PARTIALLY DRAINED RESPONSE OF SANDS

by

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Abstract

This study was carried out to improve our understanding of the behaviour of sands under different drainage conditions, especially the partially drained condition. The influence of aging on the stress-strain response of sands was also considered. The study was performed on two different sands, a Fraser River sand and a North Sea sand. As such, objectives of this study are (a) the effects of drainage conditions, (b) the partially drained response and (c) the influence of aging on the stress-strain response of sands.

Identical soil specimens were aged under constant stresses for 1, 100 and 1,000 minutes and then sheared under undrained or partially drained conditions. Effects of the aging period on the stress-strain response were studied at k_0 conditions and at different values of confining stress. A constant period of aging of 100 minutes was selected for the main laboratory program of tests.

The shear response of aged (100 mins) sands, under drained and undrained conditions, was examined. Although aging positively influences the shear response of sands, in that shear strength increases, the general behaviour regarding the effect of confining stress and stress ratio is found to be the same.

The effect of small imposed volumetric strains on the shear response of aged (100 mins) sands was studied. Imposed expansive volumetric strains negatively influence the shear response whereas the contractive strains influence positively. Certain imposed expansive volumetric strains results in continuous strain softening. Even though sands continuously strain soften for certain imposed expansive volumetric strains, the effect of confining stress and stress ratio are seen at small strain levels. The material with higher stress ratio and lower confining stress start to strain soften at smaller strain levels.

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CHAPTER 1

INTRODUCTION

Solving problems of stability and deformation in soils requires a confident characterization of their effective stress-strain behavior. Observations show this behavior to be nonlinear and inelastic, and furthermore, to depend on the stress path, strain path and stress history. Upon load application, a saturated soil element in a given earth structure experiences an effective stress path. This path depends on the induced spatial distribution of total stresses and pore water pressures, and their variation with time. These are, in turn, governed by stress regime, displacement boundary conditions and drainage conditions of the entire earth structure.

Traditionally, the response of a saturated soil element to an applied stress or strain increment is assumed to be bounded between two extremes, namely, the fully drained and undrained response. The term "undrained" characterizes a zero volume change ($d\varepsilon_v = 0$), and represents loading along a prescribed strain path with zero volumetric strain. The term "drained" represents the loading without creating any excess pore water pressure, i.e., loading along a prescribed stress path with $du = 0$. This implies that there is no time lag between the application of total and effective stresses. However, in most field cases, the soil elements can drain and therefore experience volume and pore water pressure changes with time ($d\varepsilon_v \neq 0$, $du \neq 0$).

It has been tacitly assumed that saturated granular materials behave in an undrained manner during a loading event of short duration, for example earthquake loading or wave loading. Yet, experimental shake table studies and field studies on liquefaction (Seed 1987; Liu and Qiao 1984; Whitman 1985) have demonstrated that saturated soil elements in earth structures do experience drainage both during and soon after earthquake loading. This drainage can be attributed to the spatial variation of pore water pressure generated in the soil structure, and their dissipation due to the high permeability of granular soils. In fact, in most geotechnical problems, initial loading can be regarded as undrained, with drainage commencing simultaneously. Therefore, most of the geotechnical problems related to saturated granular soil elements would experience a certain degree of drainage. Drainage conditions other than the extremes of zero drainage (undrained) and complete drainage (drained) may be referred to as "partially drained" conditions. At certain states, the partial drainage scenario may constitute a more damaging response than either the conventional drained or the undrained loading cases.

Vaid and Eliadorani (1998) demonstrated experimentally that very small expansive volumetric strains can transform the strain hardening response of a relatively dense sand that occurs with undrained loading into a strain softening response. They established that large changes in the strain increment direction, caused by small volumetric strain increments, cause such a transformation in the response to loading. The findings imply that ignoring the potential for simultaneous commencement of drainage may yield an unrealistic prediction of mobilized soil strength and therefore deformation.

In sand, under the stress states commonly encountered, pore pressure dissipates rapidly and the deformation after primary consolidation is almost negligible. Therefore, the effect of secondary compression in sand has not been studied until recently. The effect of aging on the stress-strain response of sand has now been recognized by researchers, particularly in small strain regions. A very limited number of laboratory studies has been carried out to investigate time effects in sand. It has been assumed that there are no significant effects of aging on the stress-strain response of granular material and therefore no specific controls have been exercised on the age of laboratory specimens. Typically, an arbitrary aging time occurs which varies between a few minutes and many hours.

The objective of this thesis is to investigate the partially drained response ($d\varepsilon_v \neq 0$, $du \neq 0$) of two different sands, a Fraser River Sand and a North Sea Sand. Triaxial tests were performed on reconstituted sand specimens. The influence of aging on the stress-strain response is also investigated, in order to establish a suitable constant aging period for the laboratory specimens. The effect of consolidation history is also taken into consideration by consolidating the specimens along the k_0 consolidation path. In other words, the partially drained response of both Fraser River sand and North Sea Sand is investigated taking into account the effect of aging as well as consolidation history.

CHAPTER 2

LITERATURE REVIEW

This Chapter provides a brief review of the partially drained response, and the effect of aging on the stress-strain response of sands, which are relevant to the experimental research program carried out. First, a brief review of definitions is given with schematic illustrations. The drained and undrained responses are then briefly discussed, and the "Partially drained response" subsequently reviewed. Thereafter, the effect of aging on the stress strain response of sand is addressed. The proposed research program is described at the end the chapter.

Most of our fundamental understanding of the stress-strain response of soils is derived from laboratory tests. The response of sands under triaxial compression has been the mainstay of our current knowledge in this field. The stress-strain response of saturated sands has mostly been studied under two different conditions (drained and undrained). The drained response is defined as the loading along a prescribed stress path with no excess pore water pressure ($du = 0$). It implies that the total stress increment and effective stress increment are equal and there is no time lag between them (yielding $d\sigma = d\sigma'$). The response of soils to shear loading with zero volume change ($d\varepsilon_v=0$) is defined as the undrained stress-strain response. Under undrained conditions, there will be excess pore water pressure development ($du \neq 0$). This excess pore water pressure is either positive or negative, depending on the stress-strain state and nature of the soil itself.

2.1 UNDRAINED RESPONSE OF SANDS

The undrained response of saturated sand in triaxial compression is shown in Figure 2.1. Three different types of characteristic response are depicted. The response of an element to undrained loading depends mainly on its initial state variables, relative density, effective confining stresses, and static shear stress level.

The characteristic feature of the type (1) response is continued deformation. This is called the steady state deformation (SS) or flow deformation (Castro et al., 1982; Vaid and Chern, 1985). The effective stress ratio corresponding to the maximum deviator stress, which marks the initiation of strain softening, has been called the Critical Stress Ratio (CSR) by Vaid and Chern (1983), collapse surface by Sladen et al. (1985), and instability line by Lade and his co-workers (Lade et al., 1993). Vaid and Chern (1985) and Dorby et al. (1985) suggested that the CSR is unique for a given sand in the compression mode. Once the CSR is reached, the soil exhibits a sudden loss in its strength associated with a rapid increase in pore pressure. Eventually, the soil resistance reaches a minimum strength at large strains, called steady state by Castro (1969). The type (1) phenomenon associated with loss in strength has been termed liquefaction (Castro, 1969; Casagrande, 1975; Seed, 1979; Vaid and Chern, 1985).

The type (2) response, a strain softening type of response, occurs over a limited strain range and then transforms into a strain hardening behavior with further deformation. This type of response has been termed limited liquefaction (Castro, 1969). The soil deforms essentially at constant stresses and void ratio, in much the same manner as the steady state of deformation. This state has been called a quasi steady state (QSS) by Ishihara et al. (1975). The term phase transformation (PT) corresponds to the instant at which the behavior of the sand transforms from contractive into dilative. Several studies have indicated that both of these states (PT and QSS) essentially occur at the same mobilized effective stress ratio, for a given sand, regardless of its initial density and stress state (Dorby et al., 1985; Vaid and Chern, 1985; Vaid et al.1990a.;

Ishihara, 1993; Vaid and Thomas, 1995). Further, the effective stress ratio mobilized at the phase transformation or quasi steady states has been found to be equal to that at the steady state under undrained loading (Chern, 1985).

The strain hardening type of response, type (3), is associated with a dense sand or loose sand at low effective confining stress. In this type of response sand develops negative pore water pressures following the initial positive value. The soil resistance in this type of response keeps increasing with increasing deformation. In type (2) and (3) responses, following the PT state, the effective stress paths proceed towards and then along the maximum obliquity (MO) line which represents a constant effective stress ratio specific to the sand.

Vaid and Chern (1985) have shown that in undrained triaxial compression, saturated sands respond in a strain softening manner over a range of stress states. The strain softening, if any, is triggered at the mobilized Critical Stress Ratio (CSR). The CSR line thus represents the starting point of a strain softening behavior. The softening behaviour continues until the soil reaches the effective stress ratio corresponding to the PT state. The soil upon passing the stress ratio corresponding to PT state, strain hardens with further deformation. Thus, the region of stress space between the CSR line and PT line has been termed the region of strain softening deformation (Figure 2.2). However, the region does not extend down to the origin of stress space. Instead, it terminates at an approximately horizontal cutoff "ab": the position of "ab" depends on the relative density of the sand and it moves upward with increasing relative density (Vaid and Chern, 1985).

