for all three specimens. The most striking point in these plots is that the Leighton Buzzard Sand with sucrose solutions exhibit similar behaviour, which could be described as a structure permitted space.

4.3.2 Sucrose solutions using icing sugar

In order to draw a comparison between the influences of temperature and the behaviour of a geomaterial, similar experimental investigation was performed on the Leighton Buzzard Sand with icing sugar at two different temperatures, which were room temperature and 60°C. For this investigation, two types of specimen were prepared, the first one was prepared by following the method described as 4d and tested at room temperature, the second one was prepared by following the method described as 1a and sheared at 60°C. The amount of sucrose (i.e., icing sugar) was same in both experiments, which was 207g per 100g water. The results are also compared with the Leighton Buzzard Sand with de-aired water tested at room temperature.

Figure 4.12.a shows the deviatoric stress vs. strain results for three different tests under 100 kPa effective stress. The temperature difference lead to a sawtooth behaviour beginning from the 0.091% strain level to the end of testing on the Leighton Buzzard Sand with sucrose solution tested at 60°C, whilst Leighton Buzzard Sand with sucrose solution tested at room temperature shows contractive behaviour instead a sawtooth behaviour. Table 7 in Appendix presents the stress and strain values where the fluctuations happen in the specimen tested at 60°C. The Leighton Buzzard Sand with de-aired water at room temperature gives a higher stress values at almost all the strain levels. Plot for the Leighton Buzzard Sand tested at 60°C with sucrose has a place higher than the Leighton Buzzard Sand with sucrose tested at room temperature, but lower than the Leighton Buzzard Sand with de-aired water tested at room temperature. It is interesting to note that the plot for the Leighton Buzzard Sand with sucrose solution tested at room temperature may be seen as a kind of lower boundary for the specimen tested at 60°C.

Figure 4.12.b shows a plot presenting pore water generation corresponding to the deviatoric stress vs. strain plots for three different tests described above. The Leighton Buzzard Sand with de-aired water exhibits a clear dilation process; however, the Leighton Buzzard Sand tested at room temperature with icing sugar solution shows a

continuous increase from beginning of the test to the end of the test, where the pore pressure generation goes up until above the 350 kPa. As can be seen from the Figure, Leighton Buzzard Sand tested at 60°C with sucrose solution shows a significantly different behaviour from the other two plots. The specimen tested at 60°C shows a mechanism representing a sawtooth mechanism. The pore pressure and the strain levels where the sudden pore pressure drops and building up recorded are also shown in Table 7 in Appendix.

It was noted (Figure 4.13.a) that the secant Young's modulus increased with the adding of icing sugar up until around 0.005% strain level, regardless of temperature. Referring to the Figure 4.13.a, it can be seen that the addition of sucrose to the pore fluid resulted in considerably reduced specimen stiffness at room temperature after 0.005% strain level. However, the stiffness of the specimen tested at 60°C with the icing sugar solution has higher values up until to approximately 0.044% strain level. The specimen tested at 60°C with the sucrose solution also exhibits sharp drops followed by gradual increase in stiffness.

The stress paths of the specimens tested in this series are shown in Figure 4.13.b. Comparing all three plots, it can be seen that the shape of the curves are significantly different from each other. The causes of these differences are sucrose constituents as well as the temperature. Similar to the preceding tests, the Leighton Buzzard Sand tested at 60°C with icing sugar solution gives an area. On the other hand, the result for the Leighton Buzzard Sand with icing sugar solution tested at room temperatures could be attributed to the particle effects of the sucrose solids that were not dissolved.

4.4 Comparison of the results from the different testing methods

This section makes a comparison between the results of the various testing methods used during the investigation on the mechanical properties of Leighton Buzzard Sand with sucrose (i.e., castor sugar) solutions. During testing of the specimens, five different methods (1a, 1c, 1d, 2a and 4d) were employed for similar type of mixtures using different preparation techniques. All specimens were tested in compression and under undrained conditions at 100 kPa effective consolidation pressure.

Figures 4.14 and 4.15 show a typical example for the specimens prepared by following the method described as 1c in Chapter 3. Specimens in this method were prepared at 60°C using a heater mat, and then sheared at room temperature by following a gradual decrease in temperature in three days. For the test results shown in Figures 4.14 and 4.15, 287g sucrose (i.e., castor sugar) was used per 100g water to use as a pore fluid. The reason of following this method was an intention to show a possible influence of crystal growing between the soil grains. As can be seen from the Figure 4.14.a, the Leighton Buzzard Sand tested exhibits a sawtooth behaviour changing between the 180kP and 50 kPa in deviatoric stress. All the points where the fluctuations happen are presented in Table 5 in Appendix. It is observed that, at almost every fluctuation, the amount of drop in deviatoric stress is approximately half of the maximum value. The corresponding pore water generation plot also shows a kind of sawtooth behaviour (Figure 4.14.b). The maximum pore pressure value reaches 350 kPa. It can also be seen that there is no significant dilation within the strain level measured.

In the specimen prepared following method 1c, secant Young's modulus (Figure 4.15.a) also shows a sawtooth similar behaviour. Comparing this to the Leighton Buzzard Sand tested at room temperature with de-aired water (Figures 4.16, 4.17), it can be seen that the specimen prepared by method 1c gives a higher stiffness, which is around 400 MPa at 0.001% strain level. From the Figure 4.15.a, it can be seen that the biggest drop in the stiffness takes place at a strain level between 0.0147 to 0.0182%, where the stiffness values are 270MPa and 105MPa respectively. The stress path of the specimen tested is shown in Figure 4.15.b, which gives a type of structure permitted space.

Figures 4.16 and 4.17 show data from five different tests for the specimens obtained using various methods. In Figure 4.16.a, it can be seen that all the methods, except 2a followed for the Leighton Buzzard Sand with de-aired water, exhibit a sawtooth behaviour up until the end of the test. The specimens with sucrose solutions tested by following the methods 1a and 1d have higher deviatoric stress values within the measured strain levels. Tables 4.5, 4.7, 4.8 and 4.9 in Appendix show all the points where the fluctuations take place in the test results from the methods 1c, 4d, 1d and 1a. As can be seen from the tables, the values showing the point just before the first drop in deviatoric stress are approximately twice of the amount of decrease in deviatoric stress. The values observed just before and after the first drops in deviatoric stress for all the specimens mentioned above are observed as; 42 kPa-19 kPa for the method 1c, 44 kPa-35 kPa for the method 4d, 63 kPa- 33 kPa for the 1d, and 142 kPa- 109 kPa for the method 1a where the testing temperature was around 60°C. Figure 4.16.b presents the pore pressure generation during all five tests. It can be seen that there is a clear dilation process during the specimen tested by following the method 2a, where the Leighton Buzzard Sand was tested with de-aired water as a pore fluid. On the other hand, all the other specimens tested give a sawtooth behaviour in similar characteristics to each other. The specimen tested by following the method 1c generates the highest pore pressure at almost every strain levels among the tested four specimens, which is around 350 kPa. Pore pressure generations in all the other three specimens tested are very close to each other in particular from beginning to the 1% strain level.

Figure 4.17.a presents a comparison of secant Young's modulus results for Leighton Buzzard Sand specimens prepared using different methods at an effective consolidation pressure of 100 kPa. As can be seen from the plots, the stiffness values for the Leighton Buzzard Sand prepared by following method 4d are closer to the values obtained from method 2a from the start of shearing up until a strain level of 0.0066%. Whereas, the values obtained from the other methods are higher than the values obtained by the method 2a. Method 2a can be used as a reference for all the tests obtained using the same apparatus, because it used clean Leighton Buzzard Sand with de-aired water at room temperature. Method 4d mixed the required amount of sucrose particles and Leighton Buzzard Sand grains on a bench, which seems not to make any contribution to the secant Young's modulus. However, the other methods (1a, 1c, and 1d) give an increase in secant Young's modulus to the samples up until to the around 0.1% strain level. In order to draw a comparison that can be seen clearly, Figure 4.17.b presents only

two of the tests results given in this series of experiments, which were obtained from the methods 1c and 2a. As expected from the plots already shown, the Leighton Buzzard Sand tested following the method 1c exhibits a type of structure permitted space, while the Leighton Buzzard Sand with de-aired water tested following the method 2a gives a stress path that was an expected plot from the tests in the literature (i.e., Theron, 2004).

4.5 Influence of sucrose crystal growth between the soil particles

From the pore pressure generation among the soil samples tested with sucrose solutions, the significance of the viscosity tube measurements for the sucrose solutions seem to be undisputable (see the Table 4.1, and Figure 4.1). It is observed that the higher measurement in viscosity tube leads to a pore pressure generation developing later, because a fluid having higher viscosity moves slower than a fluid having lower viscosity. It is known that the use of a pore fluid with higher viscosity will significantly decrease the permeability of a soil; in addition, the use of viscous fluids may also affect the mechanical properties of soils such as strength and stress-strain behaviour (Zeng et. al., 1998).

However, considering the value of the viscosity tube measurements of the silicon oil that was used as a pore fluid in one of the samples tested during the experimental study, it is realized that although there is a relationship between the viscosity tube measurements of sucrose solutions and pore fluid generation, this does not seem to follow for the oil-sand specimen. Therefore, viscosity tube measurements do not provide a complete understanding of the behaviour of the specimens tested in terms of the pore fluid generation among these samples. It is thus implied that viscosity tube measurements analysis results should not be applied blindly for the experimental series conducted in such researches.

In this context, microscope photographs taken of thin sections were employed to investigate the fabric of the Leighton Buzzard Sand specimen prepared using a sucrose solution. Figure 4.18 shows polarized (a) and cross-polarized (b) pictures of the thin sections. The quartz sand particles and the sucrose crystals grown between them can be seen from both pictures (Figures 4.18.a, b). The section was taken from a specimen prepared with 207g sucrose (i.e., castor sugar) per 100g water pore fluid. However, from the solubility chart given in the Figure 3.16, it is not expected to have crystallized sucrose particles for this mix ratio. Therefore, it is interpreted that the difference in behaviour between the specimens prepared with silicon oil and sucrose may be attributed to the crystal growth between the sand particles.

4.6 Influence of platy particles on the behaviour of rotund sand

The study also attempts to explain the behaviour of rotund coarse particles including platy fines. A recent investigation into the behaviour of gold mine tailings by Vermeulen (2001) showed that the behaviour of tailings might be attributed to the particle shape. A more recent experimental study by Theron (2004) was conducted on an artificial soil that consists of coarse rotund particles containing various amounts of platy fines. The study presented here provides an additional data set to compare the mica - Leighton Buzzard Sand mixtures with sucrose (i.e., icing sugar)-Leighton Buzzard Sand mixtures in a triaxial apparatus. The intension was to draw a parallel between the behaviour of the Leighton Buzzard Sand mixtures with mica and icing sugar separately. The test results show that the characteristics of the Leighton Buzzard Sand tested may be principally ascribable to the presence of the flat grains. The writer postulates that platy particles occupy the voids between Leighton Buzzard Sand particles. Depending on the amount of platy particles present (i.e., mica, icing sugar), the Leighton Buzzard Sand particles are either in contact with each other and the behaviour of the samples tested are controlled by Leighton Buzzard Sand particles, or they are separated by platy particles.

In the light of Theron (2004), and Clayton et al. (2004), the shape of particles controlling the mechanical behaviour of the Leighton Buzzard Sand-mica mixtures seems to be dependent on the contact mechanism between the Leighton Buzzard Sand particles. When the Leighton Buzzard Sand particles are in clear contact to each other, and the mica particles only partially fill the pores, the mechanical behaviour of the mixes under these situations is governed by the Leighton Buzzard Sand. As the mica particles fill the pore spaces, the Leighton Buzzard Sand particles are held apart, and the platy mica fines start to control the behaviour of the mixes. The contacts between the Leighton Buzzard Sand particles reduce; the behaviour of the samples becomes clay like. With large volume of platy particles, the Leighton Buzzard Sand particles are suspended in a mica matrix which dominates the mechanical behaviour of the mixes. In brief, this shows the significance of the amount and position of the mica particles among the mixes, and also shows that the compressibility of Leighton Buzzard Sand increases with platy particles content.

From the definition of structure, the effect of platy particles on the mechanical behaviour of Leighton Buzzard Sand particles can be described as a fabric structure rather than a bonding structure. This was achieved by studying the change in density and void ratio with a range of platy particles content by Theron (2004). She has already investigated the influence of platy particles on the behaviour of Leighton Buzzard Sand particles by an extensive experimental program on mica-coarse rotund sand particles. A parallel can be drawn between the behaviour of mica Leighton Buzzard Sand mixtures and the Leighton Buzzard Sand icing sugar mixtures in this study. The addition of 5% mica to the Leighton Buzzard Sand particles increases the bulk density of the sand. Because the platy mica particles partially fill the voids between the rotund particles without changing the volume of the sample prepared to be tested. The addition of platy particles in more amount decreases the bulk density until it falls below that of the loose coarse rotund sand sample.

The influence of particle shape on the compressibility of Leighton Buzzard Sand was assessed by one-dimensional consolidating samples containing various percentages of platy fines (i.e., mica) in oedometer. Effective consolidation pressures changing between 5 kPa and 1800 kPa were employed. The compressibility behaviour of the (i) mica-Leighton Buzzard Sand mixes at various ratios and (ii) clean Leighton Buzzard Sand at various densities are shown in Figure 4.19. It was found that the presence of platy particles in the specimens tested had a marked effect on the compressibility of the material under load, on its expansion after release of pressure and other clay-like behaviour. The loose, dense, and medium dense Leighton Buzzard Sand specimens appear to be relatively incompressible. This is noticeable from the difference in the slope of the consolidation line. The compressibility of the Leighton Buzzard Sand specimens increased with increasing platy fines mica content. A similar observation was made by Gilboy (1928). The increase in compressibility by an addition of 5% mica by weight is relatively low. As already mentioned, mica particles in such a low quantity may partially fill the pore spaces between the coarse rotund sand particles. The increase in compressibility seems to be the result of some of the stress being carried by the mica particles. Mica particles are more compressible than the Leighton Buzzard Sand particles because of their particle shape and size. When the mica content increases, the Leighton Buzzard Sand particles are separated from each other and the mixture becomes gradually more compressible. Ultimately, the platy mica particles become dominant in the overall behaviour of the soil matrix.

An experimental programme in triaxial apparatus was conducted to establish the influence of platy fines on the behaviour Leighton Buzzard Sand (Figure 4.20). The influence of mica and icing sugar particles on the small strain stiffness was separately investigated by considering secant Young's modulus (Figure 4.21.a). Degradation of the undrained secant Young's modulus of the mica- Leighton Buzzard Sand, and Leighton Buzzard Sand- icing sugar mixtures was measured at an effective consolidation pressure of 100 kPa. The addition of mica to the Leighton Buzzard Sand particles resulted in reduced specimen stiffness. However, the Young's moduli of the loose Leighton Buzzard Sand and the 5% mica- Leighton Buzzard Sand mixture were almost similar. The addition of 10% mica had more effective on the stiffness results of the mica-Leighton Buzzard Sand mixture.

Comparing the small strain stiffness behaviour of the mica-Leighton Buzzard Sand mixtures and icing sugar-Leighton Buzzard Sand mixtures at 100 kPa effective consolidation pressure (Figure 4.20), it is noted that; (i) a maximum Young's modulus value was obtained for the dense Leighton Buzzard Sand, (ii) the Young's modulus of the 10% and 15% platy fines content specimens are similar to each other, (iii) the Young's modulus of the 5% mica fines is very close to the loose Leighton Buzzard sand with water, (v) the Young's modulus of the Leighton Buzzard Sand with icing sugar is greater than the loose Leighton Buzzard sand with water at small strain level.

Following the above, it can be concluded that the Young's modulus of the mixtures tested decreases with increasing platy particle content. Figures 4.20 and 4.21 show the behaviour of coarse rotund sand particles with (i) 5% mica, (ii) 10% mica, (iii) 15% mica, and (iv) icing sugar which was prepared by following the method 1b, as well as clean Leighton Buzzard Sand particles at two different densities. From the figures presented, the Leighton Buzzard Sand specimen with icing sugar exhibits a markedly different behaviour. It has been observed that the contractive behaviour described in the specimen with icing sugar develops much slower, and at a bigger strain level. Thus, the author has postulated that all the sucrose particles might have not been remain as solid particle; however some of them might be dissolved in the solution. Accordingly, it may give insight that any system of analysis which neglects the presence and the effect of the pore fluid characteristics as well as that of the flat-grained constituents will be incomplete.

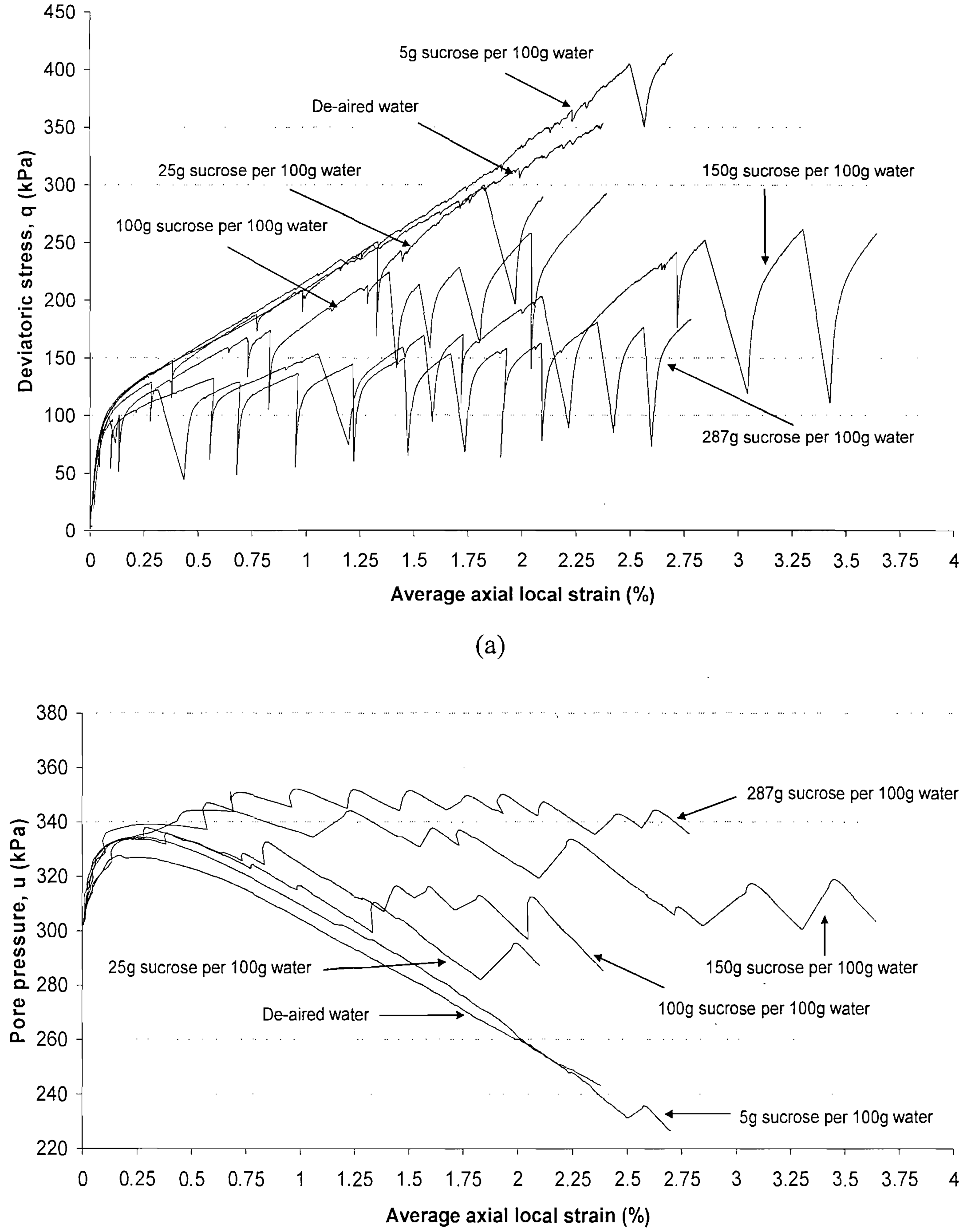
Table 4.1 Initial bulk, dry, relative densities, and void ratios of the specimens shown in Figure 4.1, and some properties of the pore fluids used in these specimens.