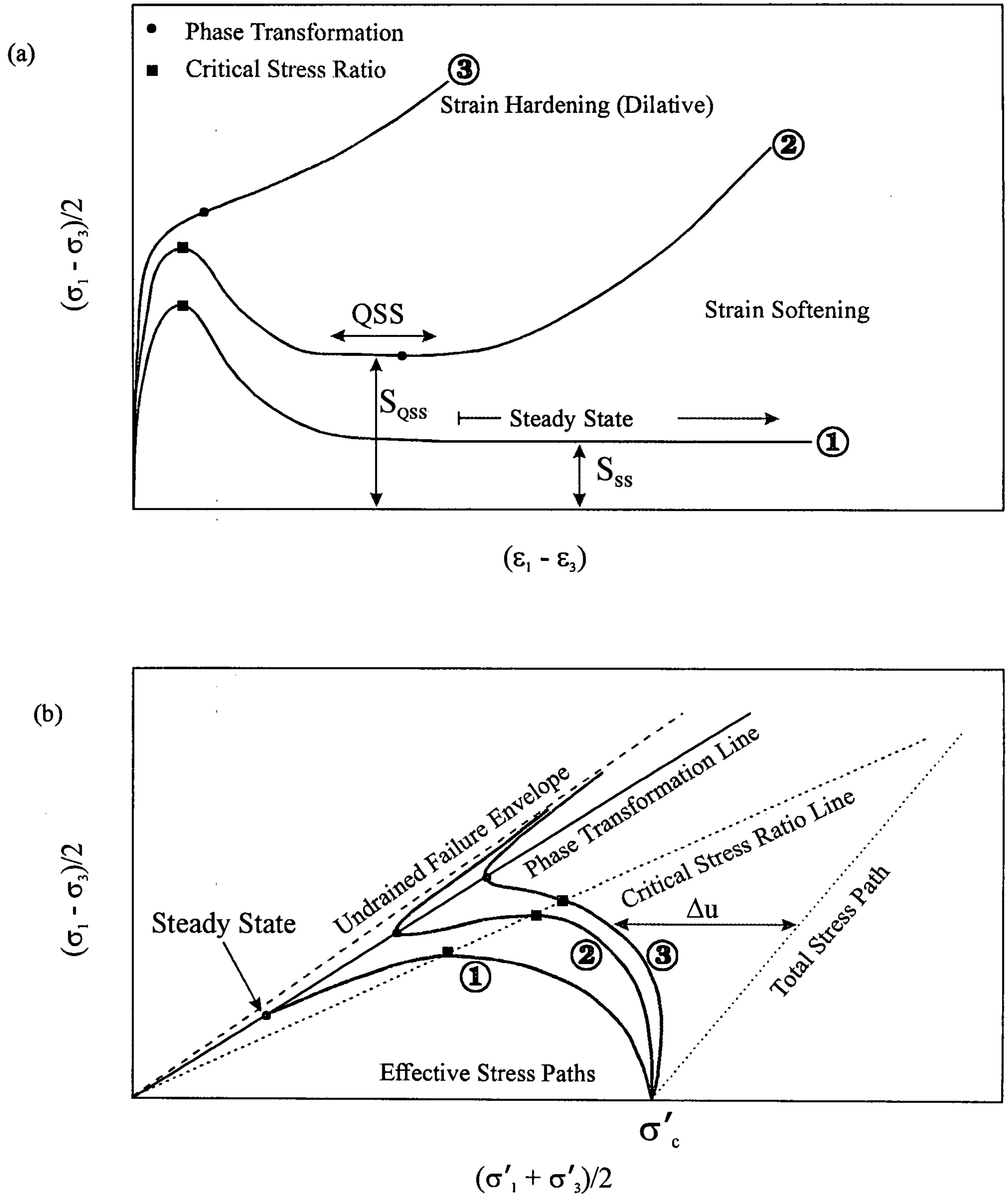


Fig.2.1 Typical Undrained Response of Sand

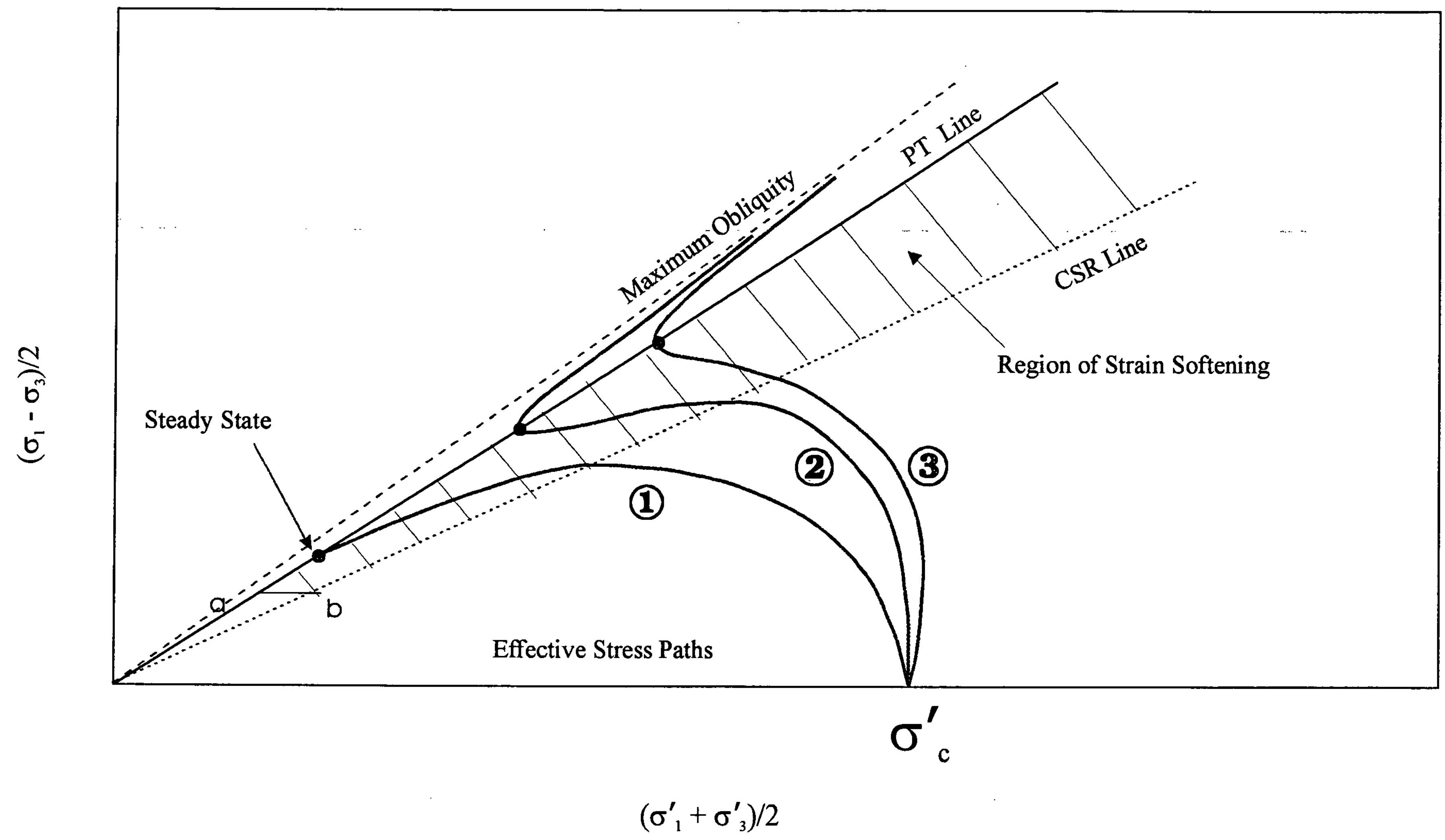


Fig. 2.2 Region of Strain Softening

2.2 PARTIALLY DRAINED RESPONSE

Traditionally, the response of a saturated soil element to an applied stress or strain increment is assumed to be bounded between two extremes, namely, the fully drained and the undrained. In all geotechnical problems related to short duration loading, it is typical to assume no drainage, and therefore no change in volume or void ratio, prior to the initiation of instability. However, in most cases, upon load application to an earth structure, there is a spatial distribution of excess pore water pressure. The spatial distribution of excess pore water pressure generated within an earth structure and the relatively high permeability of sands will lead to drainage, even during short duration events like an earth quake or the case of wave loading.

As noted earlier, experimental shake table studies and field observations on liquefaction by Seed (1987), Liu and Qiao (1984) and Whitman (1985) have demonstrated that soil elements in earth structures do experience drainage both during and soon after earthquake loading. Volume changes may also occur soon after the cessation of a short duration loading. If a flow slide develops following an earthquake (such as at the Lower San Fernando Dam), then it could not be assumed to be a completely undrained problem.

The drainage condition that is not completely undrained and fully drained is referred to as "partially drained". This partial drainage condition may, in certain cases, constitute a more damaging loading condition compared to the undrained or drained conditions. A typical example of such a situation occurs while loading a saturated dense sand with expansive volumetric stains. Vaid and Eliadorani (1998) demonstrated experimentally that very small expansive volumetric strains can transform the undrained strain hardening response of denser sand into a strain softening type. The expansive volumetric strains used were in fact very small. Thus, the transformation from strain hardening to strain softening behavior under partial expansive drainage conditions cannot be attributed towards physical loosening of the sand. They also demonstrated that very small imposed contractive volumetric strains would transform a strain softening response of a sand into a strain hardening response. Further, they found that the magnitude of the imposed volumetric strains, although small, dramatically influences the response; hence comparatively larger expansive volumetric strains result in more strain softening response (leading to flow failure).

Figure 2.3 shows schematically the effect of partially drained conditions on the stress strain response of sands. It illustrates the effect of expansive and contractive volumetric strains with reference to the undrained response. The negative value of $d\epsilon_v/d\epsilon_1$ represents an expansive volumetric strains while the positive value represents a compressive volumetric strain. As it can be seen in figure, imposing an expansive volumetric strain of $d\epsilon_v/d\epsilon_1 = -0.1$ results in a strain softening type of response, compared to strain hardening for the undrained response. In this case, the volumetric strain imposed on the specimen after 1% axial strain is only 0.1% (1%×0.1=0.1%). In contrast, imposing a contractive volumetric strain transforms the partially strain softening undrained response into a continuously strain hardening response.

For loose Fraser River sand, a positive strain increment causes a strain hardening response, and a negative strain increment gives rise to a strain softening response. The partially drained response of Fraser River sand ranges from strained hardening to partially strain softening and finally to full strain softening, depending on the imposed $d\epsilon_v/d\epsilon_1$. The behaviour is attributed to large changes in the strain increment direction caused by small strain increments (Vaid and Eliadorani, 1998).

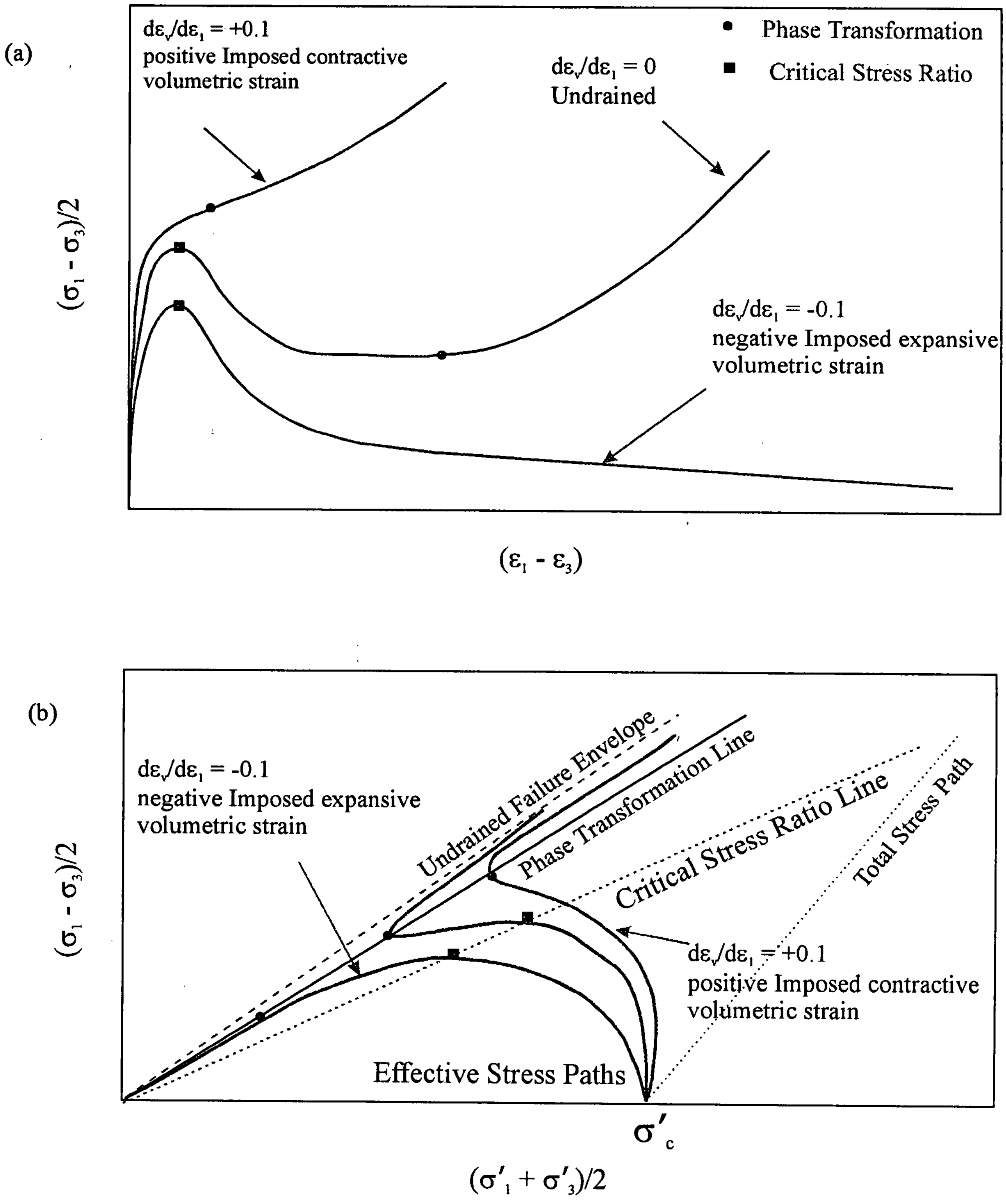


Fig.2.3 Typical Partially Drained Response of Sand

2.3 AGING

In clays, secondary compression and time effects have been recognized for many years as an important contribution to total compression. Since secondary consolidation in clay results in significant deformation and changes in soil properties, several studies have been carried out to investigate aging effects on clay.

In sand under the stress state commonly encountered, pore pressure dissipates rapidly and the deformation generated after primary consolidation is almost negligible. Therefore, the effect of secondary compression in sand has not been widely studied until recently. Now, aging effects in sand are better recognized from laboratory and field observations. Still, the aging behaviour and its subtle effects are not well understood in granular materials.

An increase in penetration resistance with age after deposition or disturbance has been observed in the field tests. Mitchell and Solymar (1984) observed a time-dependent increase in penetration resistance after compaction of a fill. They suggested that the most probable mechanism for the observed phenomenon seemed to be inter-particle bonding due to the formation of silica acid gel films on the particle surface. Charlie (1992) noted an increase in cone penetration resistance with time after ground improvement by blasting. Jamiolkowski (1998) observed that the stiffness increase is much more pronounced in fine grained soils.

Schmertmann (1991) has also presented evidence of increasing penetration resistance with time after dynamic compaction. He suggested that a dispersive particle reorientation takes place in all soils with time, resulting in an increase in particle friction and therefore a gain in modulus and strength with time. Whether clay or sand, Schmertman argued that the frictional gain is the reason for increase in modulus and strength with time. Mesri (1990) suggested that the increased frictional resistance upon aging of clean sand develops through macrointerlocking of particles and microinterlocking of surface roughness.

Whitman (1964) carried out odometer tests on a dry quartz sand to study the effect of aging. Samples were aged by holding the loads constant for a rest period during the compression test. The re-compression modulus of aged specimens was found to be much larger than the modulus of unaged specimens. Anderson and Stokoe (1978) have also studied the effect of aging on the shear modulus of air-dry Ottawa sand under hydrostatic stress conditions using the resonant column device. The shear modulus for a waiting period of 1000 minutes (G_{1000}) was used as the reference value of the stiffness. A linear increase in modulus with the logarithm of time was observed for about 10,000 minutes (see Figure 2.4). The rate of increase in shear modulus was found to be about 1 % of G_{1000} per log cycle of aging time. Interestingly, they found that the magnitude of the long-term effect seems to be relatively independent of D_{50} until a value of D_{50} less than about 0.05mm. However, it is not known whether this is the case for sands other than Ottawa sand.

Shozen (2001) have studied the effect of aging on the drained stress strain response of water pluviated Fraser River sand specimens. He found that the secant shear modulus corresponding to the 0.03 % ($G_{S\,0.03}$) and 0.15 % ($G_{S\,0.15}$) shear strain level increases with aging period, in tests up to 1000 minutes. Further, he found that the secant shear modulus is approximately linear with logarithm of aging time (see Figure 2.5). He also performed drained tests on specimens with different stress ratios, R= $d\sigma'_1/d\sigma'_3$, (1.0, 2.0 and 2.8) and established that the normalized secant shear modulus at both 0.03 % and 0.15 % exhibits a linear relationship with logarithm of time (see Figure 2.6). Additionally, it was found that the effect of aging is much more pronounced in $G_{S\,0.03}$ than $G_{S\,0.15}$, which suggests the influence of aging on the drained response of sands is more significant in the small strain range.

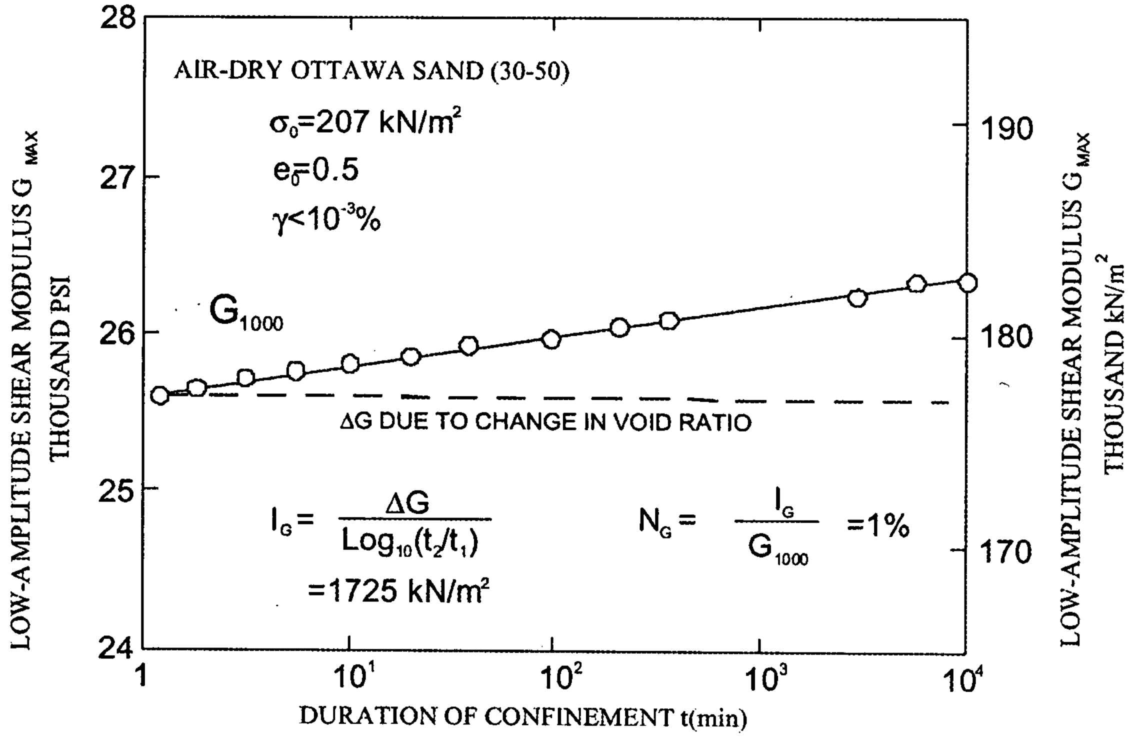


Figure 2.4 Effect of Aging Time on Shear Modulus from Resonant Column Test (Anderson and Stokoe, 1978)

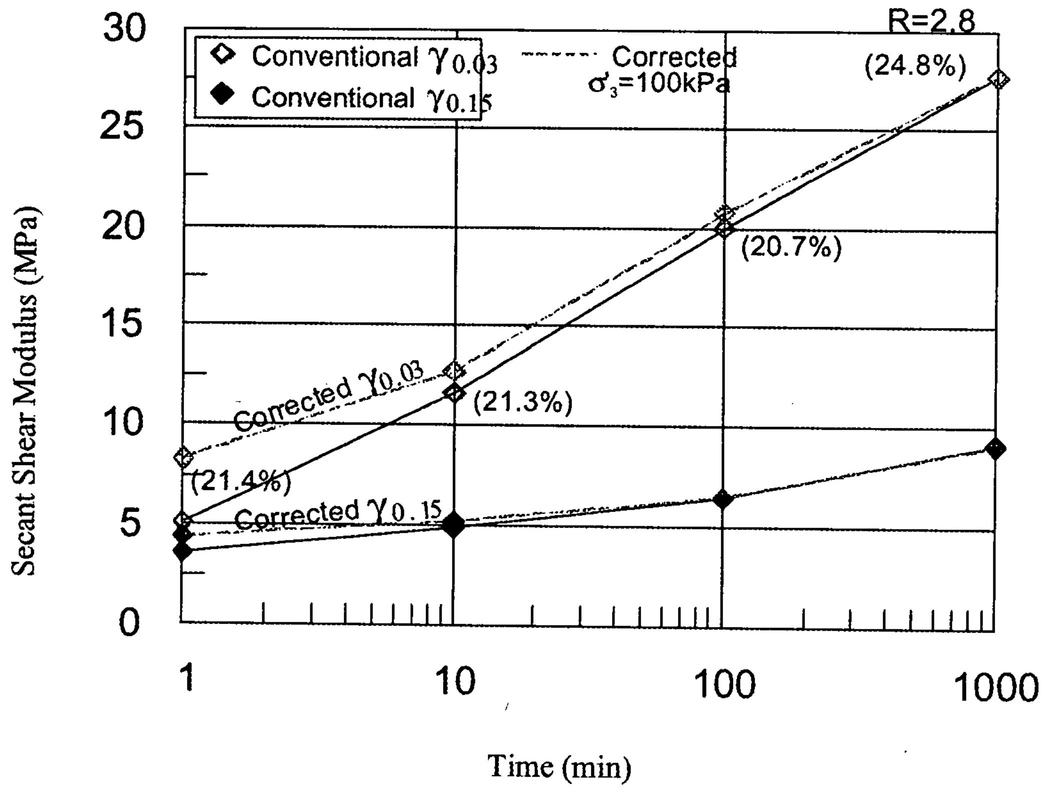


Figure 2.5 Effect of Aging period on Secant Shear Modulus(Shozen, 2001)