Specimen description	**Bulk density (Mg/m^3)**	**Dry density (Mg/m^3)**	**Relative density (%) of sand in specimen**	**Void ratio (e)**	**Viscosity tube measurements for the pore fluids (mm^2/s) at room temperature**	**Density measurements for the pore fluids (g/ml) at room temperature**
Loose Leighton Buzzard Sand with de-aired water	1.91	1.560	34.5	0.698	0.94	0.97
Loose Leighton Buzzard Sand with 5g sucrose per 100g water	1.92	1.565	36.73	0.692	1.08	0.99
Loose Leighton Buzzard Sand with 25g sucrose per 100g water	1.96	1.577	41.36	0.680	1.58	1.06
Loose Leighton Buzzard Sand with 100g sucrose per 100g water	2.03	1.581	43.04	0.675	9.35	1.19
Loose Leighton Buzzard Sand with 150g sucrose per 100g water	2.08	1.595	48.42	0.661	37.12	1.26
Loose Leighton Buzzard Sand with 287g sucrose per 100g water	2.09	1.598	49.20	0.659	-	-

Maximum and minimum dry densities and specific gravity values used in the calculations are 1.74 Mg/m^3, 1.48 Mg/m^3, and 2.65 respectively.

Table 4.2 Classification of sands based on relative density (Terzaghi and Peck, 1948).

Classification	Relative density (%)
Very loose	0-15
Loose	15-35
Medium dense	35-65
Dense	65-85
Very dense	85-100



(a)

(b)

Figure 4.1 **(a)** Stress-strain and, **(b)** pore water vs. strain curves for clean Leighton Buzzard Sand and that with various pore fluids at 100 kPa effective consolidation pressure.

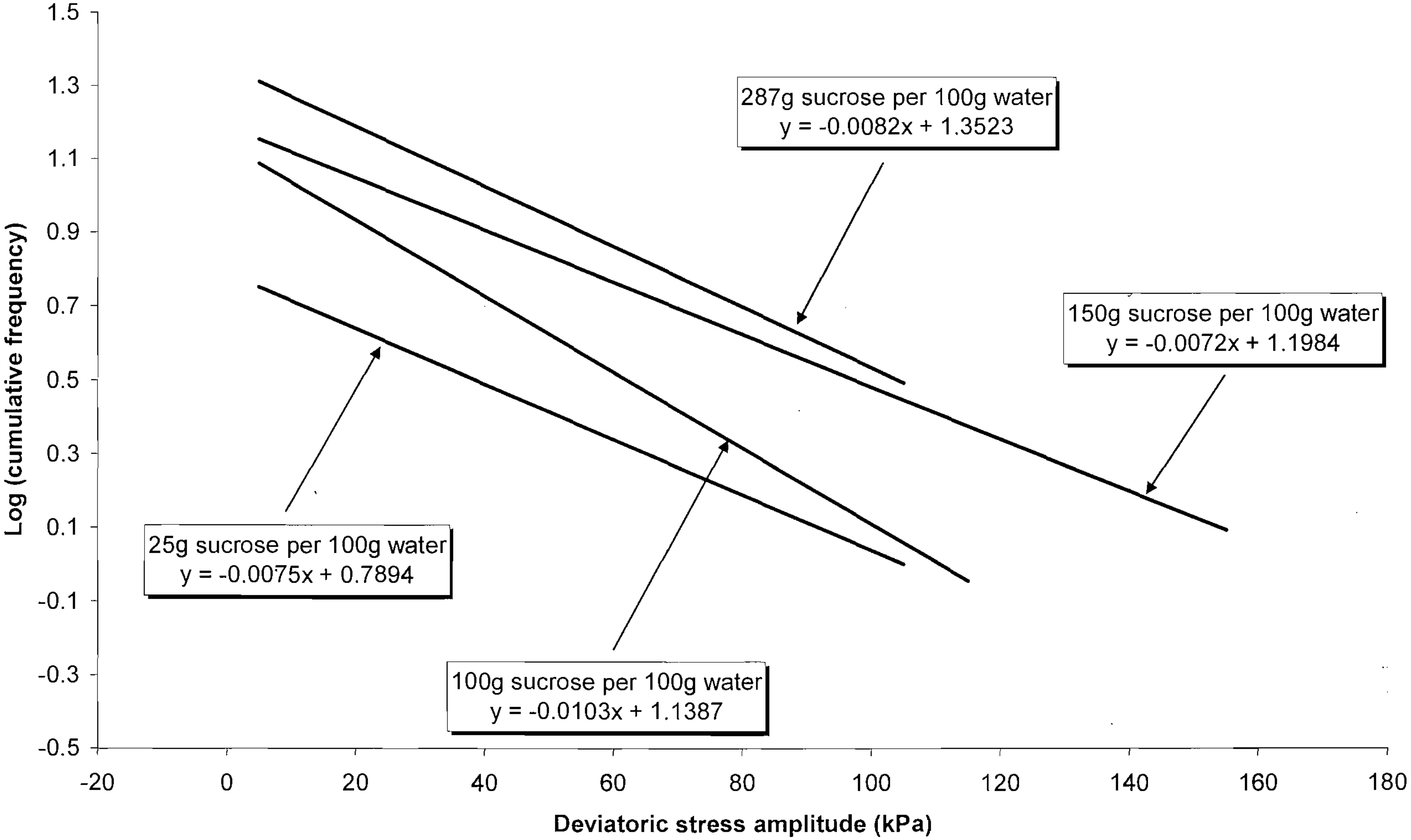


Figure 4.2 Application of Gutenberg-Richter law to the experiment results given in Figure 4.1.a

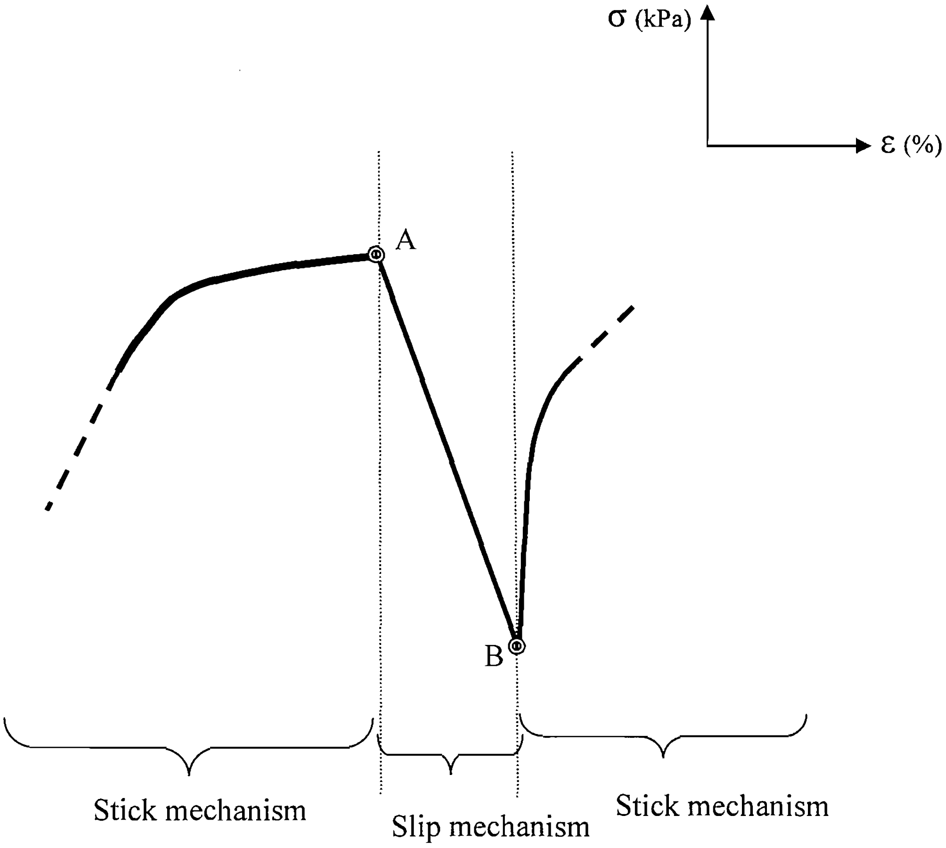


Figure 4.3 An idealization for stick-slip behaviour in a deviatoric stress-strain plot.

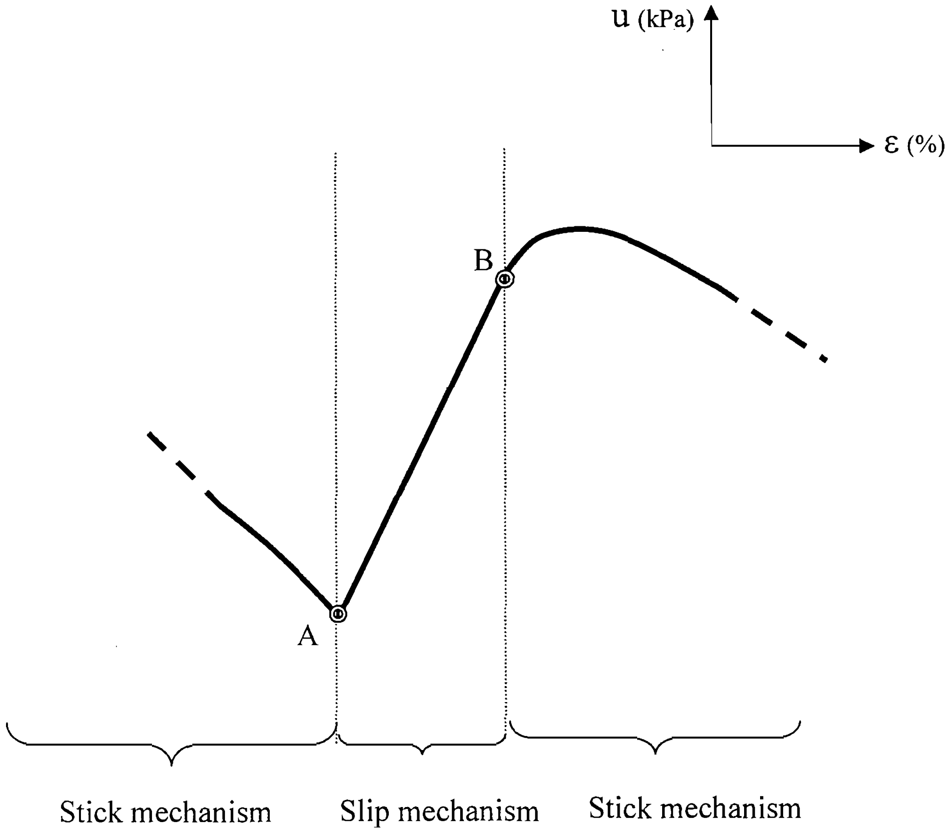
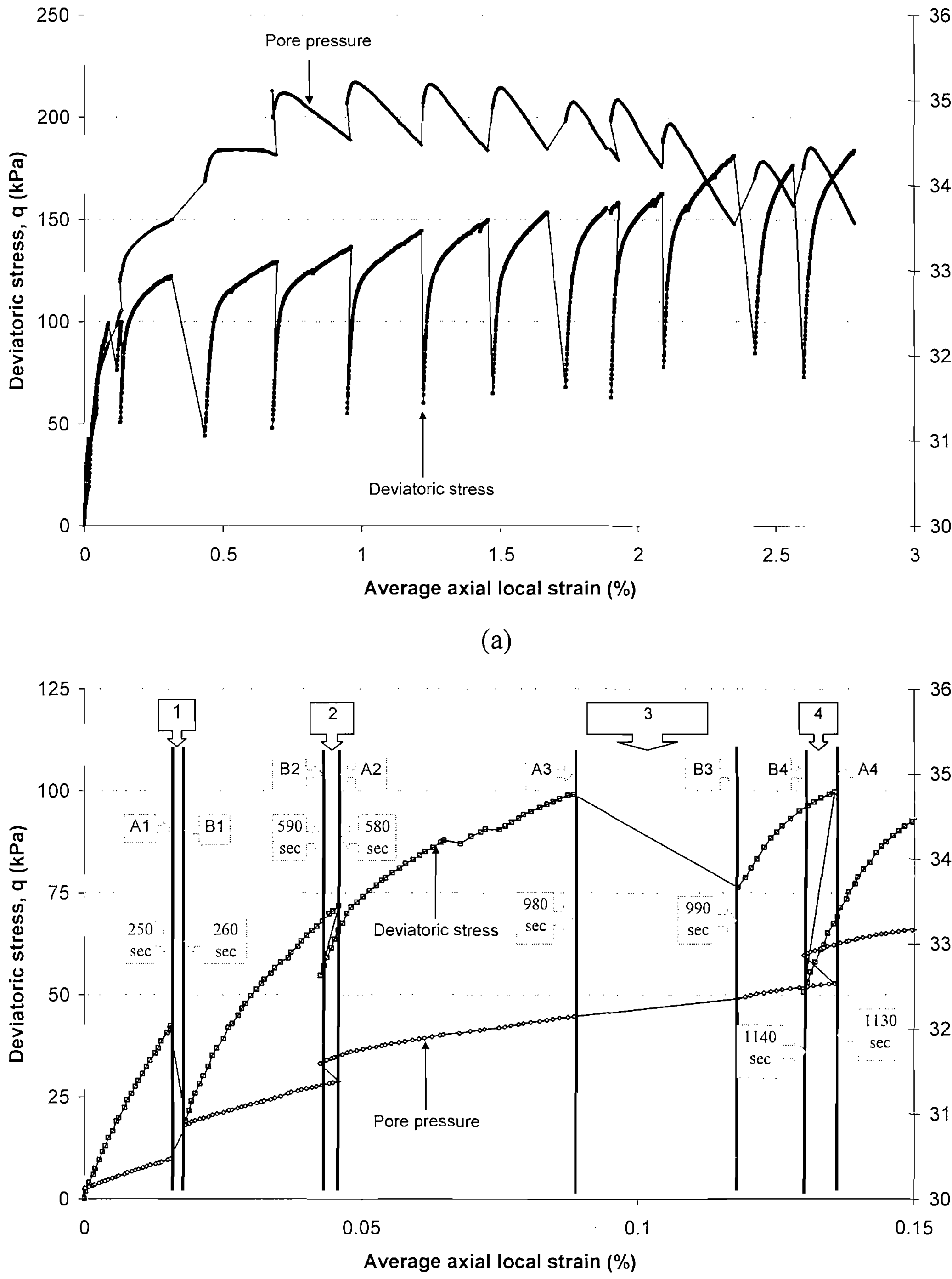


Figure 4.4 An idealization for stick-slip behaviour in a pore water pressure-strain plot.



(a)

(b)

Figure 4.5 Stress-strain and pore pressure-strain curves for the Leighton Buzzard Sand with sucrose solution (287g castor/100g water) at 100 kPa effective consolidation pressure by following method 1c **(a)** within 3% strain level, **(b)** within 0.15% strain level.

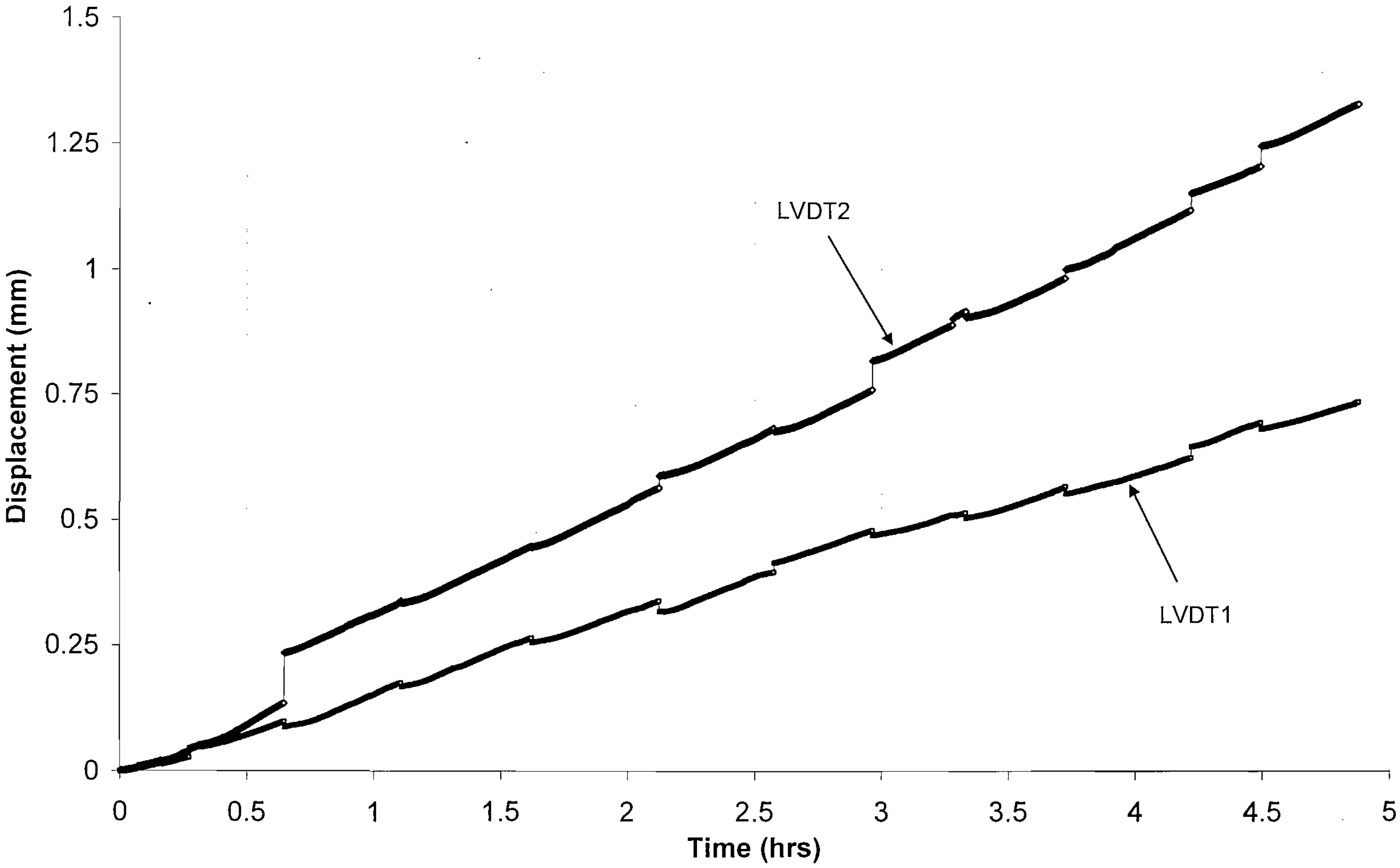


Figure 4.6 Change in local displacement with time for the Leighton Buzzard Sand with sucrose solution (287g castor/100g water) at 100 kPa effective consolidation pressure (by following method 1c).