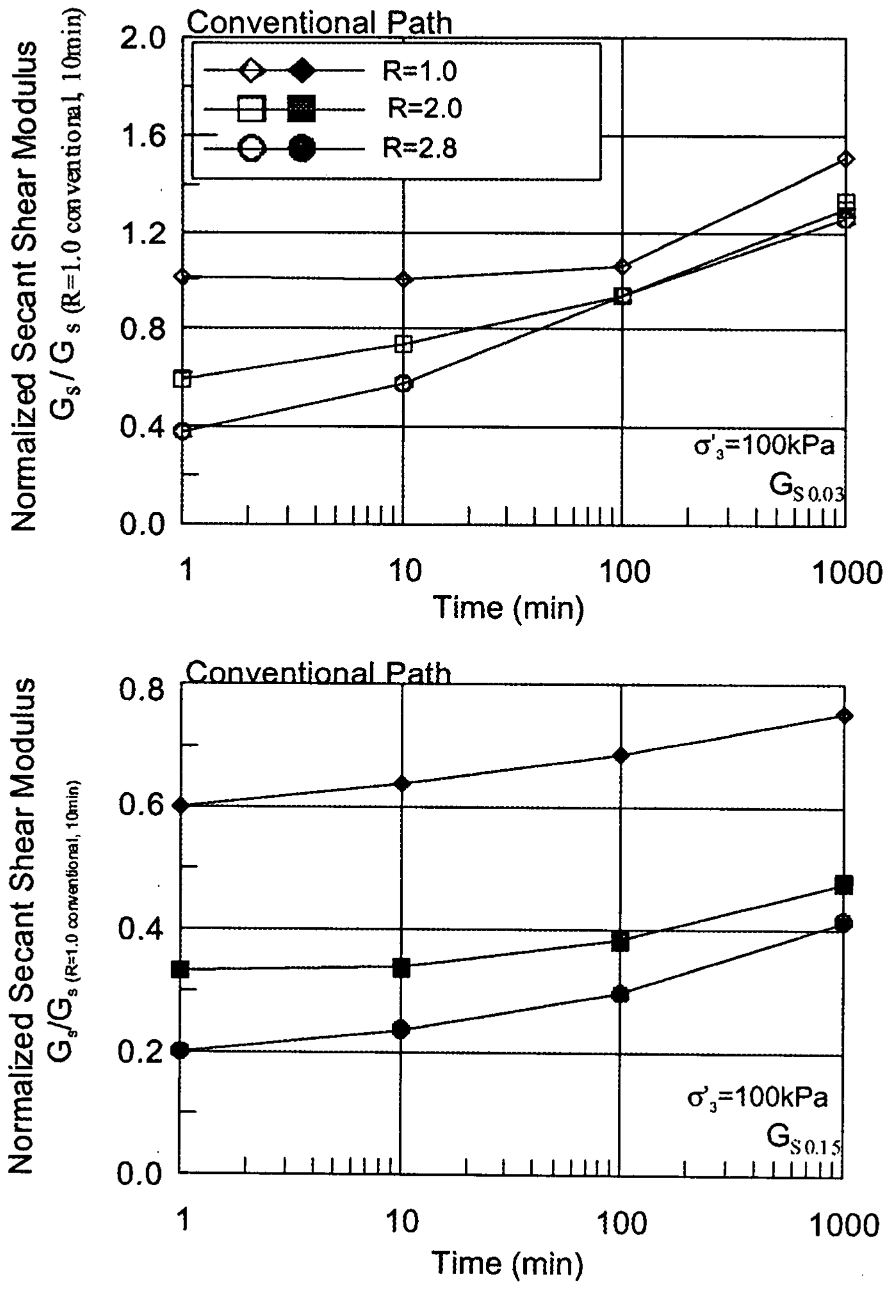


Figure 2.6 Effect of Aging period on Secant Shear Modulus (Shozen, 2001)

2.4 PROPOSED RESEARCH PROGRAM

Even though the effect of aging is well recognized, no systematic study has been carried out to investigate the effect of aging period on the undrained and partially drained response of sands. Rather, it has been assumed that there is no significant effect of aging on the stress-strain response of granular materials and therefore no specific controls have been exercised on the age of laboratory specimens. Generally, an arbitrary aging time is allowed that may vary between a few minutes and many hours. This thesis attempts to establish a reasonable aging period for laboratory specimens used in testing. The study is carried out on specimens at their loosest densities (as deposited). The effect of aging is studied for a waiting period of up to 1000 minutes.

Thereafter, for a selected and constant aging period, this program of study investigates the undrained and partially drained behaviour of two different sands, a Fraser River sand and a North Sea sand. The undrained behaviour of aged specimens is investigated at different effective confining stress levels and at different stress ratios. The partially drained response, the focus of the program, is investigated with different degrees of controlled drainage at different initial stress states. A crucial drainage condition is chosen for detailed investigation. Further investigation is carried out at the critical drainage condition and at different initial states.

Experimental procedures and material properties of the soils used are described in chapter3. The test results are reported in chapter 4. A summary of the findings is given in chapter5. Chapter 6 outlines the conclusions and addresses further research needs.

CHAPTER 3

EXPERIMENTAL STUDY

This chapter describes the experimental aspects of the study. A description is given of the advanced triaxial testing apparatus, which incorporates a computer controlled stress / strain path loading system, together with the associated instrumentation, test control and data acquisition system. A discussion of test errors and the steps taken to minimize them is also included. The procedure for specimen reconstitution, consolidation and shear loading is outlined. Discussion addresses the assessment of specimen void ratio, stability of stresses during the aging period and resolution of stress and strain measurements. Typical experimental results are presented in order to demonstrate test consistency and repeatability. An outline of the experimental program is provided at the end of the chapter.

3.1 TESTING EQUIPMENT

The conventional triaxial testing apparatus has been widely used in studies of soil behaviour. There is, in general, a broad familiarity and acceptance of behaviour characterized by the test. Consequently, it was decided to use an advanced triaxial testing system to investigate the partially-drained response of sands.

3.1.1 The Triaxial Apparatus

The arrangement of the advanced triaxial testing system is shown schematically in Figure 3.1. Axial load, confining pressure, pore pressure, axial displacement and volume change are measured using electronic transducers. These electronic transducers are coupled to a data acquisition system which is interfaced with a computer. A computer-controlled digital volume pressure controller (DVPC) is used to control the volume of the specimen. Electro-pneumatic

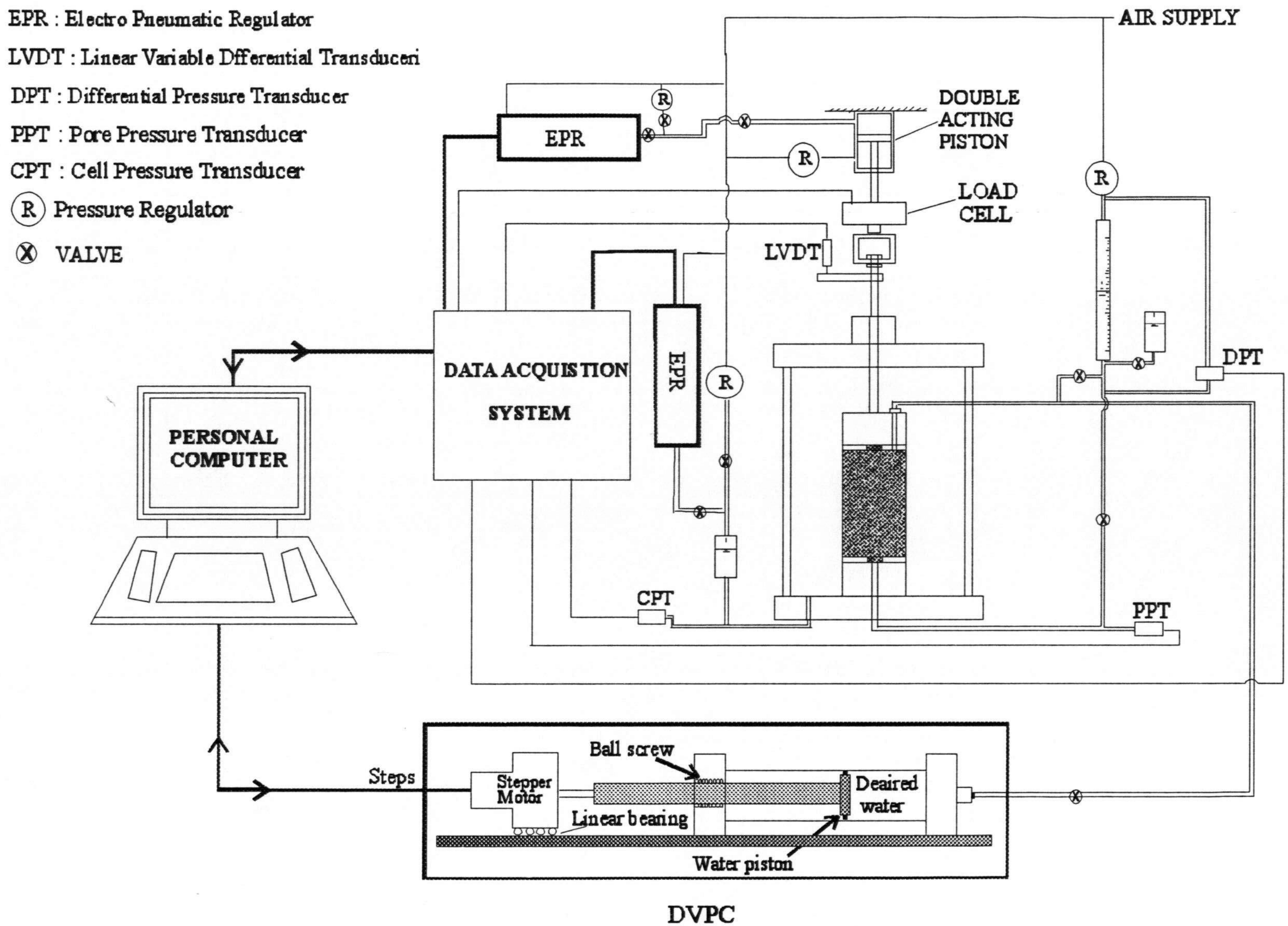


Fig. 3.1 The schematic Diagram of the Advanced Triaxial Testing System.

regulators, which are also controlled by the computer, are used to automate and follow prescribed consolidation paths.

The triaxial cell has a continuously air-leaking frictionless seal, and an external load cell for measurement of axial load. Pore water pressure is measured at the bottom of the specimen and the DVPC is connected to the top of the specimen.

Triaxial specimens were 64mm diameter and 130 mm long. Smooth, hard anodized aluminium end platens with centrally located 20 mm diameter porous stone discs were used in order to minimize the end restraint.

3.1.2 The Loading System

The axial loading system is capable of applying compression or extension loads to the triaxial specimen under stress or strain-controlled conditions. A double-acting frictionless air piston (see Figure 3.1) that is coupled in series with a constant speed motor drive enables the system to provide a smooth transition from stress-controlled to strain-controlled loading and vice versa. In addition, the air piston facilitates anisotropic consolidation of the specimen and compensates for vertical uplift on the loading ram during hydrostatic loading.

Computer-controlled electro-pneumatic regulators connect both to the double-acting air piston, which controls the axial load, and to the cell pressure, thereby allowing the imposition of a prescribed stress path. Two manual regulators (R) are connected in parallel with the electro-pneumatic regulators (EPR) to facilitate maintaining more stable stresses during the aging period.

3.1.3 Data Acquisition and Control System

The triaxial data acquisition system uses a high speed data acquisition and control system. A "*National Instrument AT-MIO-16x*" 16 - bit high speed data acquisition card is used for signal input and output. This data acquisition card is configured to use six A/D channels and two D/A channels. Five of the A/D channels are used to acquire data from the transducers used in the system (two for pressure transducers, one for axial displacement, one for axial load and one for specimen volume change). Both D/A channels are used to control the electro-pneumatic regulators. In addition to the "*National Instrument AT-MIO-16x*" data acquisition card, a 16 - bit D/A "Vextra Indexer" card is used to control the DVPC.

The electro-pneumatic regulators used are of " *SMC* " type with 786 kPa full scale output with 882 kPa input. Cell pressure and pore pressure are measured by high resolution " *Druck* " pressure transducers that have a full scale measurement of 700 kPa. A "*StrainSert*" type flat load cell with 454 kg capacity is used to measure the axial load. A high output high resolution LVDT of type " Hewlett Packard " is used to measure the axial displacement.

3.1.4 Measurement Resolutions

Carefully selected transducers and a sophisticated data acquisition system yield a high resolution of measurement and precise control of the test specimen. The "*StrainSert*" flat load cell has a resolution of ± 10 grams, which for the specimen diameter of 64 mm represents a resolution of ± 0.04 kPa. Axial displacement is measured using a LVDT with an axial strain resolution of 0.001%. The resolution of cell and pore pressure transducers is ± 0.1 kPa. Volume change of the specimen is measured with a differential pressure transducer. The resolution volumetric strain measurement is 0.001% for a 64 mm diameter and 130 mm long triaxial specimen.

In summary, the high resolution and high speed data acquisition system and transducers enabled confident measurement of stresses and strains. The radial and axial stresses are measured with a resolution of about 0.25 kPa. The axial and volumetric strains are measured with an accuracy of 0.001 %.

3.1.5 The Digital Volume Pressure Controller

Strain path controlled tests were performed using a Digital Volume Pressure Controller (DVPC). The principle of DVPC use in soils testing was described by Menzies (1987), and is illustrated in the schematic diagram (Figure 3.1). A digital signal from the computer controls the stepper motor which, in turn, imposes a corresponding volume change to the specimen either by injection or extraction of water via the top drainage line to the specimen. Thus, volume change of the saturated specimen is continuously controlled by the computer to achieve a targeted volumetric strain during loading. Each stepper motor pulse is equivalent to a volumetric strain of $3*10^{-6}$ for the size of the triaxial specimen used.

3.2 MATERIAL TESTED

Tests were performed on two different sands, a Fraser River sand from British Columbia, Canada, and a North Sea sand from the coast of Norway. The Fraser River sand was dredged from the river delta at Vancouver. The North Sea sand was taken from the sea bed. The two sands differ in mineralogy and gradation.

Fraser River sand is a uniform grey coloured medium - grained sand with sub-angular to sub-rounded particles. The typical mineral composition of the Fraser River sand is 40% quartz, quartzite and chert, 11% feldspar, 45% unstable volcanic rock fragments and 4% miscellaneous detritus (Garrison et al., 1969). The sand has an average particle size (D_{50}) of 0.35 mm (see Figure 3.2) and a coefficient of uniformity (C_U) of 1.7. The specific gravity is 2.695 according to ASTM D854. The maximum and the minimum void ratios were found to be 0.900 and 0.594 respectively according to ASTM D4253 and ASTM D4254.

The North Sea sand is a uniform medium - grained sand with sub-rounded particles. Inspections reveals the particles to be less angular than the Fraser River sand. The typical mineral composition is 90 % quartz, 5 % albite, 2 % hornblende and 3 % miscellaneous detritus. The sand is finer than the Fraser River sand, having a D_{50} of 0.183 mm and C_U of 2.0 (see Figure 3.2). The microscopic view of both Fraser River sand and North Sea sand are given in Figure 3.3. The specific gravity was found to be 2.66 according to ASTM standards. The maximum and the minimum void ratios were found to be 0.730 and 0.487 according to ASTM D4253 and D4254 respectively.

3.3 SAMPLE PREPARATION TECHNIQUE

Testing undisturbed samples represents the ideal scenario for characterizing the in-situ behavior of sand. However, as indicated by Seed et al. (1982), conventional undisturbed sampling techniques invariably alter the mechanical properties of a sand. In-situ ground freezing

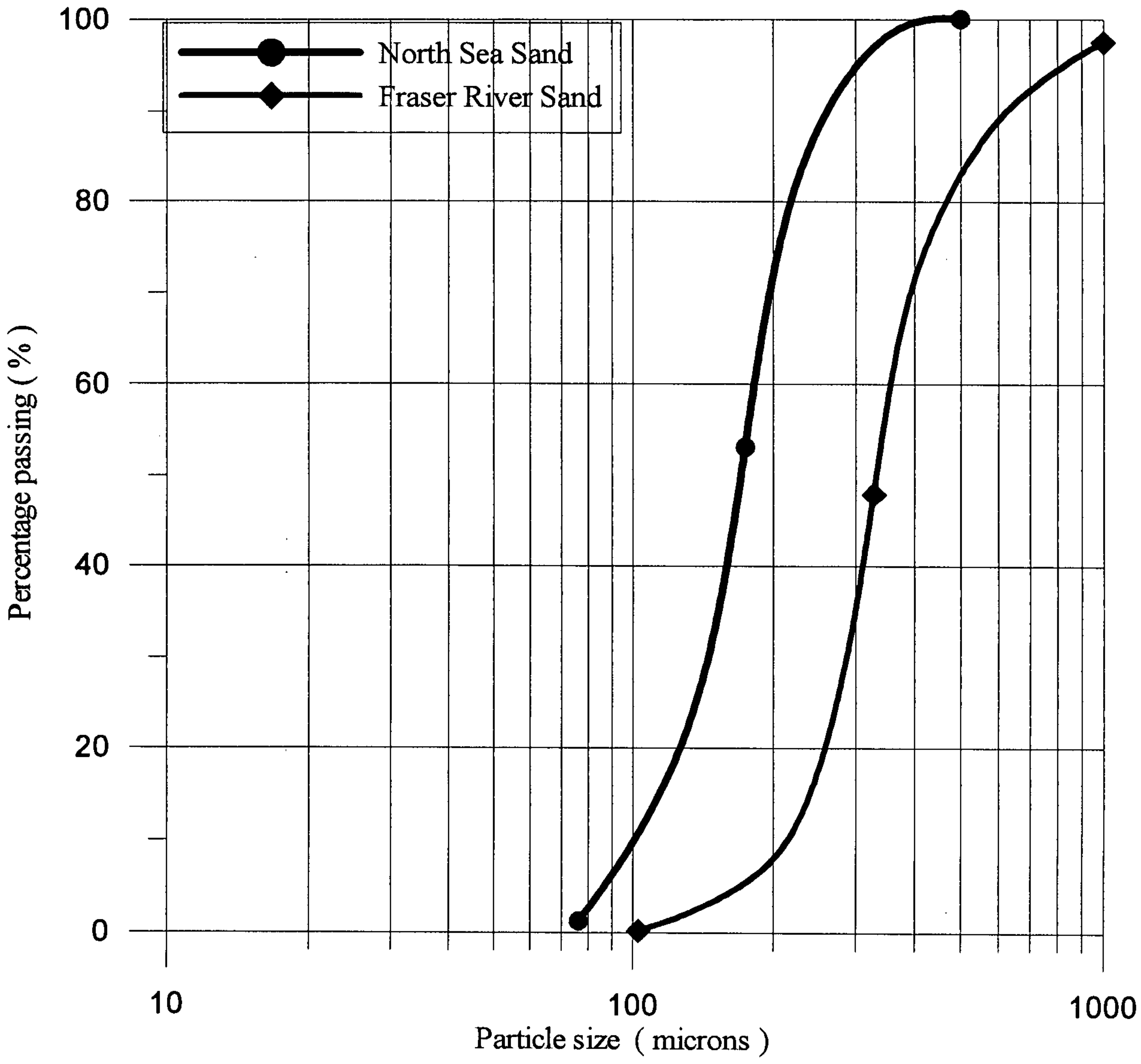
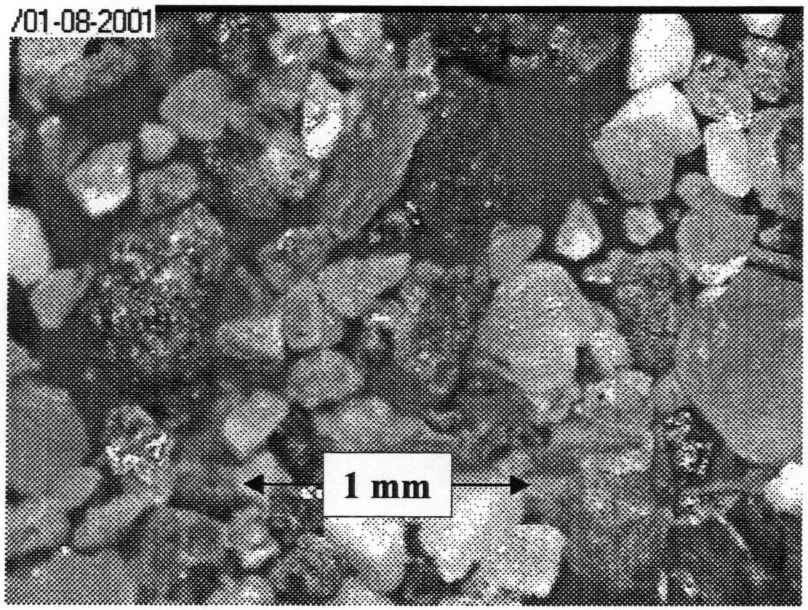
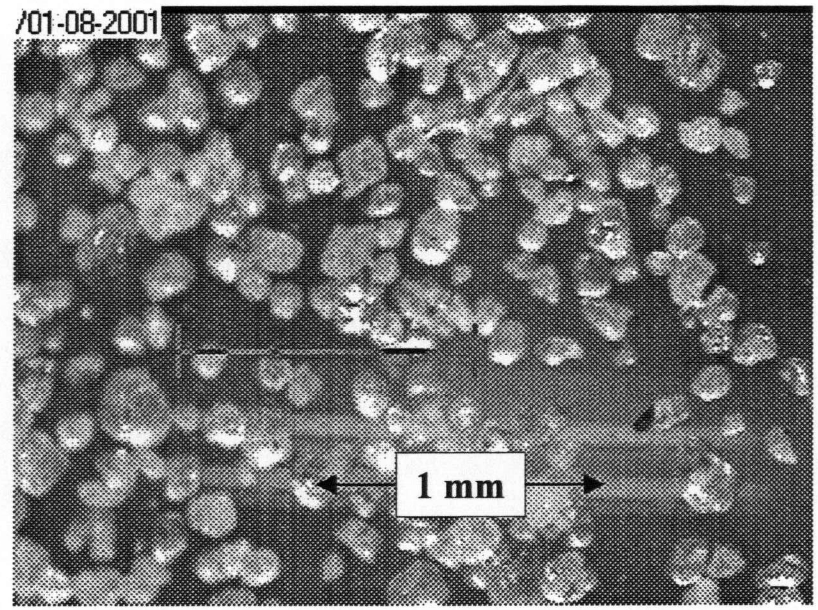