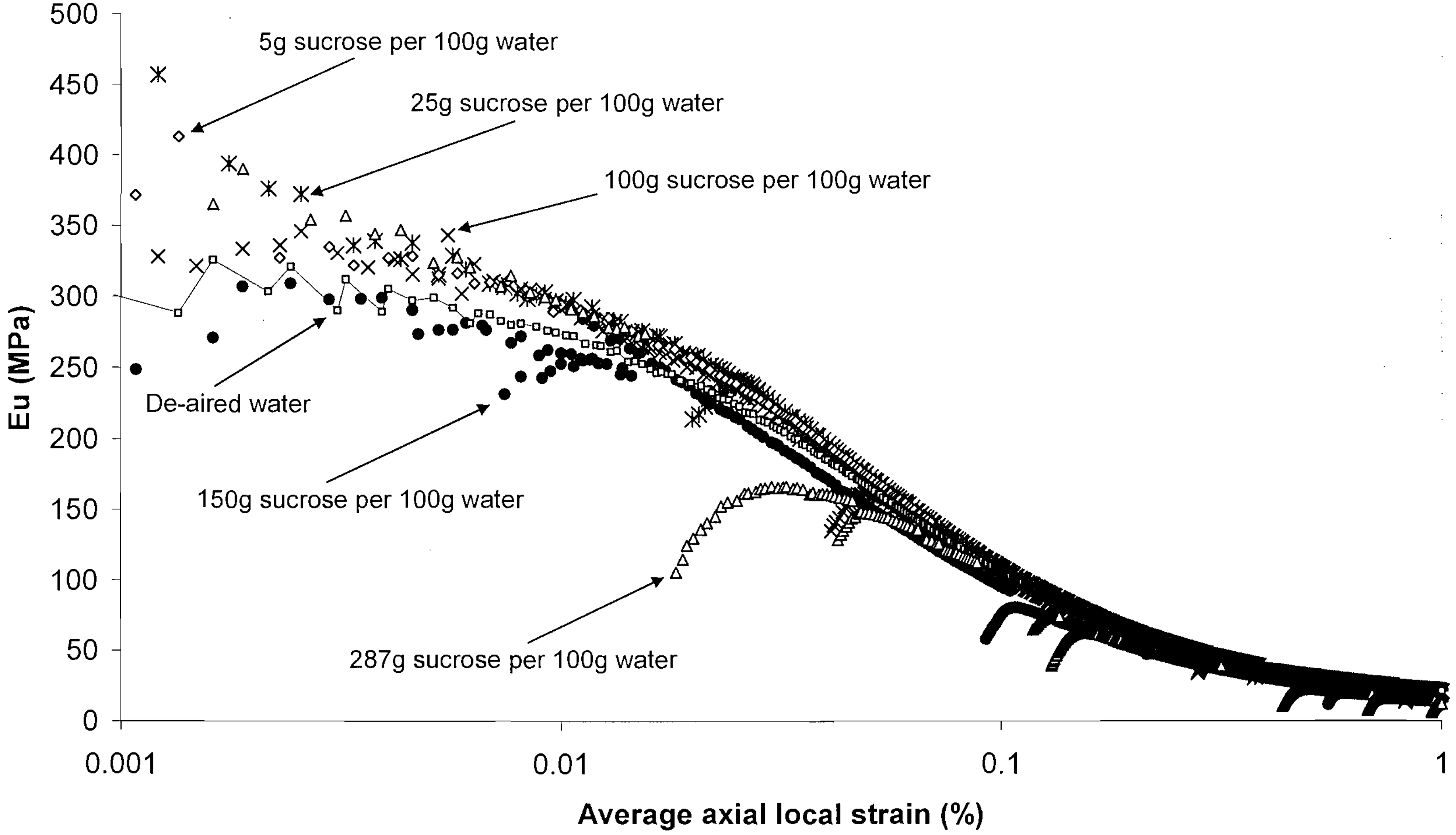


Figure 4.7.a Comparison of secant Young's modulus for Leighton Buzzard Sand with sucrose solutions at various concentrations and Leighton Buzzard Sand with de-aired water at an effective consolidation pressure of 100 kPa.

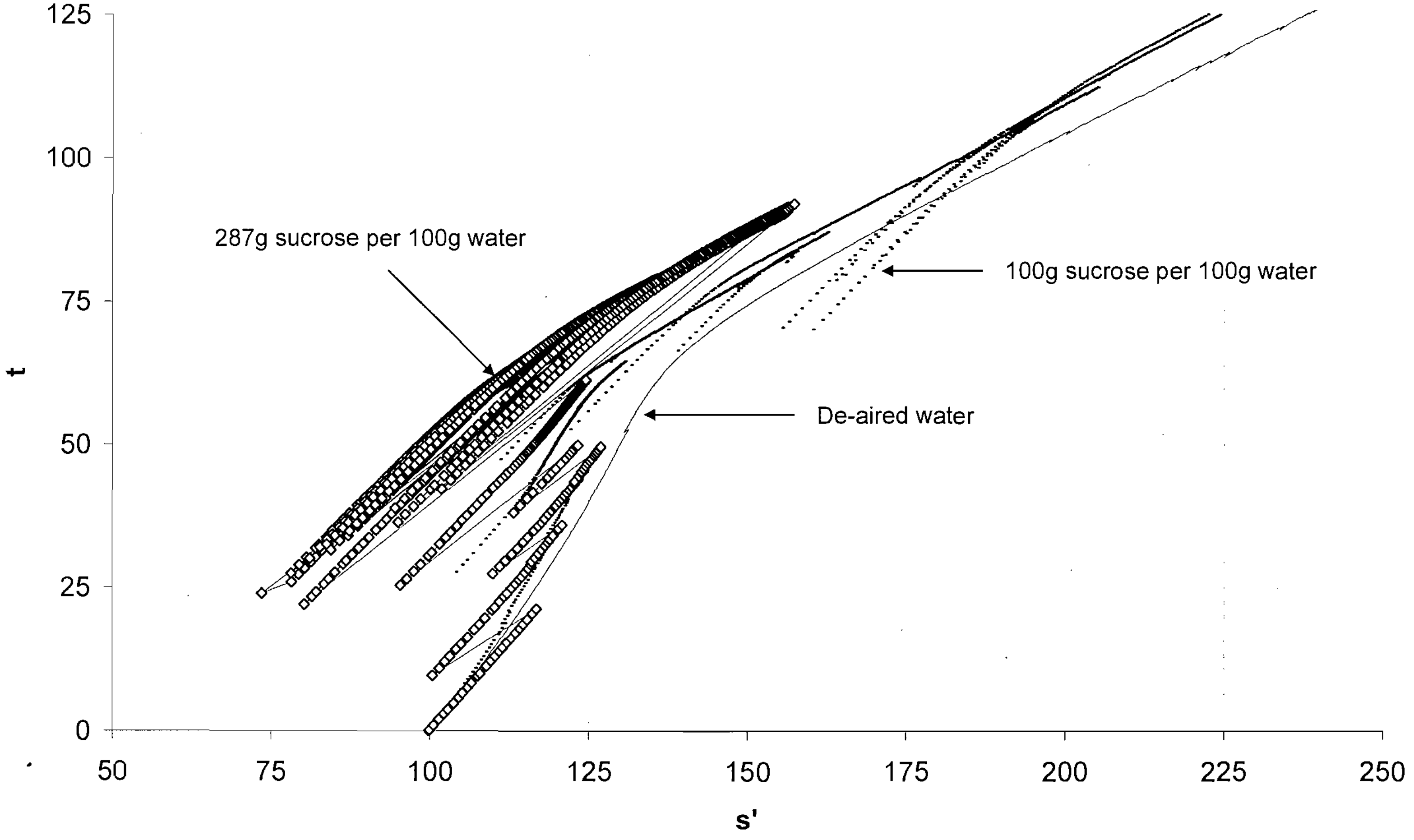
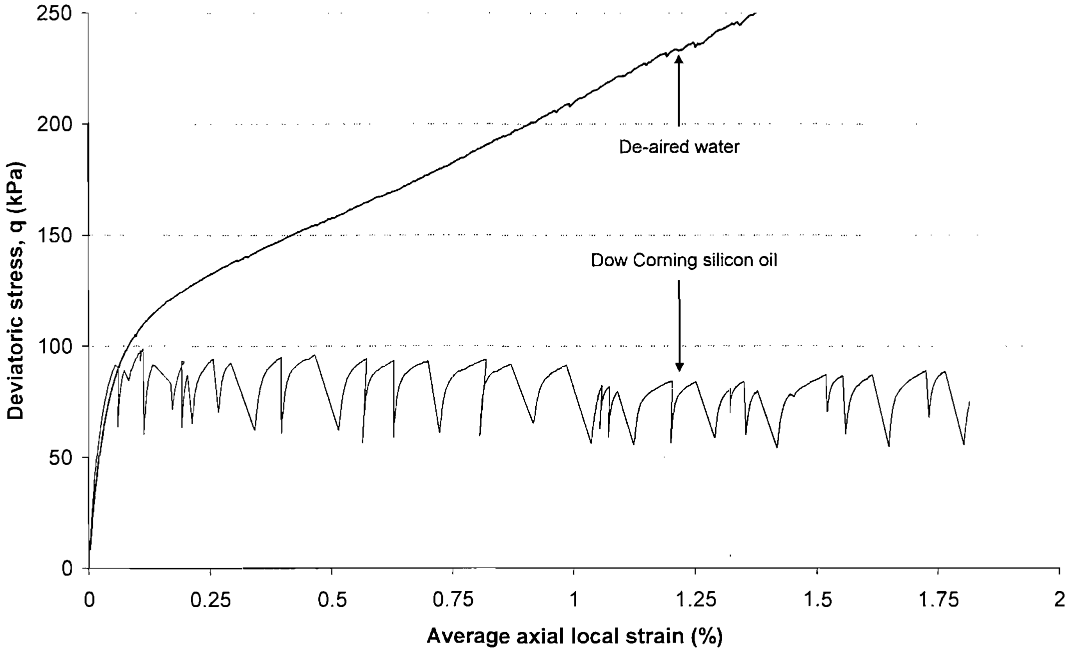
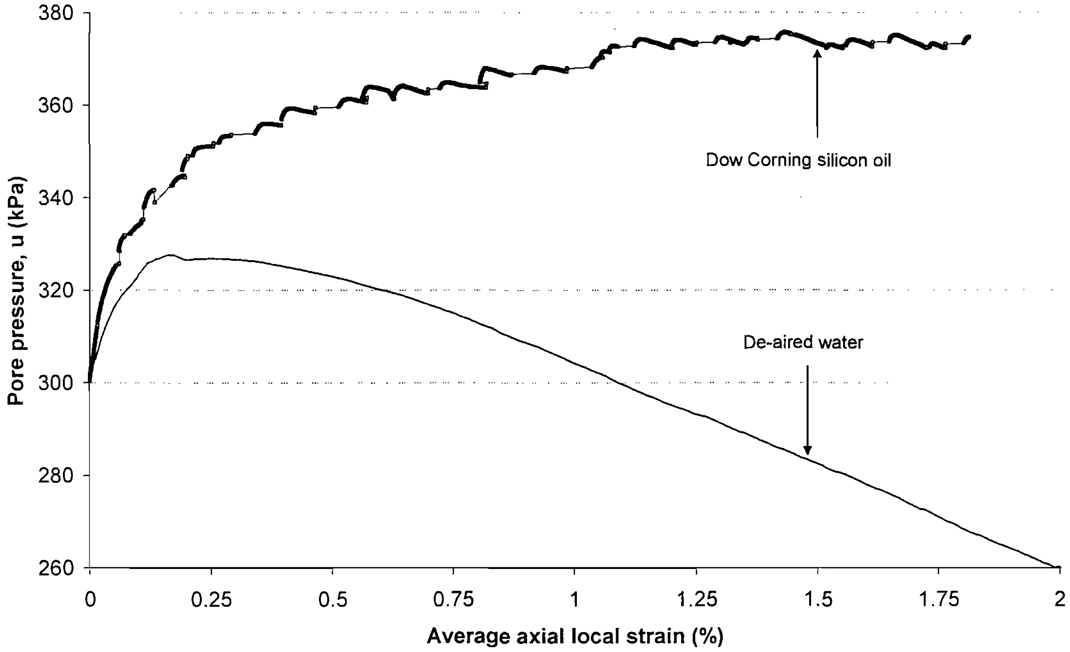


Figure 4.7.b Comparison of stress space results for Leighton Buzzard Sand with sucrose solutions at various concentrations and Leighton Buzzard Sand with de-aired water at an effective consolidation pressure of 100 kPa.

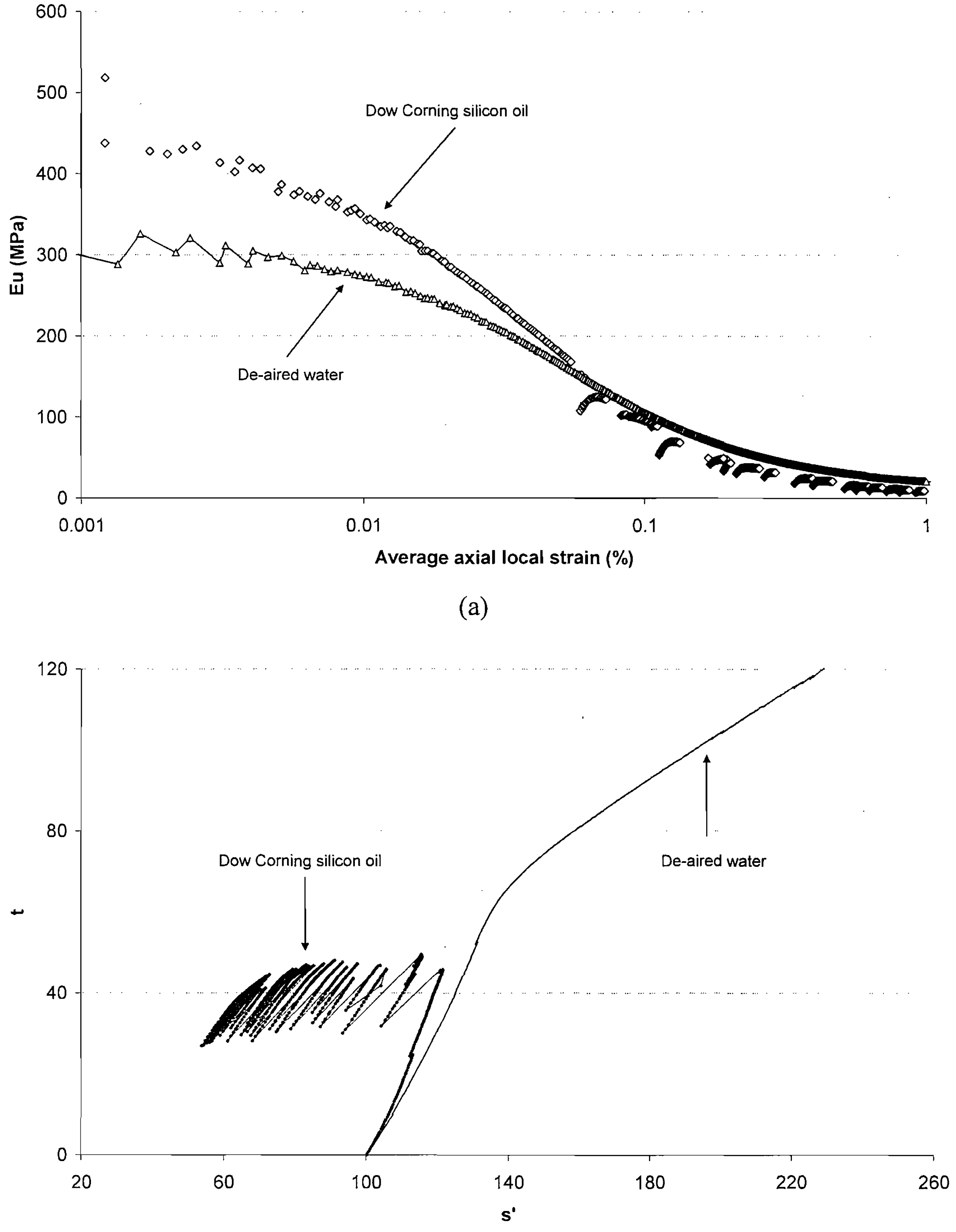


(a)



(b)

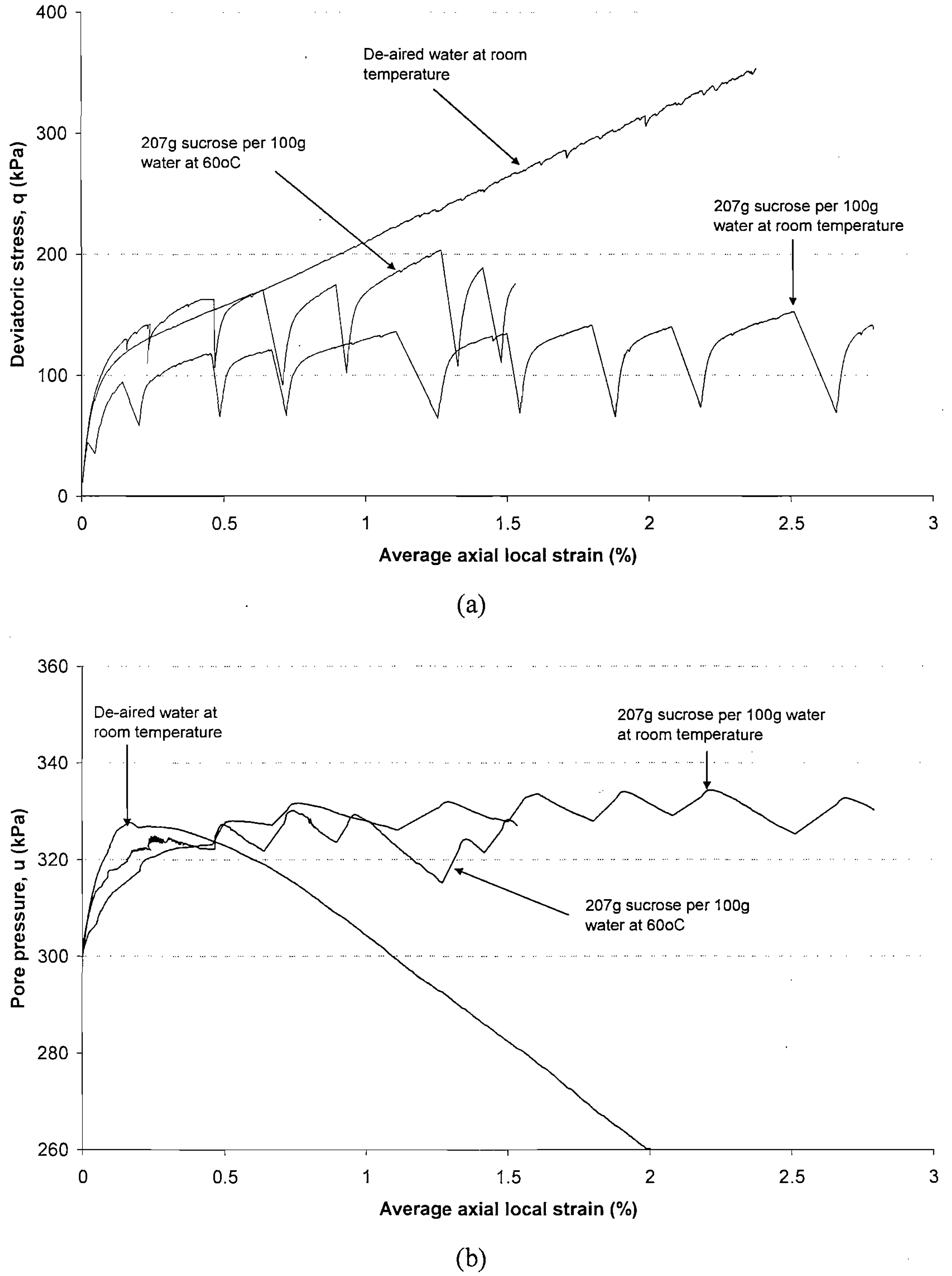
Figure 4.8 **(a)** Stress-strain and, **(b)** pore water vs. strain curves for clean Leighton Buzzard Sand and that with Dow Corning Silicon oil at 100 kPa effective consolidation pressure.



(a)

(b)

Figure 4.9 Comparison of **(a)** Young's modulus and **(b)** stress path results for Leighton Buzzard Sand with de-aired water and Leighton Buzzard Sand with silicon oil at an effective consolidation pressure of 100 kPa.