Figure 3.2 Grain size distribution of Fraser River sand and North Sea sand



Fraser River sand



North Sea sand

Fig.3.3 Microscopic view of the Fraser River sand and North Sea sand is regarded as the most desirable technique for retrieving an undisturbed sample (Yoshimi et al. 1984, 1989). However, it is very expensive. Therefore, fundamental studies on water deposited fluvial and hydraulic fill sands are frequently carried out using reconstituted specimens. Water pluviated sand specimens have been shown to possess a fabric similar to fluvial and hydraulic fill sands (Oda, 1972). This technique of specimen reconstitution provides a conventional method of studying the in-situ behavior of these materials. Specimens prepared with this technique have proved to be very uniform (Vaid and Negussey, 1984). A close agreement between the behavior of sand retrieved by in-situ ground freezing, and its water pluviated equivalent at the same initial state, has been reported by Vaid et al. (1996). Further, Sivathalayan (2000) has shown very good agreement between the stress-strain behavior of undisturbed samples retrieved from water-deposited sands and reconstituted specimens prepared using the water pluviation technique. The technique is also noted to ensure the saturation of sand, and was used for all tests in this study.

3.4 RECONSTITUTION OF TEST SPECIMENS

Oven dried sand was mixed in a flask and then boiled for about 30 minutes. Thereafter it was left to cool to room temperature, and then kept under vacuum until the time of specimen preparation. In advance of pluviation, and to establish the specimen height after deposition, a reference measurement was taken by mounting a dial gauge on a removable stand and placing an aluminum dummy specimen of known height instead of the specimen.

The sand is pluviated into a water filled cavity, formed by a latex membrane sealed to the base pedestal of the triaxial apparatus with an O-ring and held against the interior of a split mold (by a small vacuum). The flask containing the boiled sand was inverted into the mold, and sand then deposited by continuously moving the tip of the flask around the cavity in a circular path. When the deposition exceeds the targeted height of the specimen, pluviation was halted. Then the excess sand was siphoned to form a level surface (Sayao, 1989). Finally, the top cap was carefully placed on the level sand surface and the membrane pulled over it and sealed by another O-ring. Following the connection of top drainage line, a vacuum of about 20 kPa was applied to the specimen in order to provide a small confinement. The split mold was then dismantled. Volume change due to this suction was recorded by having a graduated side reservoir. At this point, the triaxial cell was assembled and filled with de - aired water. The top and bottom drainage lines were then closed to maintain the confinement and the vacuum line was disconnected. The final dial gage reading and the side reservoir level were recorded to determine the sample height and the volume. Specimens for the study were prepared without any intentional densification and with minimum disturbance so that to achieve very loose specimens.

Void ratio is computed from the known geometry of the cavity. This minimizes any potential error due to measurement of specimen circumference in the computations of void ratio (Vaid and Sivathayalan, 1996). The area of the former cavity was determined by the filling it to a known height with a known volume of water. The volume of water expelled due to the small vacuum confinement, together with the specimen height recorded, then permits an accurate computation of specimen volume and hence its void ratio at the end of preparation.

3.5 TEST ERRORS

The degree to which the conventional triaxial test satisfies the idealized assumptions regarding homogeneity of stress and strain within the test specimen is a major concern. The following steps were taken in order to minimize the deviation of the actual test conditions from the idealized assumptions, and thereby promote confidence in the measured variables.

3.5.1. Internal vs External Measurements

The stress and strain state of the sand were measured by electronic transducers mounted outside the specimen. Limitations of such external measurements have been pointed out by Atkinson and Evans (1985) and Jardine et al. (1985). However, Sayao (1989) showed that there will not be any significant difference between axial strain derived from external and internal measurements if appropriate measures are taken to eliminate the bedding, seating, and / or tilting errors.

The triaxial cell used in testing has a special continuously air-leaking frictionless seal around the loading ram. Continuous air-bleeding bushings can virtually eliminate ram friction (Chan, 1975), and permit an external measurement of axial force.

Friction between the ends of the specimen and the rigid end caps may influence the response, as a result of stress nonuniformity. However, the influence of end restraint on strength has been shown to be minimal when a height to diameter ratio of 2 or larger is used (Taylor, 1948; Bishop and Green, 1965; Lade, 1982). The specimen size in this series of tests was about 130 mm in height and 64 mm in diameter, which ensured an insignificant end-restraint effect. Moreover, any end restraint was kept to a minimum by using highly polished end platens with centrally located porous disc of only 20 mm diameter.

3.5.2 Membrane Penetration

Most laboratory triaxial tests are performed on specimens enclosed in a flexible membrane. A change in effective confining stress during the test yields a penetration of the membrane into, or conversely withdrawal from, interstices of the sand specimen. This causes a systematic error in measured volume change in a drained test, or measured excess pore pressure in an undrained test. For example, an increase in pore pressure would result in the membrane moving outwards from the interstices and, as a consequence, the excess pore pressure would be smaller than that in the truly undrained test.

The effective stress state is therefore directly influenced by membrane penetration in an undrained test. This may have a profound influence on the measured strength of a sand, depending on the magnitude of the membrane penetration. In a drained test, with changing effective confining stress, measured volumetric strain consists of both that from the soil skeleton volume change and that from volume change due to membrane penetration. Thus the measured volume change must be corrected for membrane penetration effects to obtain the actual volumetric strain of the soil skeleton.

Several researchers have identified factors that influence the magnitude of membrane penetration (Kiekbusch and Schupper, 1977, Ramana and Raju, 1982, Seed et al., 1987, Nicholson et al., 1993a). Grain size and gradation have been found to be the most dominant factors. The influence of relative density is less for a denser packing of sand particles (Ramana and Raju, 1982, Seed et al. 1989, Nicholson et al. 1993a). Grain angularity and fabric exert a negligible influence on the magnitude of membrane penetration (Banarjee et al., 1979, Nicholson et al. 1993a).

The magnitude of membrane penetration is often expressed in terms of volume change per unit surface of the membrane in contact with the specimen per 10-fold change in effective confining stress, termed the unit membrane penetration ϵ_m. The volume change induced by the penetration of membrane (ΔV_m) depends on the change in effective confining stress, surface area of the membrane (A_m) and the unit membrane penetration (ϵ_m), where:

$$\Delta V_m = \epsilon_m \bullet A_m \bullet \log(\sigma'_{current} / \sigma'_{initial}) \qquad \text{equation 3.1}$$

and σ' is the effective confining stress.

Different methods have been proposed to evaluate the magnitude of membrane penetration, and that of Sivathayalan and Vaid (1998) was used for this program of tests.

The DVPC was used for making membrane penetration corrections during the performance of tests. It allows predetermined volumes of water to be injected into, or extracted from the test specimen. The computer program designed for controlling the injection process continuously monitors changes in the effective confining stress and corrects for the corresponding volumetric error. The program computes the volumetric error and injects or extracts controlled volumes of water to exactly offset it.

3.6 TEST PROCEDURE

3.6.1 Specimen set-up

The assembled triaxial cell was centred and securely clamped under the loading frame. The loading piston was then brought down. The load cell, cell pressure transducer, pore pressure transducer, LVDT, and differential pressure transducer were set to their initial zero output. The top and bottom drainage lines were then connected to the DVPC and pore pressure transducer respectively (see Figure 3.1). The cell pressure was then increased in steps of 20 kPa, to a value of approximately 120 kPa, maintaining the specimen in an undrained condition. The initial pore pressure of –20 kPa will increase with cell pressure increment. The final pore pressure will be close to 100 kPa at the end. The Skempton's B-value was computed for each increment of cell pressure. Full saturation of the specimen was ensured by insisting on a B-value of greater than 99%.

3.6.2 Consolidation

The back pressure was set to be equal to the pore pressure developed at the end of the final Skempton's B value check and was fed in to both the drainage pipette and the DVPC (see Figure 3.1). The specimen was then consolidated to the targeted effective confining stress (σ'_{3c}) and stress ratio ($R_c = \sigma'_{1c} / \sigma'_{3c}$). The consolidation path is shown schematically in figures 3.4 a and b. The specimen was first axially loaded towards the targeted stress ratio R_c and then consolidated along the constant stress ratio line towards the desired value of σ'_{3c}. Electro-pneumatic regulators, connected to the double - acting piston and to the cell pressure reservoir, provide a precise control on the stress path of the specimen during the consolidation phase. These regulators are controlled by software through the data acquisition system.

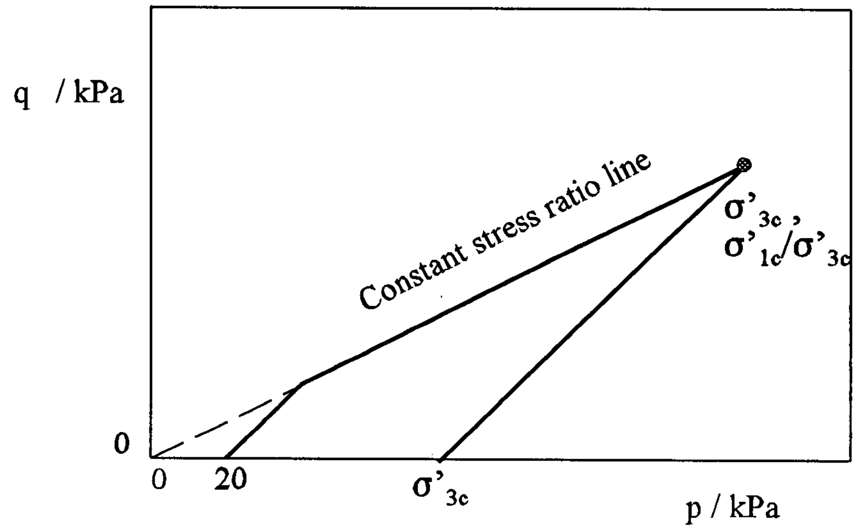
3.6.3 Aging after consolidation

Following completion of consolidation, the specimen was held at the final value of constant stress ratio ($R_c = \sigma'_{1c} / \sigma'_{3c}$) and effective confining stress σ'_{3c} for a designated time period (defined as the aging period). The effect of aging on deformation response was investigated by subjecting the sand specimens to different aging periods prior to shearing.

3.6.4 Shearing

Aged specimens were sheared at a constant rate of displacement. This displacement - controlled motor drive allowed the post-peak behaviour to be recorded, in case the sand exhibited strain-softening.

(a)



(b)

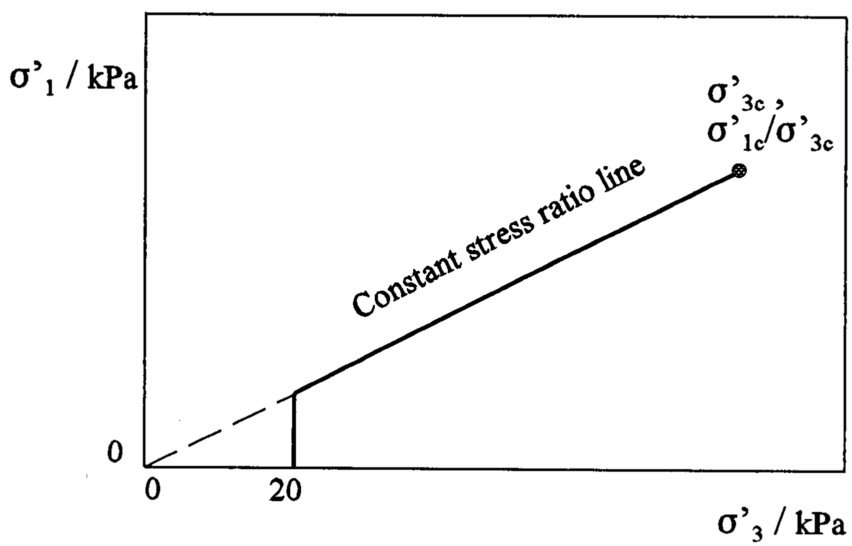


Figure 3.4 Consolidation paths shown in q - p space and σ'_1 - σ'_3 space

3.7 PERFORMANCE OF THE APPARATUS

3.7.1 Repeatability of test results

Confidence in experimental observations and results is enhanced by the ability to repeat tests. Repeatability depends on consistent reproduction of relative density, soil structure, measurement accuracy, and exact duplication of the test routine and the stress or strain path followed throughout the test. The test procedure described previously ensured that this was achieved to the highest degree. Typical results of repeated undrained tests on identical specimens of Fraser River sand with a 100 min aging period are shown in Figures 3.5 and 3.6. Figure 3.5 shows the consolidation paths in effective stress space, and strain development during the consolidation phase (σ'_{3c}=200 kPa, $\sigma'_{1c}/\sigma'_{3c}$=2.26). The stress-strain responses are shown in Figure 3.6. These plots demonstrate the excellent repeatability in terms of stresses and strains.

3.7.2 Rate of loading

For a test to be truly element test, the mobilized stresses and strains must be uniform throughout the specimen. Hence, for a strain path controlled test to be an element test, volumetric changes in the specimen should occur equally throughout its length. In the partially drained tests, water is either injected or extracted at the top of the specimen. Therefore, it was necessary to ensure that no significant gradient developed along the specimen. This was achieved by limiting the rate of effective stress changes to sufficiently small values during the partially drained tests.

(a)

(b)

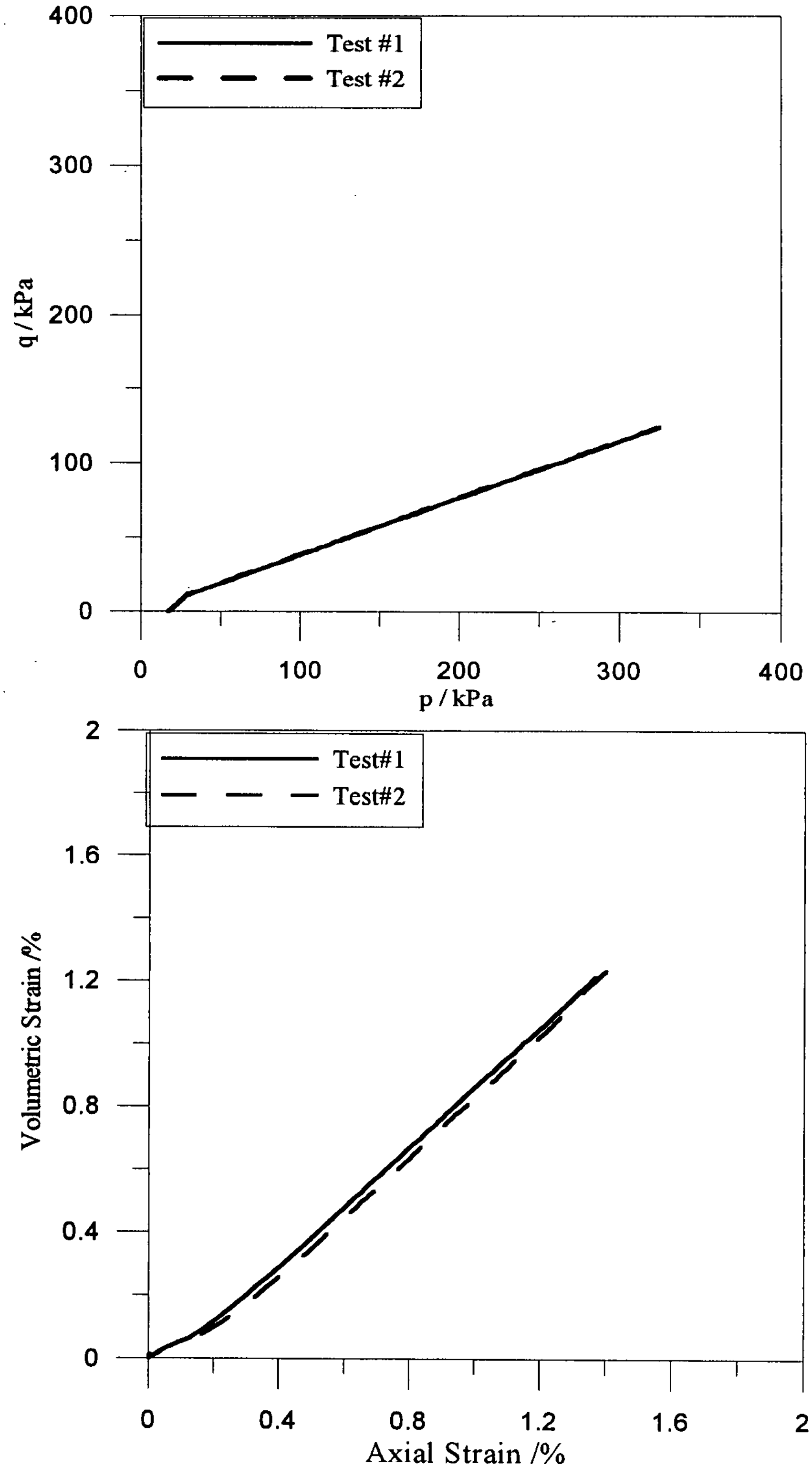


Figure 3.5 Repeatability of consolidation phase in effective stress and strain space