(a)

(b)

Figure 4.10 **(a)** Stress-strain and, **(b)** pore water vs. strain curves for clean Leighton Buzzard Sand and that with various castor sugar solutions at different temperatures at 100 kPa effective consolidation pressure.

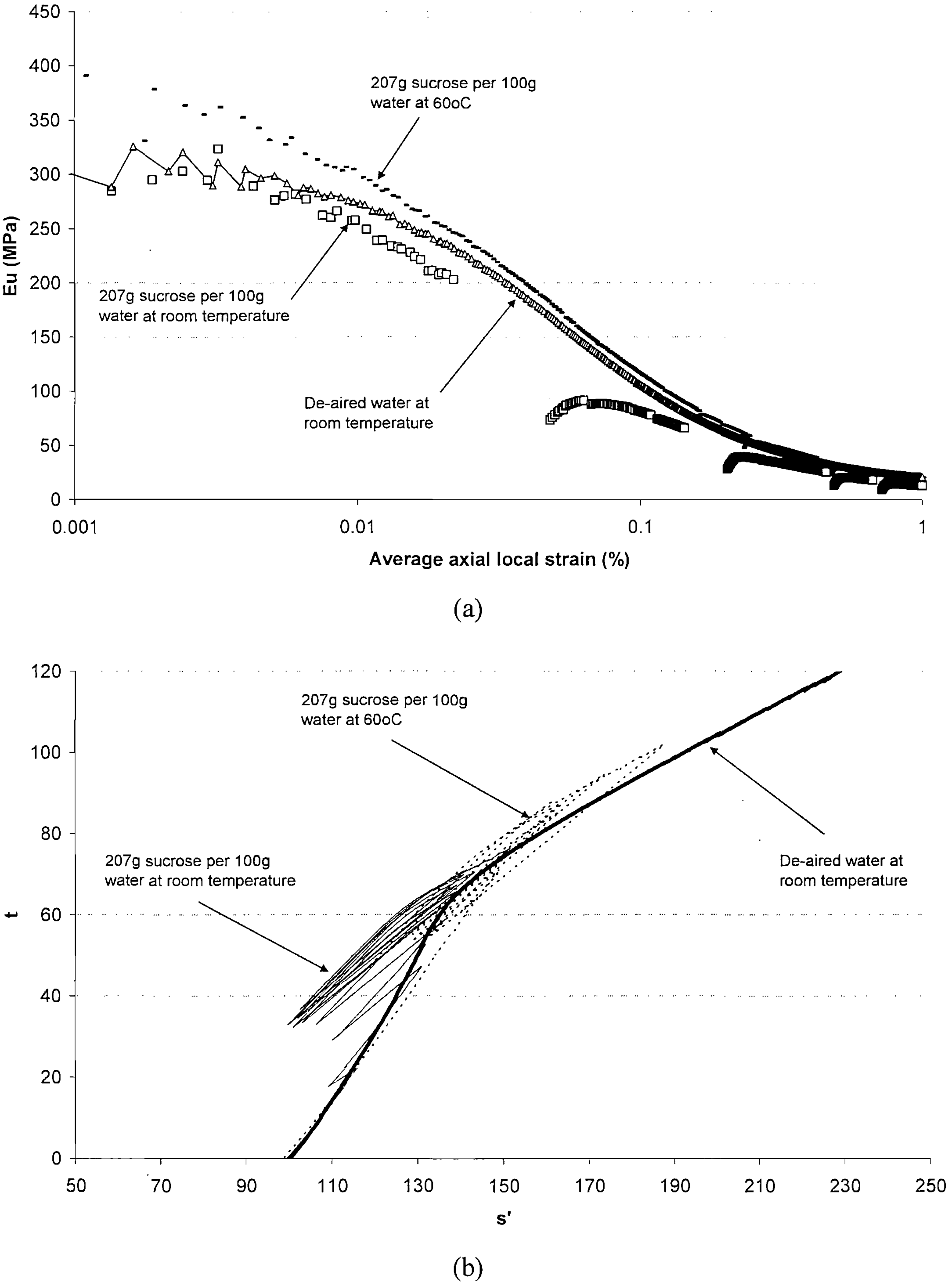
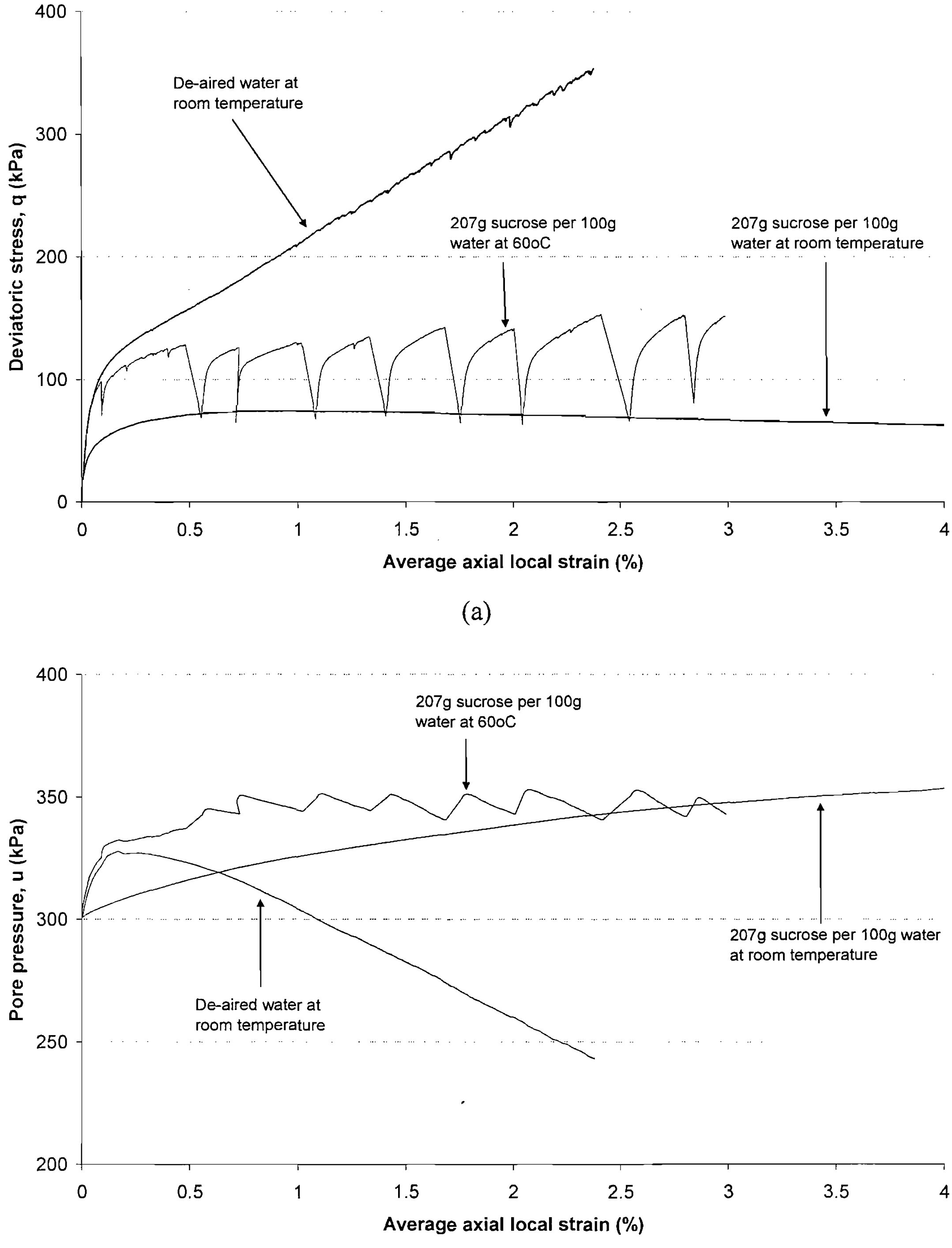


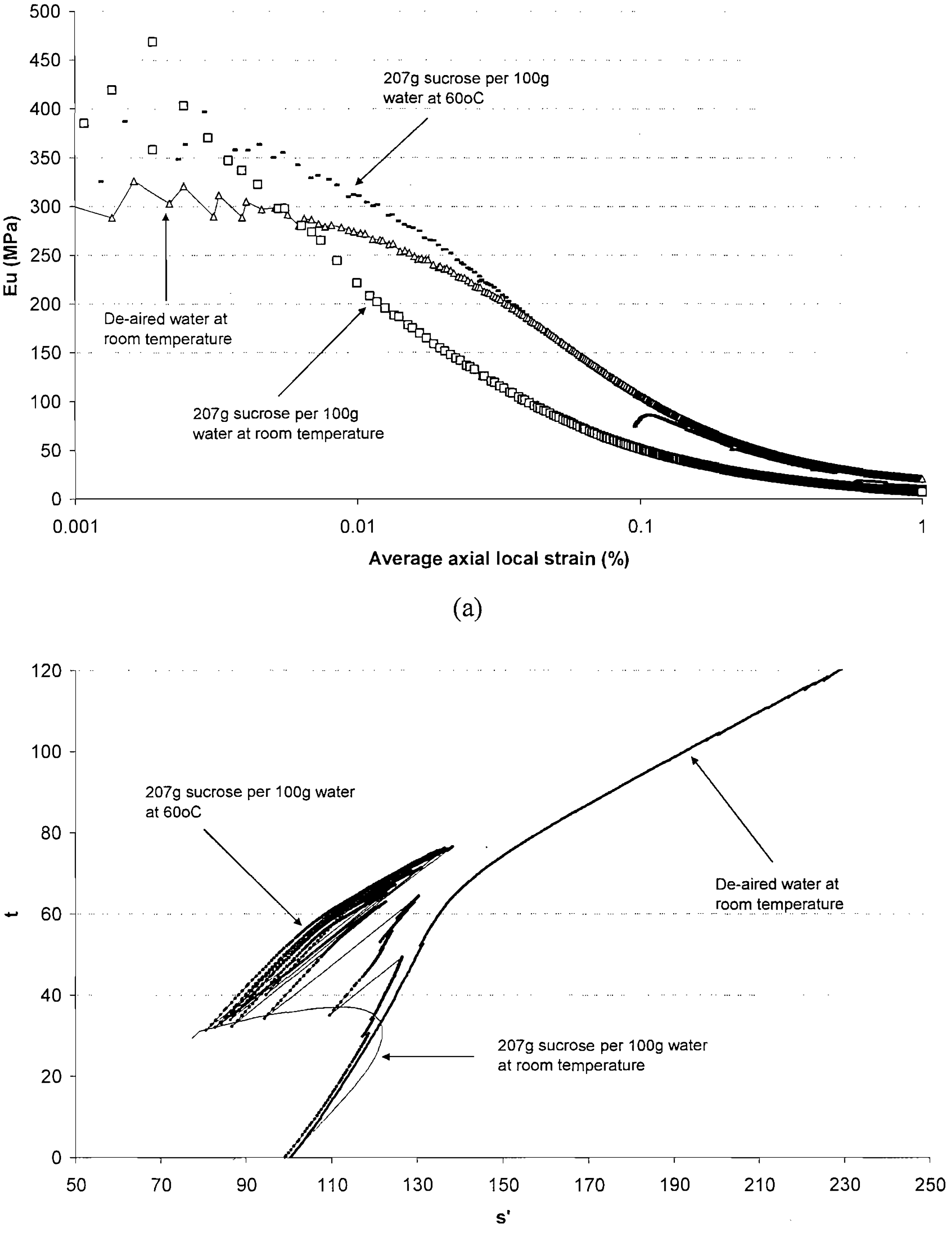
Figure 4.11 Comparison of **(a)** Young's modulus and **(b)** stress path results for Leighton Buzzard Sand with de-aired water and Leighton Buzzard Sand with castor sugar solutions at different temperatures at an effective consolidation pressure of 100 kPa.



(a)

(b)

Figure 4.12 **(a)** Stress-strain and, **(b)** pore water vs. strain curves for clean Leighton Buzzard Sand and that with various icing sugar solutions at different temperatures at 100 kPa effective consolidation pressure



(a)

(b)

Figure 4.13 Comparison of **(a)** Young's modulus and **(b)** stress path results for Leighton Buzzard Sand with de-aired water and Leighton Buzzard Sand with icing sugar solutions at different temperatures at an effective consolidation pressure of 100 kPa.

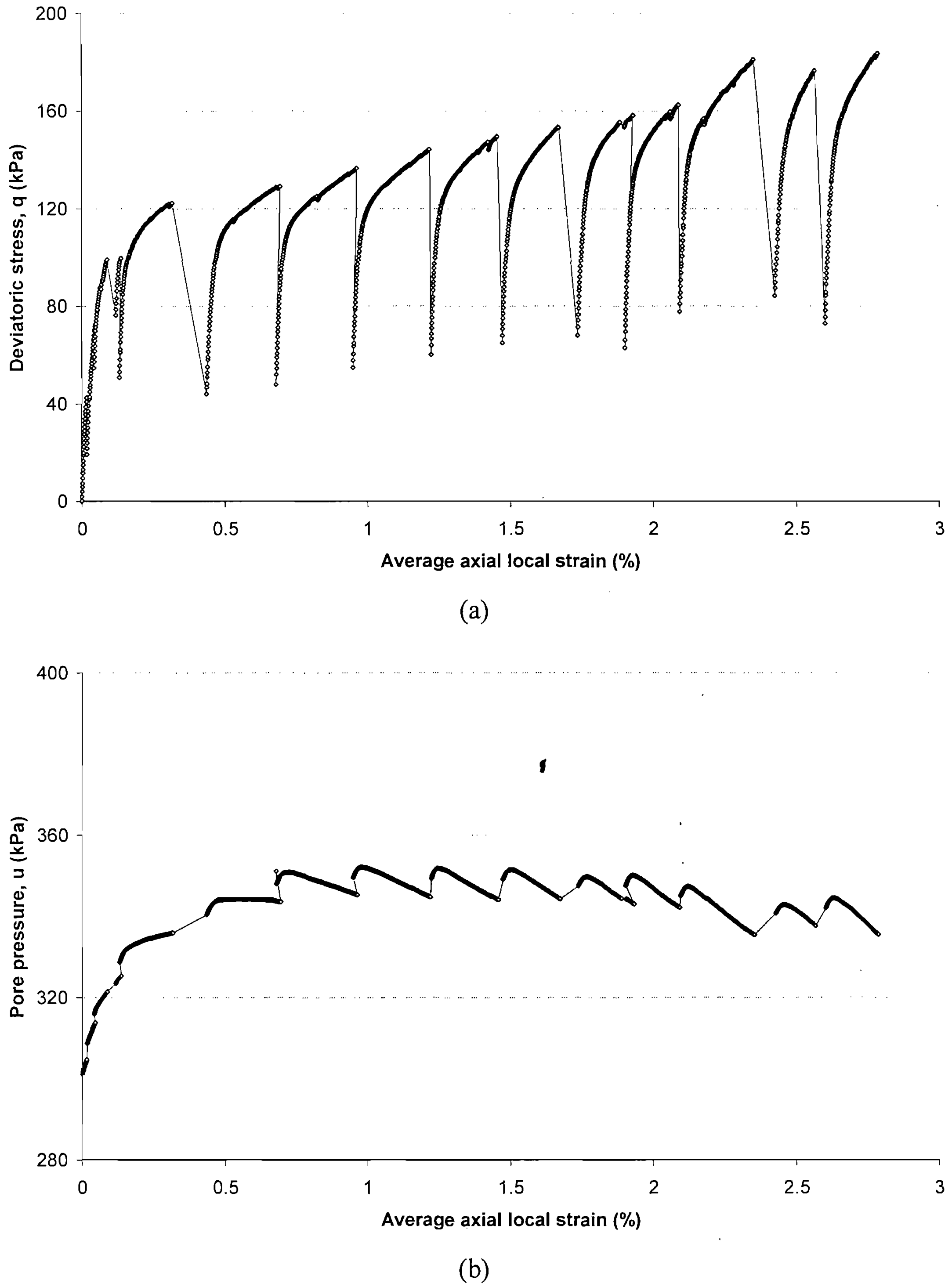
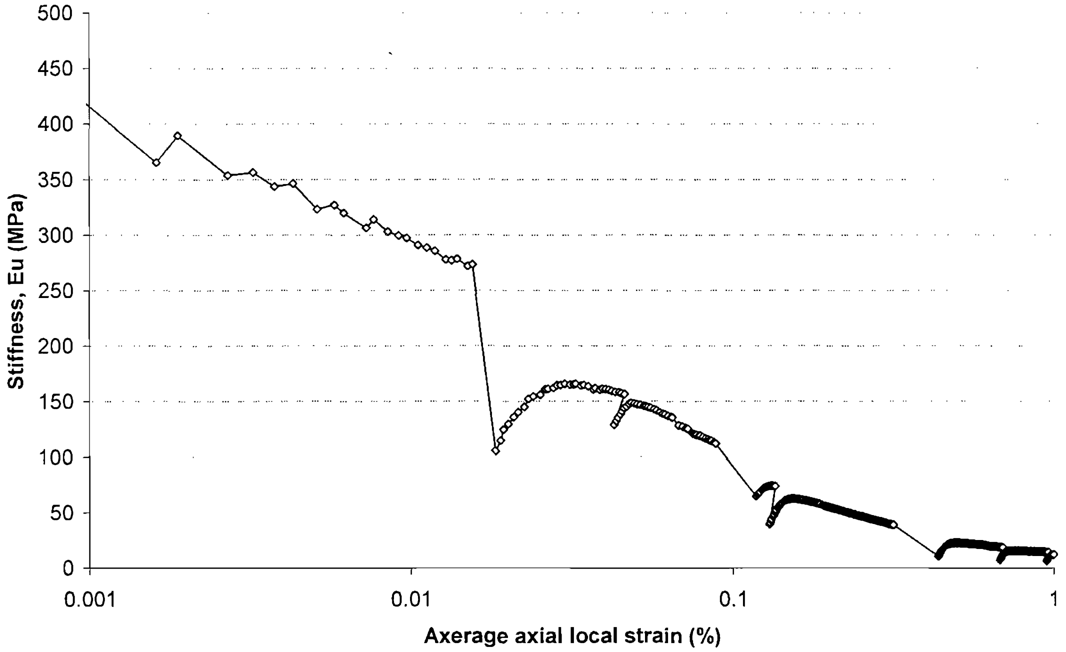
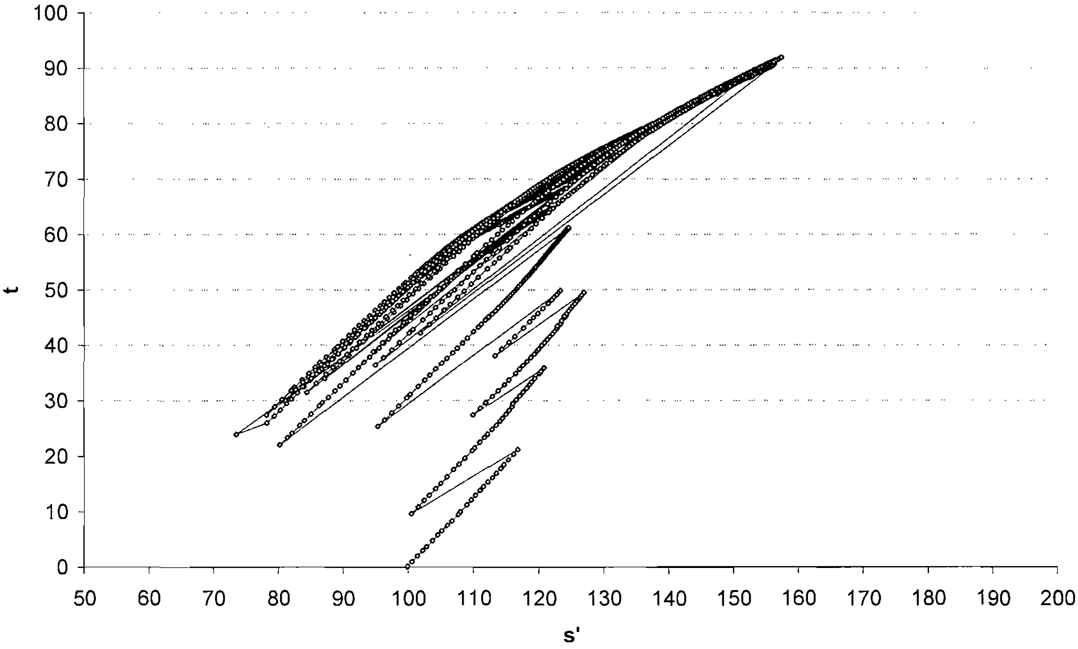


Figure 4.14 **(a)** Stress strain and **(b)** pore water vs. strain curve for Leighton Buzzard Sand with sucrose solution prepared by following method 1c, and tested at an effective consolidation pressure of 100 kPa.

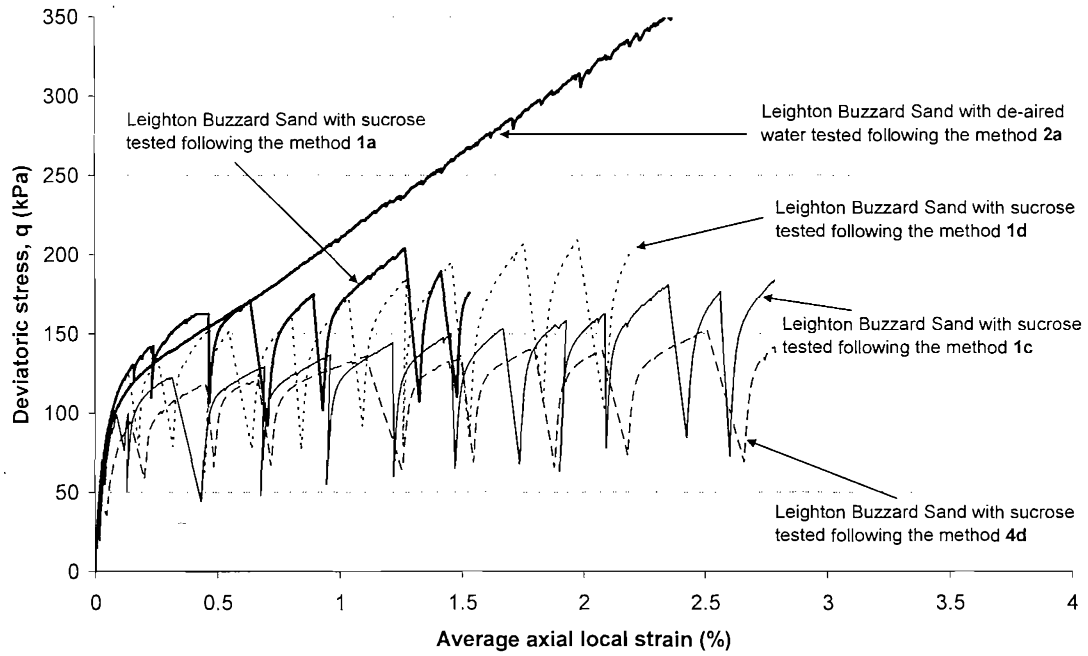


(a)

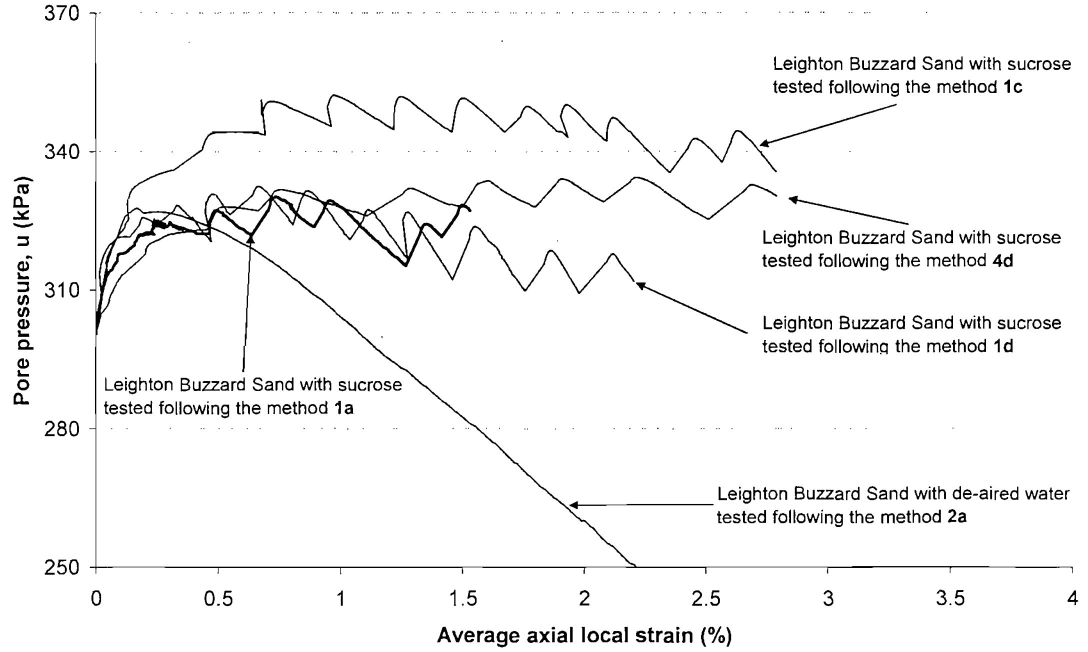


(b)

Figure 4.15 (a) Young's modulus, and **(b)** stress path results for Leighton Buzzard Sand with sucrose solution prepared by following method 1c, and tested at an effective consolidation pressure of 100 kPa.



(a)



(b)

Figure 4.16 (a) Stress-strain and, **(b)** pore water vs. strain curves for Leighton Buzzard Sand prepared by following different methods at 100 kPa effective consolidation pressure.

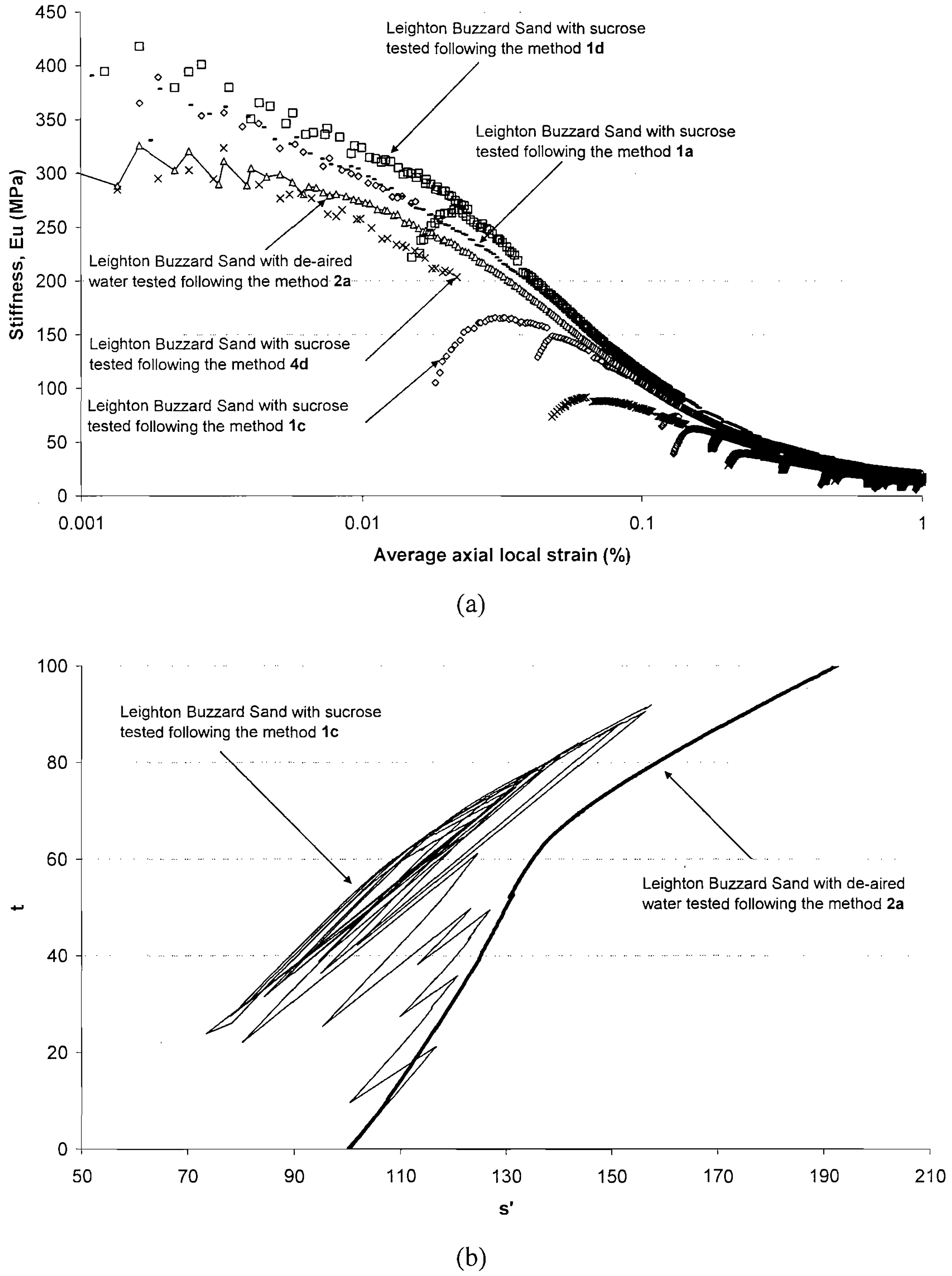
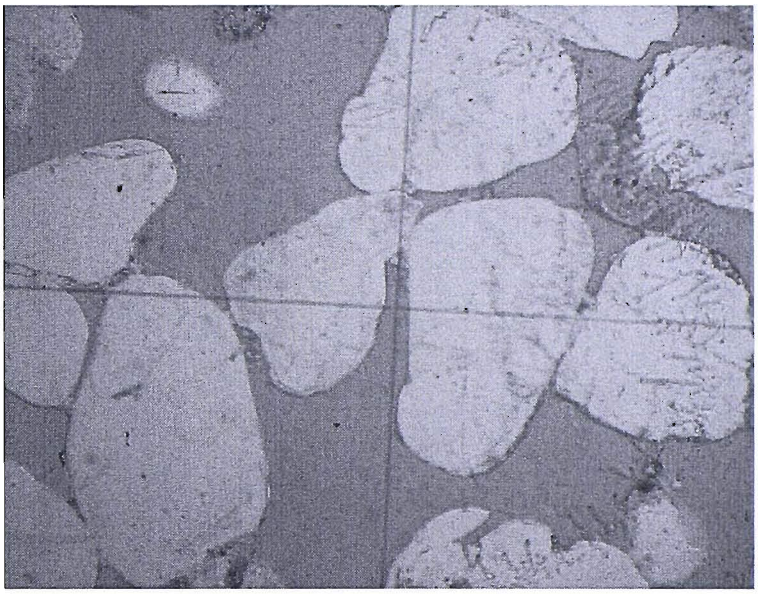
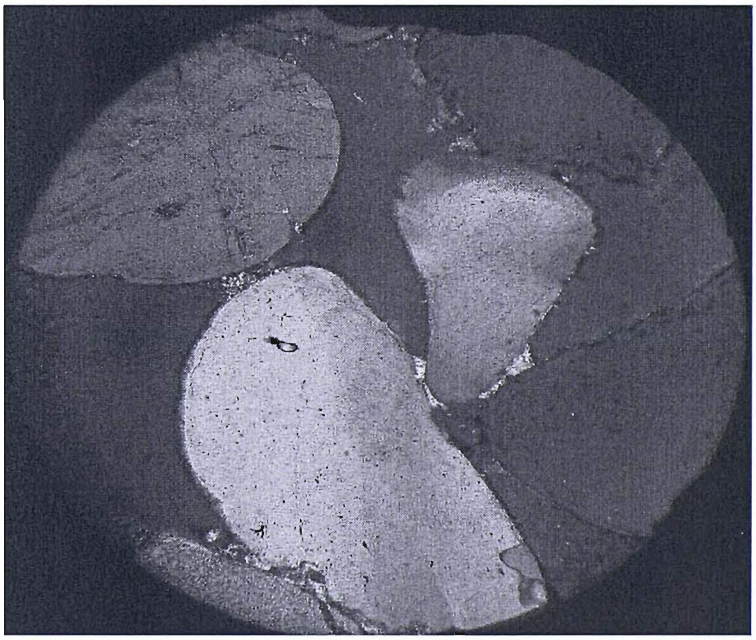


Figure 4.17 Comparison of **(a)** Young's modulus and **(b)** stress path results for Leighton Buzzard Sand with de-aired water and Leighton Buzzard Sand specimens prepared by following different methods at an effective consolidation pressure of 100 kPa.



(a)



(b)

Figure 4.18 A section of Leighton Buzzard Sand with sucrose solution containing 207g sucrose (i.e., castor sugar) per 100g water for **(a)** polarization, and **(b)** cross polarization.

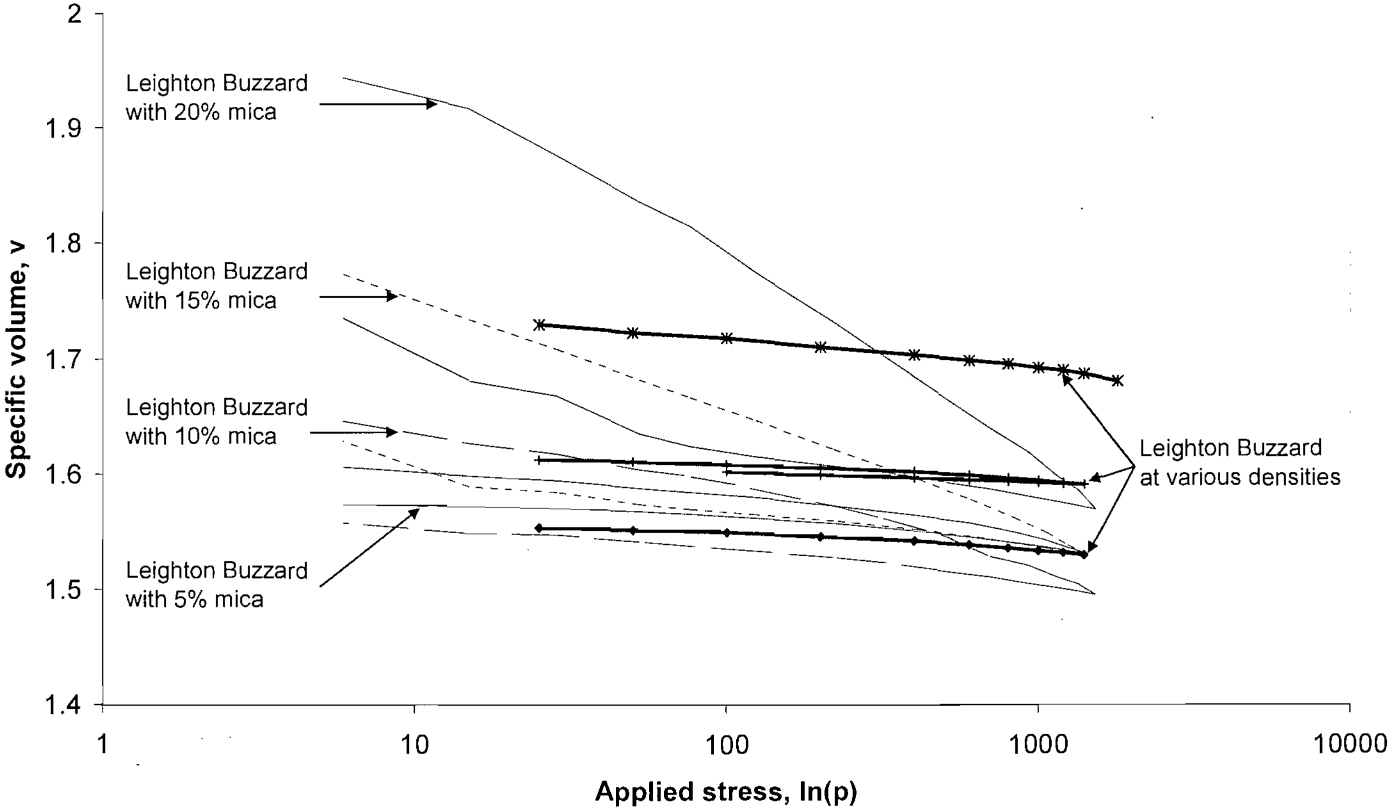
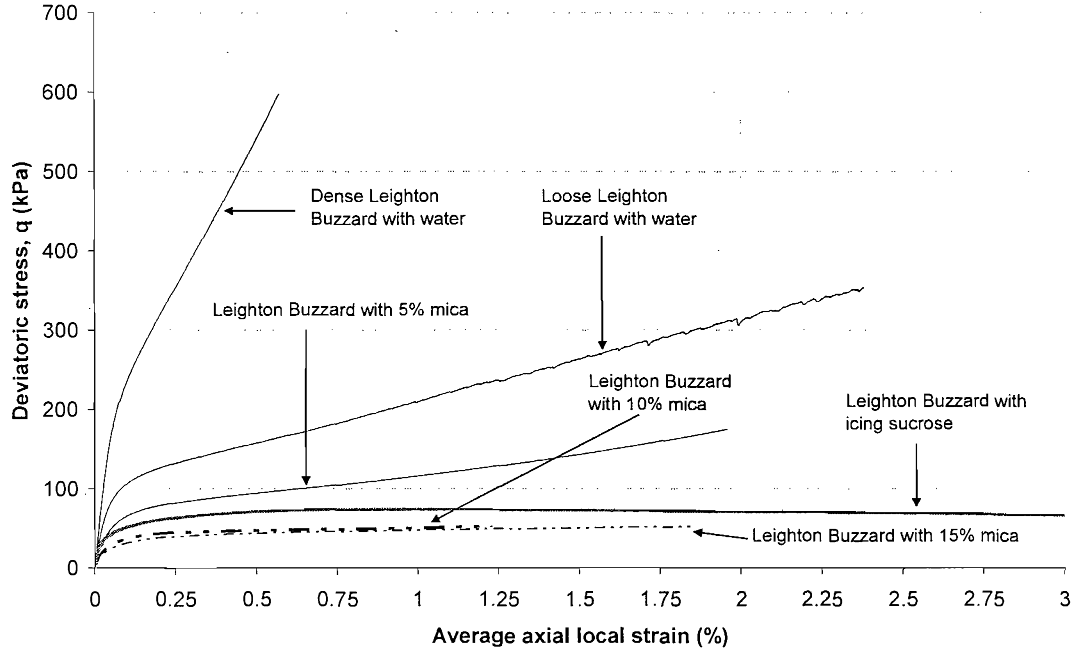
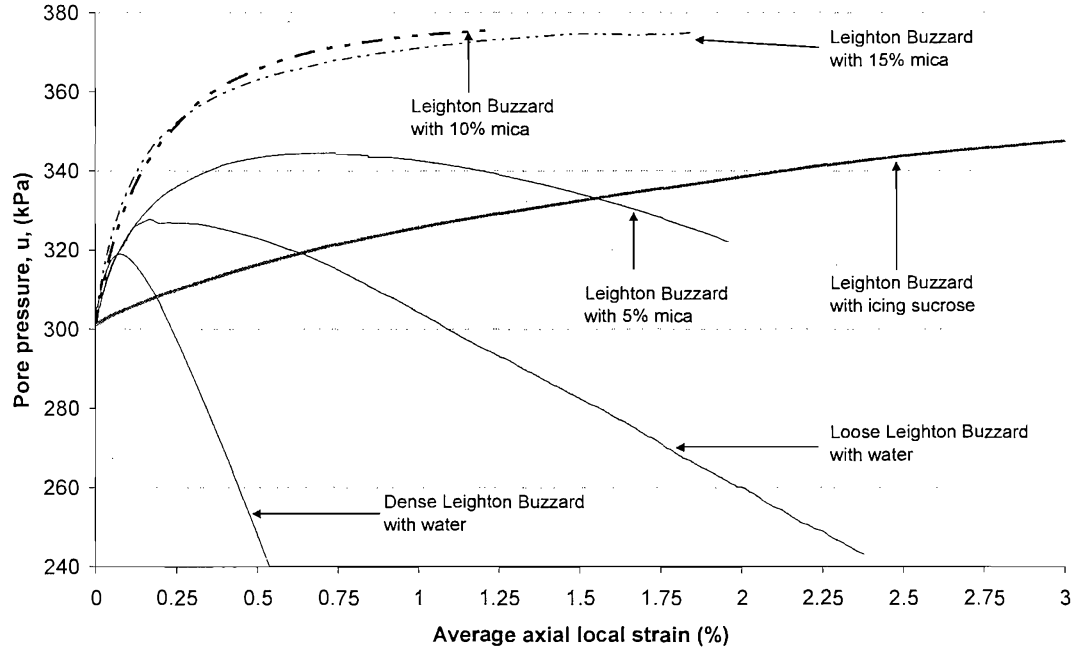


Figure 4.19 Specific volume vs. applied vertical stress for one-dimensionally consolidated rotund sand at various densities and rotund sand with various percentages of mica.



(a)



(b)

Figure 4.20 (a) Stress-strain, and **(b)** pore water vs. strain curves for clean Leighton Buzzard Sand at different densities and that mixed with various materials.

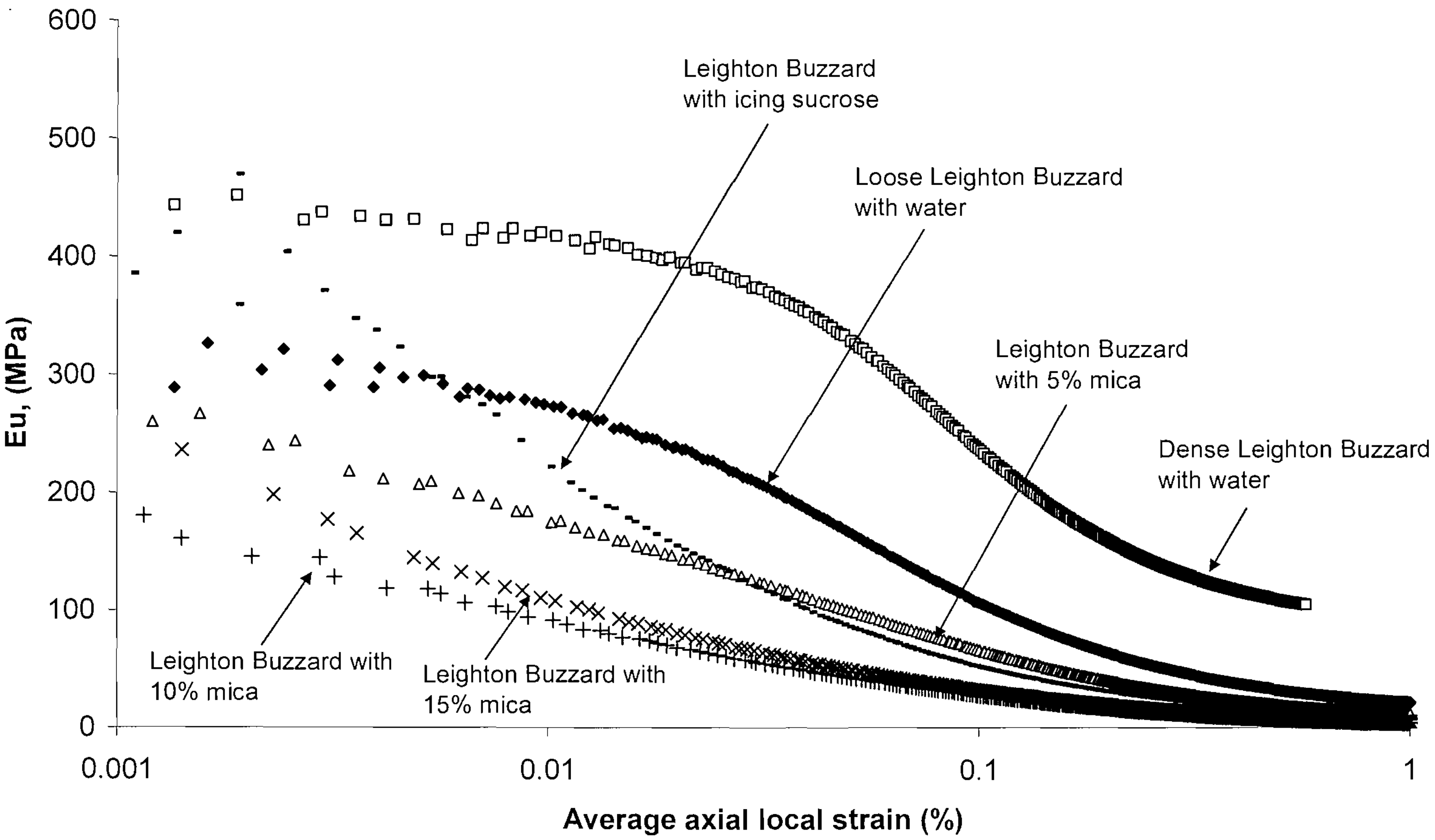


Figure 4.21.a Comparison of Young's modulus for various specimens at an effective consolidation pressure of 100 kPa.

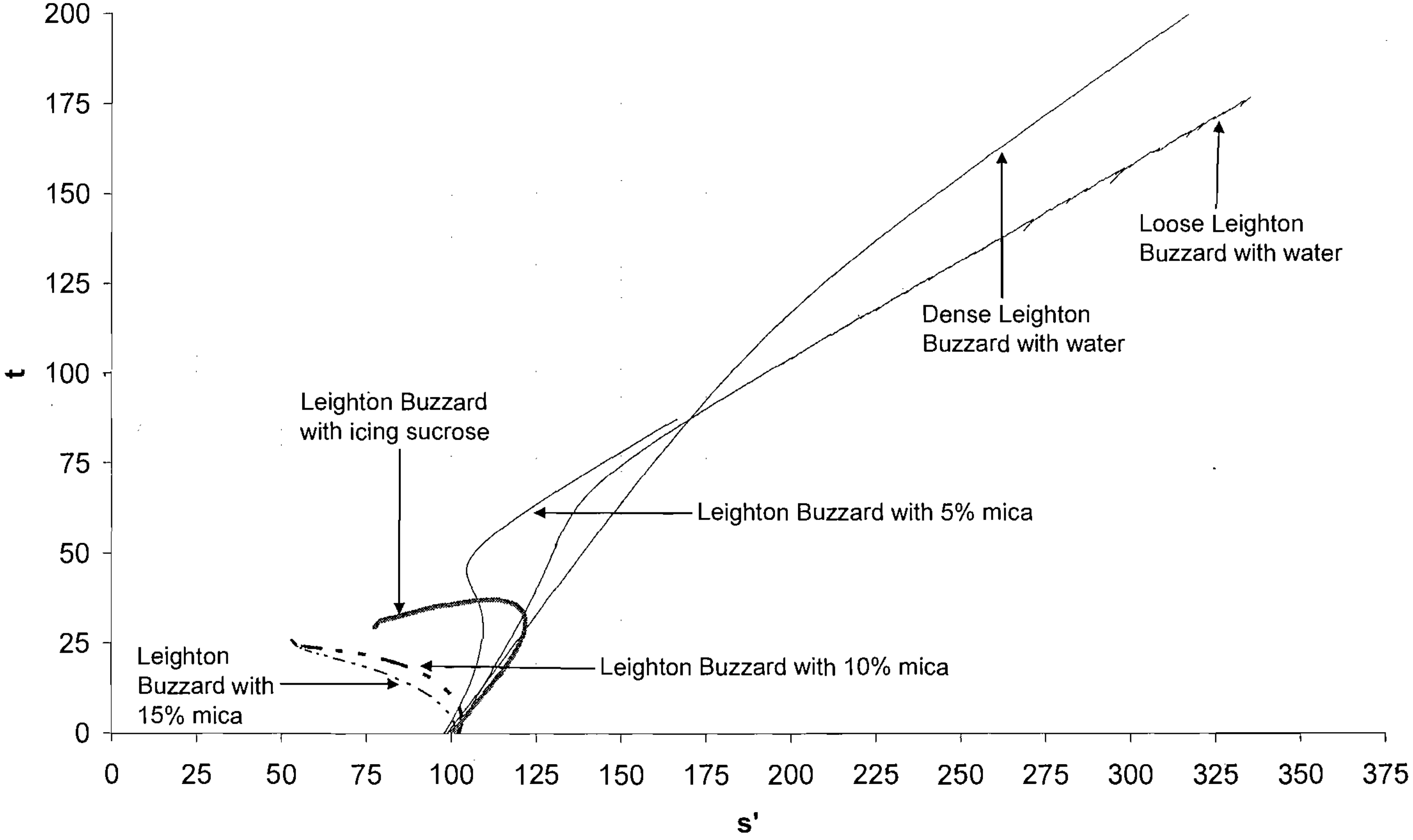


Figure 4.21.b Comparison of stress path results for various specimens at an effective consolidation pressure of 100 kPa.

CHAPTER 5

CONCLUSIONS

The objective of the study was to develop a greater understanding of the structural behaviour of geomaterials. In the light of the literature reviewed, it has been seen that there is a close linkage between many engineering characteristics of geomaterials and their structural properties. This fact indicates that any system of analysis or classification of soil behaviour in both the laboratory and field, which neglects the presence and effect of the structural constituents, will be incomplete. Two methods are likely to be employed to have a better understanding of the studies on structural behaviours; either the use of high quality samples of natural soils or that of artificially bonded geomaterials in a laboratory environment. Although, today, very high quality sampling techniques are available, these techniques may cause a relatively significant loss in structure mainly because of the disturbance in field, transportation and preparation for testing of the specimen in a laboratory.

Following the literature review of artificially bonding techniques for geomaterials, it was realised that the conventional method of artificially bonding techniques may introduce significant errors in the describing of microstructural influences. The errors may arise mainly from the compliance of the method that is achieved under zero effective stress and is followed by isotropic stress, which does not accurately simulate the geological process in the natural environment. Accordingly, it was decided to construct a modified triaxial apparatus for introducing of a more realistic artificially bonding technique. It has been also noticed that a separate framework for structured materials and a more accurate definition of bonding would be useful to extensively identify the structural properties of the geomaterials.

A series of experiments have been performed on Leighton Buzzard Sand with various sucrose solutions using the temperature controlled triaxial apparatus. Triaxial testing on the specimens resulted in sudden decreases followed by gradual increases in deviatoric stress. Deviatoric stress oscillations are observed at each Leighton Buzzard Sand specimen with oil as well as sucrose solutions differing in the deviatoric stress amplitude and the strain intervals corresponding to the each fluctuation. Interpretation on the test results reveals that the mechanism raised above could be attributed in many cases mainly to (i) the pore fluid characteristics (e.g., viscosity tube measurements), (ii) the higher number of contact point because of the higher relative density, (iii) crystal growth, (iv) the surface friction of the particles, (v) open fabric structure in the specimens, (vi) a compliance in the apparatus used in the experimental study, (vi) stick-slip behaviour between soil particles because of the distinctive characteristics of sucrose solutions.

The influence of sucrose solutions on the behaviour of rotund coarse particles was investigated by studying the change in amount of sucrose (i.e., castor sugar) in the Leighton Buzzard Sand specimens. It was observed that an increase in sucrose quantity at every ratio resulted in an increase in the bulk density and relative density. The deviatoric stress vs. strain, and pore pressure generation vs. strain behaviours of the specimens with sucrose solutions also change with the amount of sucrose particles added to the solution. The fluctuations take place earlier and more frequently as the amount of sucrose increases. In addition, bigger amplitude in deviatoric stress drops is observed at larger strain levels. Examining all the plots in this series further (Figure 4.1), it is observed that the initial amount of sucrose may give an insight to predict the characteristics of the remaining test.

This successive formation gives a stick-slip behaviour nature to the fluctuations. During the sticking phase, the sand grains are more closely packed and the sample exhibits a gradual increase in deviatoric stress, however; when the soil matrix becomes relatively unstable, some grains slide out of the column resulting in the deviatoric stress to sharply drop. The deviatoric stress subsequently builds up so as to support an applied stress. Taking a close look at the test results (e.g., Figure 4.1.a) showing the change in deviatoric stresses with strain, one can note that the deviatoric stress amplitude shows an overall increases as the sucrose content decreases. It is seen that sharp drops in deviatoric stresses occur later as the sucrose content in the mixtures decreases. While deviatoric stress increases, pore water pressure decreases gradually. The test results on the specimens also result in fluctuations in stiffness and an area in stress plots.

It is postulated that force chain networks within the specimens tested in the triaxial apparatus lead to the effects observed in the experimental study. While the increase in deviatoric stress develops, geometry of the force chain network in the specimens with water as pore fluid is unstable in time domain that leads to homogeneity in stress distribution because of the higher contact numbers between the particles. A deviatoric stress increase in a specimen tested using water causes a contact distribution that is relatively quicker than the gradually occurring contact distribution in the other specimens tested using sucrose solutions and oil. During the gradual increase in deviatoric stress in the specimens with sucrose solutions and oil, the force chain network is relatively stable in time domain that leads to a relatively non-homogenous stress distribution because of the less contact numbers developing between the particles by the time. This mechanism implies that, up until a sudden drop in deviatoric stress where there may be a kind of threshold energy, the force chain network in the specimens tested using sucrose solution and oil is stronger than the force chain network in the specimens tested using water as pore fluid. That is why stress distribution in the specimens with water is relatively quicker than the specimens tested using sucrose solutions and oil. It has been also noted that the later the stress in a force chain network distributes, the bigger the amplitude of the drop in stress that is observed.

An experimental programme was also followed to understand the change in the behaviour of Leighton Buzzard Sand produced by a silicon oil (i.e., Dow Corning 200/50) pore fluid. Despite the fact that Leighton Buzzard Sand specimen with oil and that with de-aired water were prepared by following similar methods, the obtained bulk densities, dry densities and relative densities for the Leighton Buzzard Sand at these two different tests were significantly different from each other. This was considered to be the result of lubrication being set up within the soil particles. Similar to the specimens tested with sucrose solutions, the Leighton Buzzard Sand with oil also gives a series of fluctuations with a more limited range (Figures 4.8, and 4.9).

Influence of temperature on the behaviour of Leighton Buzzard Sand particles was studied on the Leighton Buzzard Sands with sucrose solutions (i.e., castor sugar, and icing sugar) at elevated temperature (i.e., 60°C) and room temperature individually. For the tests where the same materials and apparatus were used, it was observed that a change in temperature seems to affect the stress-strain relationship. While the effects of temperature on the Leighton Buzzard Sand with castor sugar was observed within a reasonable range (Figures 4.10, and 4.11), the effects of temperature on the Leighton Buzzard Sand with icing sugar produced a significant difference in mechanical behaviour comparing to the Leighton Buzzard Sand with de-aired water (Figures 4.12, and 4.13).

During the investigation on the mechanical properties of Leighton Buzzard Sand with castor sugar solutions, four different methods (i.e., 1a, 1c, 1d, and 4d) were employed for the same type of mixtures. Comparing the results of these testing methods, it was observed that the methods followed in this research have no significantly different effect on behaviour (Figures 4.16, and 4.17).

Thin section pictures of the Leighton Buzzard Sand tested with a castor sugar solution were taken to observe the sucrose crystals growing between the soil particles. Sucrose crystals growth between adjacent Leighton Buzzard Sand particles appeared to be a solid bridge for bonding the rotund particles to each other (Figure 4.18). It is interpreted that clean contacts between the Leighton Buzzard Sand particles seem to decrease with amount of sucrose content. This may imply that governing constituent change from Leighton Buzzard to sucrose crystals. The difference in the behaviour of the Leighton Buzzard Sand with sucrose solution and that with oil could be attributed to the sucrose crystal growth, and some other distinctive characteristics of sucrose solutions. In conclusion, this may imply that viscosity tube measurements analysis results should not be applied blindly for such experimental investigations.

The study also provides an additional data set to compare the mica-Leighton Buzzard Sand mixtures with icing sugar- Leighton Buzzard Sand mixtures in the triaxial apparatus. The intension was to draw a parallel between the behaviour of the Leighton Buzzard Sand mixtures with mica and icing sugar separately. The test results on mica and Leighton Buzzard Sand show that the characteristics of the sands tested may be ascribable to the presence of the flat grains in the specimens tested in oedometer (Figure 4.19). From the 1-dimensional consolidation results, it was found that the presence of platy particles in the specimens tested had a marked effect on the compressibility of the material under load, on its expansive after release or pressure and other clay-like behaviours. In the light of the Theron (2004), the author postulates that mica particles occupy the voids between coarse rotund sand particles. Based on the amount of mica particles present, the Leighton Buzzard Sand particles are in contact with each other and the behaviour of the samples tested are controlled by Leighton Buzzard Sand particles (Figures 4.20, and 4.21). When the contacts between the Leighton Buzzard Sand particles reduce, the behaviour of the samples becomes to clay like. From the triaxial test results on the Leighton Buzzard Sand with sucrose (i.e., icing sugar) solution at room temperature, a contraction behaviour developing at the measured strain level was observed. The result was attributed to the presence of (i) undissolved sucrose particles, and (ii) tricalcium phosphate (Table 3.2).

CHAPTER 6

RECOMMENDATIONS FOR FUTURE WORKS

These studies have identified various important aspects of microstructure on the mechanical behaviour of geomaterials, which warrant additional research that could be described as follows.

1. Within the scope of these studies, for the sake of simplicity and robustness, a number of conventional consolidated undrained triaxial compression experiments at one confining (400 kPa) and back pressure (300 kPa) values, and at a constant displacement 0.015 mm/min were adopted as representative values. However, improvements to this procedure will be achieved by fully representing the variability of rate of loading and types of testing are possible.
2. These studies were conducted using various pore fluids (i.e., de-aired water, sucrose solutions, silicon oil) and fines (i.e., mica, sucrose) under various temperatures. These studies could be extended to use other pore fluids (e.g., polymer based chemical/biological materials) and fines (e.g., Leighton Buzzard fraction E) for a more accurate estimation of soil behaviour.
3. The process that governs the stick-slip mechanism observed in the experimental study is accompanied by mechanical noise. Therefore, the availability of a sound recorder and implementation of a sound analysis setup to analyze the behaviour

of geomaterials may enable further experiments to address new aspect of microstructural level behaviour.

4. An extensive investigation on the influence of system compliance could be pursued as a new research topic to eliminate the effects of apparatus on the behaviour of soils tested.
5. Finally, having understood the 'stick-slip' mechanism, efforts could be focused on developing a probabilistically-based approach for the estimation of soil deformation.

REFERENCES

Acar, B.Y. and El-Tahir A.E. (1986). Low strain dynamic properties of artificially cemented sands. Journal of Geotechnical Engineering Division, A.S.C.E., Vol. 112, No. 11, 1001-1015.

Adjemian, F. and Evesque, P. (2004). Experimental study of stick-slip behaviour. Int. J. Numer. Anal. Meth. Geomech., Vol. 28, pp. 501-530.

Airey, D.W. (1993). Triaxial testing of naturally cemented carbonate soil. Journal of Geotechnical Engineering Division, A.S.C.E., Vol. 119, No. 9, 1379-1398.

Albert, I., Tegzes, P., Kahng, B., Albert, R., Sample, J.G., and Pfeifer, M. (2000). Jamming and fluctuations in granular drag. Physical Review Letters, Vol. 84, No. 22, 5122-5125.

Albert, I., Tegzes, P., Albert, R., Sample, J.G., Barabasi, A.L., Vicsek, A.L., Kahng, B., and Schiffer, P. (2001). Stick-slip fluctuations in granular drag. Physical Review E., Vol. 64, No. 031307.

Al Hattamleh, O., Muhunthan, B., and Zbib, H.M. (2005). Stress distribution in granular heaps using multi-slip formulation. Int. J. Numer. Anal. Meth. Geomech, Vol. 29, 713-727.

Alshibli, K.A., and Roussel, L.E. (2006). Experimental investigation of slip-stick behaviour in granular materials. Int. J. Numer. Anal. Meth. Geomech (in press).

Allman. M.A. and Poulos, H.G. (1988). Stress-stress behaviour of an artificially cemented calcareous soil. Proceedings of the "International Conference on

Calcareous Sediments", Perth, Jewell and Andrews (eds), Balkema: Rotterdam, Vol. 2, 51-58.

Atterberg, A. (1919). *Die plastizität und bindigkeit liefernden bestandteile der tone, Intern. Mitt fur Bodenkunde.*

Aversa, S. (1991). Mechanical behaviour of soft rocks: some remarks. Proceedings of the Workshop on "Experimental Characterization and Modelling of Soils and Soft Rocks", Napoli, Italy, 191-223.

Aversa, S., Evangelista, A., Leroueil, S. and Picarelli, L. (1993). Some aspects of the mechanical behaviour of structured soils and soft rocks. In Geotechnical Engineering of Hard Soils-Soft Rocks, A. Anagnostopoulos et al., (eds), 359-366. Rotterdam: Balkema.

Aylmore, L. and Quirk, J.P. (1960). Domain or turbostratic structure of clays. Nature, Lond. 187, 1046.

Bak, P., Tang, C., Wiesenfeld, K. (1987). Self-organized critically: An explanation of 1/f noise. Physical Review Letters, Vol. 59, No. 4, 381-384.

Baldi, G., Hight, D.W., and Thomas, G.E. (1998). A Re-evaluation of conventional triaxial test methods, 'Advanced Triaxial Testing of Soil and Rock, ASTM STP 977', R.T. Donaghe, R.C, Chaney, and M.L. Silver (eds), American Society for Testing and Materials, Philadelphia, 219-263.

Barden, L. and Sides, G. (1971). Sample disturbance in the investigation of clay structure. Géotechnique, Vol. 21, No. 3, 211-222.

Barden, L. (1973). Macro and microstructure of soils, Appendix to the Proceedings of the International Symposium on Soil Structure, Swedish Geotechnical Society, Swedish Society for Clay Research, Gothenburg, 21-26.

Barrett, P. J. (1980). The shape of rock particles, a critical review. Sedimentology, Vol. 27, 291-303.

Barton, M. E. (1993). Cohesive sands: the natural transition from sands to sandstones. Geotechnical Engineering of Hard Soils-Soft Rocks, Anagnostopoulos et al., (eds), Balkema: Rotterdam, 367-374.

Baudet B., and Stallebrass, S. (2004). A constitutive model for structured clays. Géotechnique, Vol. 54, No. 4, 269-278.

Behner, U. J. A. (1998). Recipes for Growing Sugar Crystals http://www. crystalgrowing. com/index_e.htm (28 April 2006).

Bellotti, R., Jamiolkowski, D., Lo Presto, D. C. F. and O'Neill, D. A. (1996). Anisotropy of small strain stiffness in Ticino sand. Géotechnique, Vol. 46, No. 1, 115-131.

Biot, M. A. (1956a). Theory of propagation of elastic waves in a fluid-saturated porous solid. I. Low frequency range. J. Acoustic Soc of America, No. 28, 168- 178.

Biot, M. A. (1956b). Theory of propagation of elastic waves in a fluid-saturated porous solid. II. High frequency range. J. Acoustic Soc of America, No. 28, 179- 191.

Bjerrum, L. (1954). Geotechnical properties of Norwegian marine clays. Géotechnique, No. 4, 49-69.

Bjerrum, L. (1967). Engineering Geology of Norwegian normally-consolidated marine clays as related to settlement of buildings. 7th Rankine Lecture, Géotechnique, Vol. 17, No. 2, 81-118.

Blyth, F. G. H. and de Freitas, M. H. (1984). A Geology for Engineers. Edward Arnold (Australia) Pty Ltd, 80 Waverly Road, Caulfield East, Victoria 3141, Australia.

Bolt, G. H. (1956). Physico-chemical analysis of the compressibility of pure clays. Géotechnique, Vol. 6, No. 1, 86-93.

Bolton, M.D. (2000). The role of micro-mechanics in soil mechanics. International Workshop on Soil Crushability Yamaguchi University, Japan, July 1999. CUED/D-Soils/TR313 September 2000.

Bouissou, S., Petit, J.P., and Barquins, M. (1998). Experimental evidence of contact loss during stick-slip: possible implications for seismic behaviour. Tectonophysics, Vol. 295, 341-350.

Brace, W.F. and Byerlee, J.D. (1966). Stick-slip as a mechanism for earthquakes. Science, Vol. 153, 990-992.

Braun, O.M. and Naumovets, A.G. (2006). Nanotribology: Microscopic mechanics of friction. Surface Science Reports, Vol. 60, 79-158.

Bressani, L. A. (1990). Experimental properties of bonded soils, PhD thesis, University of London.

BS 5930: (1990). Code of Practice for Site Investigations, British Standards Institution, London.

BS 1377; Part 4: (1990). British Standard Methods of test for soils for civil engineering purposes, Part 4: Compaction-related tests, British Standard Institution, London.

Burland, J. B. (1990). On the compressibility and shear strength of natural clays. Géotechnique, Vol. 40, No. 3, 329-378.

Burland, J.B., Jardine, R.J., and Symes, M.J. (1984). The measurements of soil stiffness in the triaxial apparatus. Géotechnique, Vol. 34, No. 3, 323-340.

Burland, J. B. and Symes, M. (1982). A simple axial displacement gauge for use in the triaxial apparatus. Géotechnique, Vol. 32, No. 1, 62-65.

Burridge, R. and Knopoff, L. (1967). Model and theoretical seismicity. Bulletin of Seismological Society of America, Vol. 57, No. 3, 341-371.

Çabalar, A.F. (2002). Ground improvement by bacteria and synthetic polymers. MSc thesis, University of Gaziantep, Turkey.

Çabalar, A.F. and Çanakcı, H. (2005). Ground improvement by bacteria. Abousleiman, Y., Cheng, A.H.-D., and Ulm, F.-J. (eds.), Poromechanics III- Biot Centennial (1905-2005), Proceedings of the 3rd Biot Conference on Poromechanics, A. A. Balkema, Leidon / London / New York / Philadelphia/ Singapore, 828 p., Norman, Oklahoma, USA.

Çabalar A. F. (2006). A Note on the Influences of Bacteria on the Compressibility of a Sand Matrice, Geotechnical Engineering-New Frontiers& Challenges, the 9th Young Geotechnical Engineers' Conference, 4- 6 September 2006, Belfast, UK.

Çabalar, A.F. (2007). Discussion of "Biological Considerations in Geotechnical Engineering" by J.K. Mitchell and J.C. Santamarina, ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2005, Vol. 131, No.10, 1222-1233 (in press).

Cain, R.G., Page, N.W., and Biggs, S. (2001). Microscopic and macroscopic aspects of stick-slip motion in granular shear. Physical Review E, Vol. 64, 016413.

Callisto, L. and Calabresi, G. (1998). Mechanical behaviour of a natural soft clay. Géotechnique, Vol. 48, No. 4, 495-513.

Canestrari, F. and Scrapelli, G. (1993). Stress-dilatancy and strength of Ancona clay. In Geotechnical Engineering of Hard Soils-Soft Rocks (A. Anagnostopoulos et al., eds), Athens, Greece, Balkema: Rotterdam, pp. 417-424.

Casagrande, A. (1932). The structure of clay and its importance in foundation engineering. J. Boston Soc. Civ. Engrs., Vol. 19, No. 4, 168-209.

Cates, M.E., Wittmer, J.P., Bouchaud, J.P., and Claudin, P. (1998). Jamming, Forces Chains, and Fragile Matter. Physical Review Letters, Vol. 81, No. 9. 1841-1844.

Chandler, R. J. (2000). Clay sediments in depositional basins: the geotechnical cycle. Quarterly Journal of Engineering Geology and Hydrogeology, Vol. 33, 7-39.

Clayton, C.R.I. and Khatrush, S.A. (1986). A new device for measuring local axial strains on triaxial specimens. Géotechnique, Vol. 36, No. 4, 593-597.

Clayton, C. R. I., Khatrush, S. A., Bica, A. V. D., and Siddique, A. (1989). The use of hall effect semi conductors in geotechnical instrumentation. Geotechnical Testing Journal, Vol. 12, Mar., 69-76.

Clayton, C. R. I., Hight, D. W., and Hopper, R. J. (1992). Progressive destructuring of Bothkennar clay: implications for sampling and reconsolidation procedures. Géotechnique, Vol. 42, No. 2, 219-240.

Clayton, C. R. I. and Serratrice, J. F. (1993). The mechanical properties and behaviour of hard soils and soft rocks. In Geotechnical Engineering of Hard Soils-Soft Rocks (A. Anagnostopoulos et al., eds), Athens, Greece, Balkema: Rotterdam, 1839-1877.

Clayton, C. R. I., Matthews, M. C., and Simons, N. E. (1995). Site Investigation. Blackwell Science Ltd., Second edition, Printed and bound in Great Britain.

Clayton, C. R. I. and Heymann, G. (2001). Stiffness of geomaterials at very small strains. Géotechnique, Vol. 51, No. 3, 245-255.

Clayton, C. R. I., Theron, M., and Vermeulen, N. J. (2004). The effect of particle shape on the behaviour of gold tailings. Advances in Geotechnical Engineering: The Skempton Conference, Thomas Telford, London, 393-404.

Clough, G.W., Kuck, W.M. and Kasali, G. (1979). Silicate-stabilized sands. Journal of the Geotechnical Engineering Division, A.S.C.E., Vol. 105, No. GT1, 65-82.

Clough, G.W., Sitar, N., Bachus, R.C., and Shaffi Rad, N. (1981). Cemented sands under static loading. Journal of the Geotechnical Engineering Division, A.S.C.E. Vol. 107, No. GT6, 799-817.

Coop, M.R. and Atkinson, J.H. (1993). The mechanics of cemented carbonate sands. Géotechnique 43, No.1, 53-67.

Cotecchia, F. (1996). The effects of structure on the properties of an Italian Pleistocene clay. PhD thesis, University of London.

Cotecchia, F. and Chandler, R.J. (1995). Geotechnical properties of the Pleistocene clays of the Pappadai Valley, Taranto, Italy. Quarterly Journal of Engineering Geology, Vol. 28, 5-22.

Cotecchia, F. and Chandler, R.J. (1997). The influence of structure on the pre-failure behaviour of natural clay. Géotechnique, Vol. 47, No. 3, 523-544.

Cotecchia, F. and Chandler, R.J. (2000). A general framework for the mechanical behaviour of clays. Géotechnique, Vol. 50, No. 4, 431-447.

Collins, K. and McGown, A. (1974). The form and function of microfabric features in a variety of natural soils. Géotechnique, Vol. 24, No. 2, 223-254.

Craig, R.F. (1983). Soil Mechanics. Van Nostrand Reinhold (UK) Co. Ltd. Printed and bound in Hong Kong.

Craig, R.F. (1997). Soil Mechanics, 6th edition, E and FN Spon, New York.

Crawford, C.B. (1986). State of the Art: Evaluation and interpretation of Soil Consolidation Tests, Consolidation of Soils: Testing and Evaluation, ASTM STP 892, R.N. Yong and F.C. Townsend (eds), American Society for Testing and Materials, Philadelphia, 71-103.

Cresswell, A. (1999). Sampling and strength testing an unbonded locked sand. PhD thesis, University of Southampton, UK.

Cresswell, A., Barton, M.E., Brown, R. (1999). Determining the maximum density of sands by pluviation. Geotechnical Testing Journal, GTJODJ, Vol. 22, No. 4, December, 324-328.

Cresswell, A., and Powrie, W. (2004). Triaxial tests on an unbonded locked sand. Géotechnique, Vol. 54, No. 2, 107-115.

Cuccovillo, T. and Coop, M.R. (1993). The influence of bond strength on the mechanics of carbonate soft rocks. In Geotechnical Engineering of Hard Soils-Soft Rocks (A. Anagnostopoulos et al., eds), Athens, Greece, Balkema: Rotterdam, 447-455.

Cuccovillo, T. and Coop, M.R. (1999). On the mechanics of structured sands. Géotechnique, Vol. 49, No. 6, 741-760.

Cuccovillo, T. and Coop, M.R. (1997). The measurement of local axial strains in triaxial tests using LVDTs. Géotechnique, Vol. 47, No. 1, 167-171.

Das, B.M. (1990). Principles of Geotechnical Engineering. PWS-KENT Publishing Company.

Demirel, A.L. and Granick, S. (1996). Friction fluctuations and friction memory in stick-slip motion. Physical Review Letters, Vol. 77, No. 21, 4330- 4333.

DeJong, J.T., Fritzges, M.B., and Nusslein, K. (2006). Microbially induced cementation to control sand response to undrained shear. A.S.C.E. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 132, No. 11, 1381- 1392.

Dobereiner, L. and De Freitas, M.H. (1986). Geotechnical properties of weak sandstones. Géotechnique, Vol. 36, No. 1, 79-94.

Duchesne A. (1998). *Etude du Comportement Mécanique d'un Combustible Granulaire soumis á des Sollicitations d'origine Thermique dans un Propulseur Thermonucl!éeaire Spatial.* Ph.D. Thesis, Ecole Centrale Paris, 1998; 116–1327.

Dumbleton, M.J. (1981). The classification and description of soils for engineering purposes: its development and relation to other comparable systems. TRRL Report LR 1030, Transport and Road Research Laboratory, Crowthorne, Berks.

Ellis, E.A., Soga, K., Bransby, M.F., and Sato, M. (2000). Resonant column testing of sands with different viscosity pore fluids. A.S.C.E. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 126, No. 1, 10- 17.

Fearon, R.E. and Coop, M.R. (2000). Reconstitution: what makes an appropriate reference material? Géotechnique, Vol. 50, No. 4, 471-477.

Feda, J. (2004). Physical models of soil behaviour. Engineering Geology, Vol. 72, 121-129.

Feder, H.J., and Feder, J. (1991). Self-organized critically in a stick-slip process. Physical Review Letters, Vol. 66, No. 20, 2669-2672.

Fernandez, A.L. and Santamarina, J.C. (2001). Effect of cementation on the small-strain parameters of sands. Canadian Geotechnical Journal, Vol. 38, 191-199.

Fourie, A.B., Blight, G.E., and Papageorgiou, G. (2001). Static liquefaction as a possible explanation for the Merriespruit tailings dam failure. Canadian Geotechnical Journal, Vol. 38, 707-719.

Fourie, A.B. and Papageorgiou, G. (2001). Defining an appropriate steady state line for Merriespruit gold tailings. Canadian Geotechnical Journal, Vol. 38, 695-706.

Fitzpatrick, E. A. (1971). Pedology: A systematic approach to soil science. Oliver&Boyd.

FitzPatrick, E. A. (1984). Micromorphology of Soils. Printed in Great Britain at the University Press, Cambridge.

Gajo, A. (2004). The influence of system compliance on collapse of triaxial sand samples. Canadian Geotechnical Journal, Vol. 41, 257- 273.

GDS Instruments Ltd, (2001). GDSLAB v2, GDS Laboratory Users Handbook.

Gens, A. and Nova, R. (1993). Conceptual bases for a constitutive model for model for bonded soils and weak rocks. Geotechnical Engineering of Hard Soils-Soft Rocks, A. Anagnostopoulos et al., (eds), Athens, Greece, Balkema: Rotterdam, Vol. 1, 485-494.

Gilboy, G. (1928). The compressibility of sand-mica mixtures. Proceedings of the A.S.C.E., Vol. 2, 555-568.

Gourdon, D. and Israelachvili, J.N. (2003). Transitions between smooth and complex stick-slip sliding of surfaces. Physical Review E, Vol. 68, No. 021602.

Gutenberg, B. and Richter, C.F. (1956). Earthquake magnitude, intensity, energy and acceleration, Bull. Seism. Soc. of America, Vol. 32, No.3, 104-145.

Harvey, J.C. (1982). Geology for Geotechnical Engineers. Cambridge University Press

Heymann, G., Clayton, C.R.I., Reed, G.T. (1997). Laser interferometry to evaluate the performance of local displacement transducers. Géotechnique, Vol. 47, No. 3, 399-405.

Hight, D.W., Boese, R., Butcher, A.P., Clayton, C.R.I. and Smith, P.R. (1992). Disturbance of Bothkennar clay prior to laboratory testing. Géotechnique, Vol. 42, No. 2, 199-218.

Hillgärtner, H., Dupraz, C., and Wolfgang, H. (2001). Microbially induced cementation of carbonate sands: are micritic meniscus cements good indicators of vadose diagenesis? Sedimentology, Vol. 48, 117-131.

Huang, J.T. and Airey, D.W. (1998). Properties of artificially cemented carbonate sand. J. Geotech. Geoenvir. Engng Div., A.S.C.E., Vol. 124, No. 6, 492-499.

Ismail, M.A., Joer, H.A. and Randolph, M.F. (2000). Sample preparation technique for artificially cemented sands. Geotechnical Testing Journal, A.S.T.M., Vol. 23, No. 1, 141-157.

Ismail, M.A., Joer, H.A., Randolph, M.F., and Meritt, A. (2002). Cementation of porous materials using calcite. Géotechnique, Vol. 52, No. 5, 313-324.

Ismail, M.A., Joer, H.A., Sim, W.E., and Randolph, M.F. (2002). Effect of cement type on shear behaviour of cemented calcareous soil. Journal of Geotechnical and Geoenvironmental Engineering, June 2002, 520-529.

Jardine, R.J., Symes, M.J., and Burland, J.B. (1984). The measurement of soil stiffness in the triaxial apparatus. Géotechnique, Vol. 34, No. 3, 323-340.

Jardine, R.J. (1994). One perspective of the pre-failure deformation characteristics of some geomaterials. Proceedings of the First International Conference on Cre-Failure Deformation of Geomaterials. Shibuya, Mitachi and Miura (eds), Sapporo, Japan, 12-14 September, 855-885.

Jensen, R.P., Bosscher, P.J., Plesha, M.E., and Edil, T.B. (1999). DEM Simulation of granular media-structure interface: Effects of surface roughness and particles shape. Int. J. Numer. Anal. Meth. Geomech., Vol. 23, 531-547.

Kavvadas, M., Anagnostopoulos, A., and Kalteziotis, N. (1993). A framework for the mechanical behaviour of the cemented Corinth marl. Proceedings of International Geotechnical Engineering of Hard Soils-Soft Rocks, A. Anagnostopoulos et al., (eds), Athens, Greece, Balkema: Rotterdam, 577-583.

Kim, M.S. (1995). *Expérimentale du Comportement Mécanique des matériaux Granulaires sous Forte Contrainte.* Ph.D. Thesis, Ecole Centrale Paris, 1995; 158–167.

Krinsley, D.H. and Smalley, I.J. (1973). Shape and nature of small sedimentary quartz particles. Science, Vol. 180, 1277-1279, 22 June 1973.

Kucharski, E., Price, G., Li, H., and Joer, H.A. (1996). Engineering properties of CIPS cemented calcareous sand. In Engineering Geology: Proceedings of the 30th International Geological Congress, Beijing, China, 4-14 August 1996. W. Sijing and P. Marinos (eds). Brill Academic Publisher, Amsterdam, The Netherlands. Vol. 23, 92-97.

Lafaber, D. (1966). Soil structural concepts. Engineering Geology. Vol. 1, No. 4, 261-290.

Lambe, T.W. (1951). Soil Testing for Engineers. John Wiley and Sons, New York.

Lambe, T.W. (1967). Stress path method. Journal of the Soil Mechanics and Foundations Division, A.S.C.E., Vol. 93, No. SM6, Proc. Paper 5613, November, 309-330.

Lambe, T.W. and Whitman, R.V. (1969). Soil mechanics. John Wiley & Sons, Inc.

Lambe, T.W. and Marr, W.A. (1979). Stress path method: Second edition. Journal of the Geotechnical Engineering Division, A.S.C.E., Vol. 105, GT6, Proc. Paper 14655, June, 727-738.

Leroueil, S and Hight, D.W. (2003). Behaviour of properties of natural soils and soft rocks. Characterisation and Engineering Properties of Natural Soils, Tan et al., (eds), Swets and Zeitlinger, Lisse, 29-254.

Leroueil, S. and Vaughan, P.R. (1990). The general and congruent effects of structure in natural soils and weak rocks. Géotechnique, Vol. 40, No. 3, 467-488.

Liu, M.D. and Carter, J.P. (1999). Virgin compression of structured soils. Géotechnique, Vol. 49, No. 1, 43-57.

Liu, M.D. and Carter, J.P. (2000). Modelling the destructuring of soils during virgin compression. Géotechnique, Vol. 50, No. 4, 479-483.

Luding, S. (2003). The micro-macro mechanics of granular materials. GACM report 2, 22-28.

Luo, D., Macleod, J.E.S., Leng, X., and Smart, P. (1992). Automatic orientation analysis of particles of soil microstructres. Géotechnique, Vol. 42, No. 1, 97-107.

Maccarini, M. (1987). Laboratory studies of weakly bonded artificial soil. PhD thesis, University of London.

Malandraki, V. (1994). The engineering behaviour of a weakly bonded artificial soil, PhD thesis, University of Durham, UK.

Malandraki, V. and Toll, D.G. (2001). Triaxial tests on weakly bonded soil with changes in stress path. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 3, March, 2001, 282-291.

McLean, A.C. and Gribble, C.D. (1982). Geology for Civil Engineers. GEORGE ALLEN & UNWIN, Third impression.

Mohamedelhassan, E., Shang, J.Q., Ismail, M.A., and Randolph, M.F. (2005). Electrochemical cementation of calcareous sand for offshore foundations. International Journal of Offshore and Polar Engineering, Vol. 15, No. 1, 71-79.

Micic, S., Shang, J.Q., and Lo. K.Y. (2003). Improvement of the load-carrying capacity of offshore skirted foundations by electrokinetics. Canadian Geotechnical Journal, Vol. 40, No. 5, 949-963.

Miller, B., O'Hern, C., and Behringer, R.P. (1996). Stress fluctuations for continuously sheared granular materials. Physical Review Letters, Vol. 77, No. 15, 3110-3113.

Mirghasemi, A.A., Rothenburg, L., and Matyas, E.L. (2002). Influence of particle shape on engineering properties of assemblies of two-dimensional polygon-shaped particles. Géotechnique, Vol. 52, No. 3, 209-217.

Mitchell, J.K. (1976). Fundamentals of Soil Behaviour. John Wiley & Sons, Inc. Printed in the United States of America.

Mitchell, J.K. (1993). Fundamentals of Soil Behaviour. John Wiley & Sons, Inc. Printed in the United States of America, Second edition.

Mitchell, J.K. and Santamarina, J.C. (2005). Biological Considerations in Geotechnical Engineering, Journal of Geotechnical and Geoenvironmental Engineering, A.S.C.E., Vol. 131, No. 10, 1222-1233.

Nasuno, S., Kudrolli, A., Bak, A., and Gollub, J.P. (1998). Time-resolved studies of stick-slip friction in sheared granular layers. Physical Review E, Vol. 58, No. 2. 2161- 2171.

Oda, M. (1972). Initial fabrics and their relations to mechanical properties of granular material. Soils and Foundations, Vol. 12, No. 1, March, 17-37.

Olson, R.E. and Mesri, G. (1970). Mechanisms controlling the compressibility of clay. Journal of the Soil Mechanics and Foundations Division, A.S.C.E., Vol. 96, No. SM6, Proc. Paper 7649, November, 1970, 1863-1878.

Palomino, A.M. and Santamarina, J.C. (2005). Fabric map for kaolinite: effects of pH and ionic concentration on behaviour. Clays and Clay Minerals, Vol. 53, No. 3, 211-223.

Perkins, S.W., Gyr, P., and James, G. (2000). The influence of biofilm on the mechanical behaviour of sand. Geotechnical Testing Journal, GTJODJ, Vol. 23, No. 3, September, 300-312.

Powrie, W. (1997). Soil mechanics: concepts and applications. E & FN SPON.

Priest, J.A. (2004). The dynamic response of soils containing gas hydrates. PhD thesis, University of Southampton, UK.

Ramachandran, S.K., Ramakrishnan, V., and Bang, S.S. (2001). Remediation of concrete using microorganisms. American Concrete Institute Materials Journal, Vol. 98, 3-9.

Ratnaweera, P. and Meegoda, J.N. (2006). Shear strength and stress-strain behaviour of contaminated soils. Geotechnical Testing Journal, Vol. 29, No. 2, March.

Redlich, K.A., Terzaghi, K. and Kampe, R. (1929). *Ingenieurgeologie* (Engineering Geology), Springer, Vienna.

Rippa, F. and Picarelli, L. (1977). Some considerations on index properties of Southern Italian shales. Proceedings of the International Symposium on the Geotechnics of Structurally Complex Formations, Capri, Vol. 1, 401-406.

Roberts, J.E. (1964). Sand compression as a factor in oil field subsidence, thesis submitted in partial fulfilment of Sc.D., Department of Civil Engineering, M.I.T. (Feb.).

Roscoe, K.H., Schofield, A.N., and Wroth, C.P. (1958). On the yielding of soils. Géotechnique, Vol. 8, No. 1, 22-53.

Rothenburg, L. and Bathurst, R.J. (1989). Analytical study of induced anisotropy in idealized granular materials. Géotechnique, Vol. 39, No. 4, 601-614.

Rotta, G.V., Consoli, N.C., Prietto, P.D.M., Coop, M.R., and Graham, J. (2003). Isotropic yielding in an artificially cemented soil cured under stress. Géotechnique, Vol. 53, No. 5, 493-501.

Rowe, P.W. (1972). The relevance of soil fabric to site investigation practice. Géotechnique, Vol. 22, No. 2, 195-300.

Santamarina, J.C. (2001). Soil behaviour at the microscale: particle forces. Proc. Symp. Soil Behaviour and Soft Ground Construction, in honour of Charles C. Ladd, October, MIT, USA.

Santamarina, J.C. and Cho, G.C. (2004). Soil behaviour: The role of particle shape. The Skempton Conference, Thomas Telford, London, 604-617.

Santamarina, C. and Cascante, G. (1998). Effect of surface roughness on wave propagation parameters. Géotechnique, Vol. 48, No. 1, 129-136.

Saxena, S.K. and Lastrico, R.M. (1978). Static properties of lightly cemented sand. Journal of the Geotechnical Engineering Division. GT12, December 1978, 1449-1464.

Scholey, G.K., Frost, J.D., Lo Presti, D.C.F., and Jamiolkowski, M. (1995). A Review of instrumentation for measuring small strain during triaxial testing of soil specimens. Geotechnical Testing Journal, GTJODJ, Vol. 18, No. 2, June, 137-156

Schnaid, F., Prietto, P.D. and Consoli, N.C. (2001). Characterization of cemented sand in triaxial compression. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 10, 857-868.

Shang, J.Q., Mohamedelhassan, E., and Ismail, M. (2004). Electrochemical cementation of offshore calcareous soil. Canadian Geotechnical Journal, Vol. 41, No. 5, 877-893.

Sides, G.R. (1971). Soil microstructure and sample disturbance observations in the stereoscan electron microscope. Proceedings of the Roscoe Memorial Symposium, Cambridge University, 89-98.

Skempton, A.W. (1944). Notes on the compressibility of clays. Quarterly Journal Geological Soc., Vol. 100, 119-135.

Skempton, A.W. and Northey, R.D. (1952). The sensitivity of clays. Géotechnique 3, 30-53.

Skempton, A.W. (1970). The consolidation of clays by gravitational compaction. Q. J. Geol. Soc. 125, 373-411.

Smith, P.R. (1992). The behaviour of natural high compressibility clay with special references to construction on soft ground. PhD thesis, University of London.

Sornette, A. and Sornette, D. (1989). Self-organized critically and earthquakes. Europhys. Lett. Vol. 9, No. 3, 197-202.

Stool, R.D. (1989). Sediment acoustics. Lecture notes in earth sciences, Vol. 26, Springer, New York.

Subramanian, S., and Sloan, E.D. Jr. (2000). Microscopic measurements and modelling of hydrate formation kinetics. Ann NY Acad Sci., 912: 583-592.

Sukumaran, B. and Ashmawy, A. K. (2001) Quantitative characterisation of the geometry of discrete particles, Géotechnique, Vol. 51, No.7, 619-627.

Taylor, D.W. (1942). Research on the consolidation of clays, Massachusetts Institute of Technology, Department of Civil and Sanitary Engineering, Serial 82, Cambridge.

Terzaghi, K. (1925). *Erdbaumechanik auf bodenphysikalischer grundlage. Deuticke, Leipzig/Vienna.*

Terzaghi, K. (1941). Undisturbed clay samples and undisturbed clays. Journal of the Boston Soc. Civil Engineers, Vol. 28, No. 3, 211-231.

Terzaghi, K. and Peck, R.B. (1962). Soil Mechanics in Engineering Practice. John Wiley & Sons, Inc. 12th Edition, printed in the United State of America..

Terzaghi, K., Peck, R.B., and Mesri, G. (1996). Soil Mechanics in Engineering Practice. John Wiley & Sons, Inc. Third Edition, Printed in the United States of America.

Theron, M. (2004). The Effect of Particle Shape on the Behaviour of Gold Tailings. PhD thesis, University of Southampton, U.K.

Thompson, P.A. and Grest, G.S. (1991). Granular flow: Friction and Dilatancy Transition. Physical Review Letters, Vol. 67, No. 13, 1751- 1754.

Toll, D.G. and Malandraki, V. (1993). Triaxial testing of a weakly bonded soil. Geotechnical Engineering of Hard Soils-Soft Rocks, A. Anagnostopoulos et al., (eds), Athens, Greece, Balkema: Rotterdam, 817-825.

Toll, D.G. and Malandraki, V. (2000). The engineering behaviour of structured soils, Proc., 1st Central Asian Geotech Symp., Vol. 2, 629-634.

Vallejo, L.E. (1995). Fractal analysis of granular materials. Géotechnique, Vol. 45, No. 1, 159-163.

Van Olphen, H. (1963). An introduction to clay colloid chemistry. New York: John Wiley and Sons.

Vanel, L., Howell, D., Clark, D., Behringer, R.P., and Clement, E. (1999). Memories in sand: Experimental tests of construction history on stress distributions under sandpiles. Physical Review E, Vol. 60, No. 5, rapid communication R5040.

Vermeulen, N.J. (2001). The composition and state of gold tailings. PhD Thesis. University of Pretoria, South Africa.

Wilson, J.M.R. (1988). A theoretical and experimental investigation into the dynamic behaviour of soil. PhD thesis, University of Cambridge, U.K.

Wright, P.G. (1977). The variation of viscosity with temperature. Physics Education, July, 323-325.

Yang, L.A., Tan, T.-S., Tan, S.-A, and Leung, C.-F. (2002). One-dimensional self-weight consolidation of a lumpy clay fill. Géotechnique, Vol. 52, No. 10, 713-725.

Yang, Y.I.C., Li, Y., Park, J.K. and Yen, T.F. (1994). Subsurface application of slime-forming bacteria in soil matrices. Applied Biotechnology for Site Remediation by Robert E. Hinchee et al. CRC Press Inc. 268-274.

Zeng, X., Wu, J., and Young, B.A. (1998). Influences of viscous fluids on properties of sand. Geotechnical Testing Journal, GTJODJ, Vol. 21, No. 1, 45-51.

APPENDIX

Table 1 List of the deviatoric stress-pore water pressure-strain points where the fluctuations happened at the Leighton Buzzard Sand with castor sugar tested at room temperature with the ration of 5g sucrose /100g water.

The last value just before the slip				The first value just after the slip			
Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)	Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)
404	231	2.50	19730	351	234	2.57	19740

Table 2 List of the deviatoric stress-pore water pressure-strain points where the fluctuations happened at the Leighton Buzzard Sand with castor sugar tested at room temperature with the ration of 25g sucrose /100g water.

The last value just before the slip				The first value just after the slip			
Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)	Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)
65	312	0.03	550	42	317	0.02	560
148	331	0.38	4030	115	334	0.38	4040
187	322	0.77	7320	173	323	0.77	7330
208	315	0.98	9030	189	315	0.98	9040
251	299	1.34	11860	168	308	1.33	11870
300	282	1.83	15850	196	293	1.97	15860

Table 3 List of the deviatoric stress-pore water pressure-strain points where the fluctuations happened at the Leighton Buzzard Sand with castor sugar tested at room temperature with the ration of 100g sucrose /100g water.

The last value just before the slip				**The first value just after the slip**			
Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)	Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)
84	319	0.05	790	55	323	0.04	800
128	333	0.28	2750	94	336	0.28	2760
165	325	0.74	5880	132	327	0.73	5890
174	324	0.84	6680	105	330	0.83	6690
212	311	1.29	9690	196	311	1.28	9700
224	307	1.39	10430	140	314	1.42	10440
213	311	1.53	11190	159	314	1.58	11200
229	307	1.71	12250	162	311	1.81	12260
257	296	2.05	14050	140	309	2.04	14060

Table 4 List of the deviatoric stress-pore water pressure-strain points where the fluctuations happened at the Leighton Buzzard Sand with castor sugar tested at room temperature with the ration of 150g sucrose /100g water.

The last value just before the slip				The first value just after the slip			
Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)	Deviatoric Stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)
35	306	0.02	280	17	311	0.01	290
96	328	0.10	1310	53	333	0.09	1320
132	337	0.57	4300	61	344	0.55	4310
153	334	1.06	7250	74	341	1.19	7260
169	330	1.55	9550	94	335	1.59	9560
170	331	1.73	10620	110	335	1.72	10630
203	319	2.09	13130	89	331	2.22	13140
242	305	2.71	16600	175	307	2.72	16610
252	301	2.85	17540	119	314	3.04	17550
262	300	3.30	19500	110	316	3.42	19510

Table 5 List of the deviatoric stress-pore water pressure-strain points where the fluctuations happened at the Leighton Buzzard Sand with castor sugar tested at room temperature with the ration of 287g sucrose /100g water (method 1c).

The last value just before the slip				**The first value just after the slip**			
Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)	Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)
42	304	0.02	250	19	308	0.02	260
71	313	0.05	580	54	315	0.04	590
99	321	0.09	980	76	323	0.12	990
99	325	0.14	1130	50	328	0.13	1140
122	336	0.32	2330	44	340	0.44	2340
129	343	0.69	3990	47	351	0.68	4000
136	345	0.97	5830	54	349	0.95	5840
144	344	1.22	7640	60	349	1.22	7650
149	344	1.46	9270	65	349	1.47	9280
153	344	1.67	10670	67	347	1.74	10680
158	343	1.93	11990	63	347	1.90	12000
162	342	2.09	13400	77	345	2.09	13410
181	335	2.35	15180	84	340	2.43	15190
176	337	2.57	16170	72	342	2.60	16180

Table 6 List of the deviatoric stress- pore water pressure- strain points where the fluctuations happened at the Leighton Buzzard Sand with Dow Corning Silicon Oil.

The last value just before the slip				**The first value just after the slip**			
Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)	Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)
91	325	0.06	1030	63	328	0.06	1040
97	334	0.11	1460	93	334	0.11	1470
98	335	0.11	1530	60	337	0.11	1540
91	341	0.13	1790	71	342	0.17	1810
93	344	0.19	2040	63	346	0.19	2050
87	348	0.20	2220	65	349	0.21	2230
94	351	0.26	2640	70	351	0.27	2650
92	353	0.29	2890	62	353	0.34	2900
95	355	0.39	3380	60	356	0.39	3390
96	358	0.47	4040	61	359	0.51	4050
94	360	0.57	4550	56	361	0.56	4560
93	361	0.63	5160	58	361	0.63	5170
93	362	0.69	5780	60	363	0.72	5790
93	363	0.82	6530	59	363	0.82	6540
91	366	0.87	7130	65	366	0.92	7140
91	367	0.96	7640	56	367	1.04	7650
82	370	1.05	7870	62	370	1.05	7880
81	371	1.07	8100	58	371	1.07	8110
79	372	1.09	8330	55	372	1.12	8340
84	372	1.20	8960	56	372	1.19	8970
84	372	1.25	9460	58	373	1.29	9470
80	374	1.32	9770	69	373	1.32	9780
84	373	1.35	10070	60	373	1.35	10080
79	374	1.38	10320	53	374	1.42	10330
87	372	1.52	11140	70	372	1.52	11150
86	372	1.55	11440	60	372	1.56	11450
87	372	1.61	11950	54	373	1.65	11960
88	372	1.73	12700	67	372	1.73	12710
88	372	1.77	13040	55	373	1.80	13050

Table 7 List of the deviatoric stress- pore water pressure- strain points where the fluctuations happened at the Leighton Buzzard Sand with icing sugar tested at 60°C.

The last value just before the slip				The first value just after the slip			
Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)	Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)
98	325	0.09	1040	70	327	0.09	1050
128	337	0.48	3670	68	342	0.56	3680
125	342	0.73	4910	64	348	0.72	4920
127	344	1.02	6970	67	349	1.09	6980
134	344	1.33	8630	69	349	1.41	8640
142	340	1.68	10470	64	349	1.76	10480
141	342	2.01	12180	62	351	2.05	12190
153	340	2.41	14680	66	350	2.54	14690
152	341	2.80	16530	80	347	2.84	16540

Table 7 Leighton Buzzard specimens tested at room temperature with sucrose by following method 4d.

The last value just before the slip				The first value just after the slip			
Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (hrs)	Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (hrs)
44	304	0.02	330	35	306	0.05	340
94	314	0.14	1120	58	317	0.20	1130
117	323	0.46	2780	65	326	0.49	2790
121	327	0.67	3870	66	330	0.72	3880
135	326	1.11	6020	64	330	1.26	6030
134	327	1.49	7450	68	331	1.55	7460
141	327	1.79	9070	65	332	1.88	9080
140	329	2.08	10400	73	333	2.18	10410
152	325	2.51	12520	68	331	2.66	12530

Table 8 Leighton Buzzard specimens tested at room temperature with sucrose by following method 1d.

The last value just before the slip				The first value just after the slip			
Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)	Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)
63	307	0.02	470	33	313	0.02	480
125	321	0.14	1870	79	323	0.18	1880
134	324	0.26	2700	78	327	0.32	2710
153	320	0.47	4310	62	326	0.45	4320
151	326	0.66	5420	77	330	0.78	5430
161	324	0.81	6760	76	329	0.84	6770
173	320	1.04	8380	91	325	1.09	8390
184	316	1.27	9870	79	325	1.26	9880
195	312	1.46	11750	83	321	1.53	11760
207	309	1.76	13620	105	315	1.84	13630
209	309	1.98	14880	98	316	2.09	14890

Table 9 Leighton Buzzard specimens tested at room temperature with sucrose by following method 1a.

The last value just before the slip				The first value just after the slip			
Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)	Deviatoric stress (kPa)	Pore pressure (kPa)	Average local strain (%)	Time (sec)
142	321	0.24	2510	109	322	0.23	2520
162	322	0.47	4100	106	325	0.47	4110
171	321	0.64	5490	92	327	0.71	5500
174	323	0.89	7030	101	326	0.93	7040
203	315	1.27	9820	107	322	1.33	9830
189	321	1.42	10660	110	326	1.48	10670