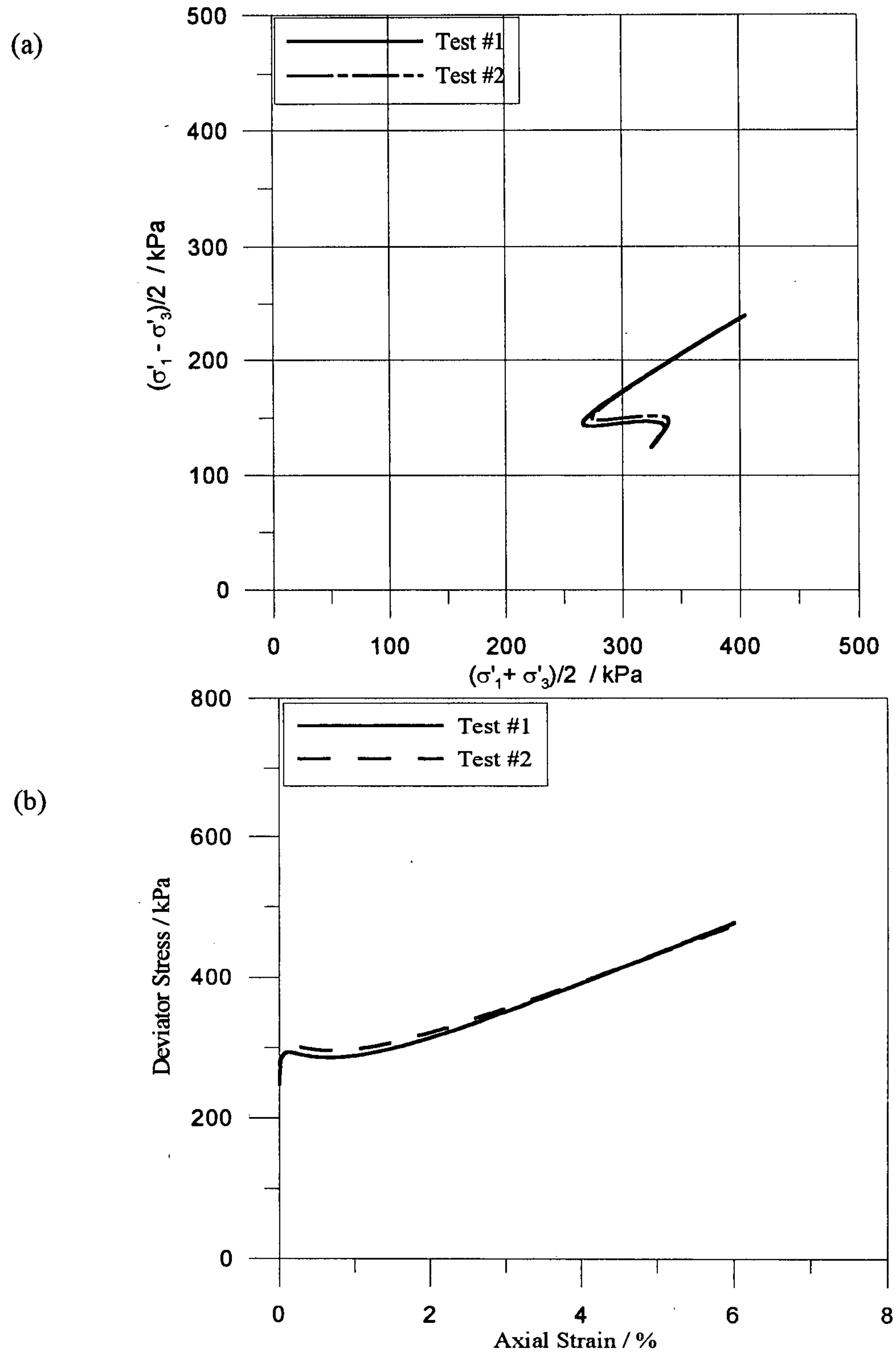


Figure 3.6 Repeatability of shear response in effective stress space and stress strain space

3.8 COEFFICIENT OF LATERAL EARTH PRESSURE AT REST (k_o)

The value of k_o for the Fraser River sand and North Sea sand was assessed by subjecting one test specimen to the stain path of $d\epsilon_v / d\epsilon_1 = +1$. The strain path along $d\epsilon_v / d\epsilon_1 = +1$ results in zero radial strain ($d\epsilon_r = 0$) thus representing the k_o conditions. Since the resulting radial effective stress increases continuously, a correction for membrane penetration is essential. The correction is done in addition to that required to the strain path applied. In other words, the specimen should be subjected to a strain path which has a one-to-one correspondence between axial strain and net skeleton volumetric strain. Results for Fraser River sand and North Sea sand are shown in Figure 3.7, which establish a value for k_o of 2.26 and 2.39 respectively.

3.9 TEST PROGRAM

The experimental investigation was performed on two different sands, namely the Fraser River sand (FRS) and North Sea sand (NSS). The main focus of the test program was on FRS, with complementary tests on NSS. Objectives were to examine the effect of aging on the stress-strain response of sand, and a characterization of sand with respect to undrained and drained behaviour and the partially-drained response.

A series of aging tests were performed on both the FRS and NSS. Based on these test results, a constant aging period was adopted for characterization of the undrained, drained and partially-drained behaviour. The partially-drained response was investigated by (i) subjecting specimens (reconstituted to identical initial state) to different degrees of drainage ($d\epsilon_v/d\epsilon_1$) and by (ii) imposing a targeted degree of drainage on specimens with different initial states. Tables 3.1 through 3.4 show the test program in detail.

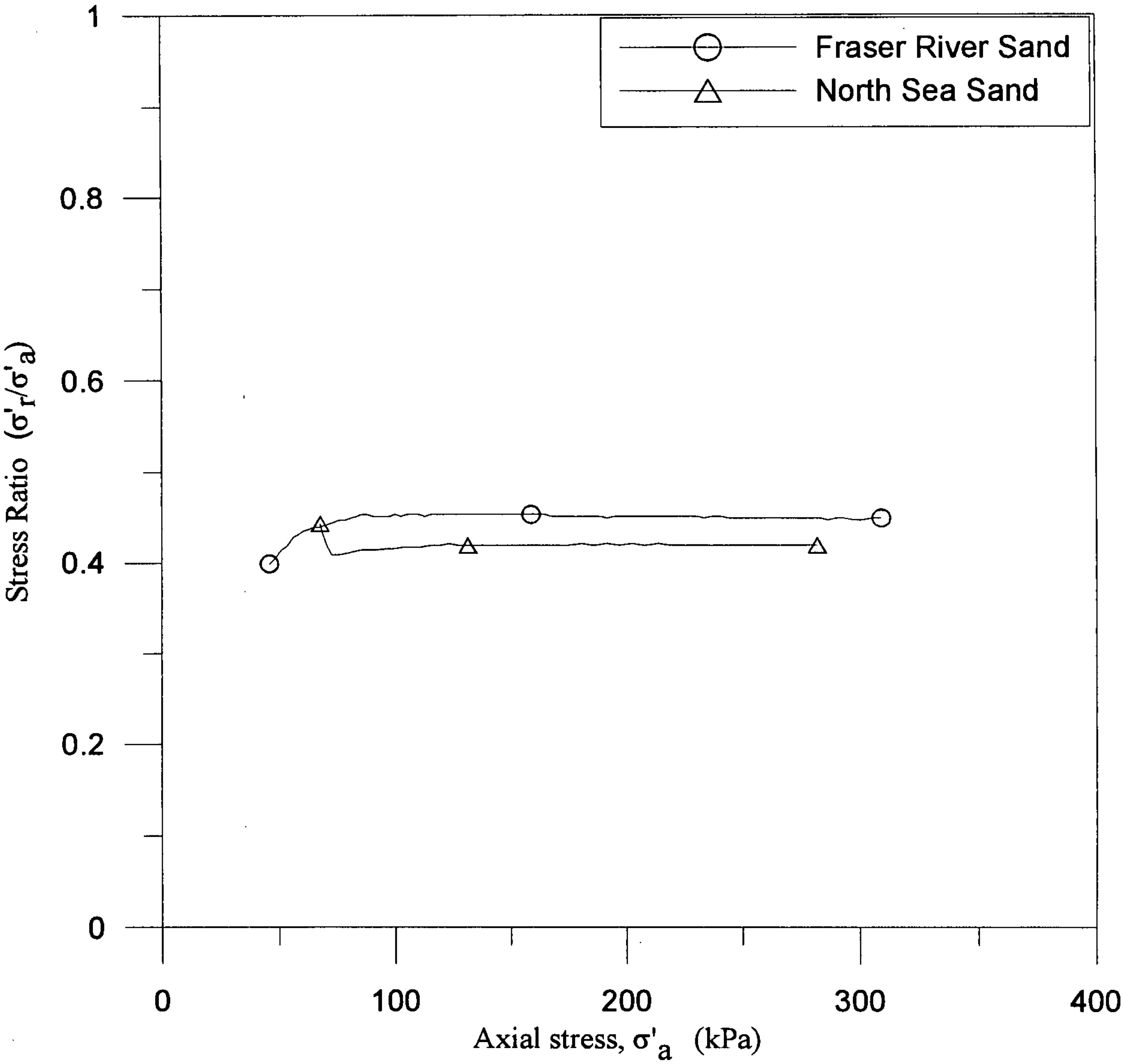


Figure 3.7. Determination of k_0 of Fraser River Sand and North Sea Sand

TEST TYPE	σ_{3c}' (kPa)	$\sigma_{1c}'/\sigma_{3c}'$	$d\varepsilon_v/d\varepsilon_1$	Aging Time / mins
TRULY UNDRAINED TESTS	200	k_0	0.00	1
				100
				1000
PARTIALLY DRAINED TESTS	200	k_0	-0.05	1
				100
				1000
TRULY UNDRAINED TESTS	100	k_0	0.00	1
				100
				1000

Table 3.1 Aging tests on Fraser River sand

TEST TYPE	σ_{3c}' (kPa)	$\sigma_{1c}'/\sigma_{3c}'$	$d\varepsilon_v/d\varepsilon_1$
TRULY UNDRAINED TESTS	50, 100, 200	k_0	0
	100	1.0, 1.5, k_0, 2.8	0
FULLY DRAINED TESTS	50, 100, 200	k_0	Not applicable
PARTIALLY DRAINED TESTS	100	k_0	-0.2, -0.1, -0.05, [0.0], 0.1
	50, 100, 200	k_0	-0.2
	100	1.0, 1.5, k_0, 2.8	

Table 3.2 Undrained, Drained and Partially drained test on Fraser River sand (with t_{aging} = 100 mins)

TEST TYPE	σ_{3c}' (kPa)	σ_{1c}'/ σ_{3c}'	$d\varepsilon_v/d\varepsilon_1$	Aging Time / mins
PARTIALLY DRAINED TESTS	200	k_0	-0.05	1
				100
				1000

Table 3.3 Aging tests on North Sea sand

TEST TYPE	σ_{3c}' (kPa)	σ_{1c}'/ σ_{3c}'	$d\varepsilon_v/d\varepsilon_1$
TRULY UNDRAINED TESTS	50, 100, 200	k_0	0
FULLY DRAINED TESTS	100	k_0	Not applicable
PARTIALLY DRAINED TESTS	200	k_0	[0.0], -0.05, -0.2
	50, 100, 200	k_0	-0.2

Table 3.4 Undrained, Drained and Partially drained test on North Sea sand (with t_{aging} = 100 mins)

CHAPTER 4

RESULTS AND DISCUSSION

As indicated in the previous chapters, experiments were conducted on two different sands, a Fraser River sand and a North Sea sand. Results for the Fraser River sand are presented and discussed first. The North Sea sand behaviour is presented and discussed thereafter.

For each sand, results of tests performed to investigate the effect of aging time on the stress-strain response are presented first. The results of undrained, partially drained and fully drained shear tests performed on specimens with constant aging period are presented thereafter.

4.1 RESULTS AND DISCUSSION OF FRASER RIVER SAND

4.1.1 Effect of aging on the stress-strain response

A series of tests were performed on Fraser River sand (FRS) to investigate the influence of aging effects on its undrained and partially drained stress-strain response. Six undrained and three partially drained tests were performed on specimens with identical initial states (stress state and relative density) but with different aging time periods of 1, 100 and 1000 minutes (see Table 3.1).

Figure 4.1 shows a typical effective stress path followed in order to reach an effective confining stress (σ_{3c}') and effective stress ratio (R) of 200 kPa and 2.26 respectively during consolidation phase. Following the consolidation phase, the specimens were aged to different times by keeping the stress state unchanged for periods of 1, 100 and 1000 mins. Stresses and strains were monitored over the aging period. The aged specimens were then subjected to shearing under either undrained or partially drained conditions. As described earlier in Chapter 3, an identical set of undrained tests with 100 mins of aging were performed to assess their repeatability (see Figures 3.5 and 3.6).

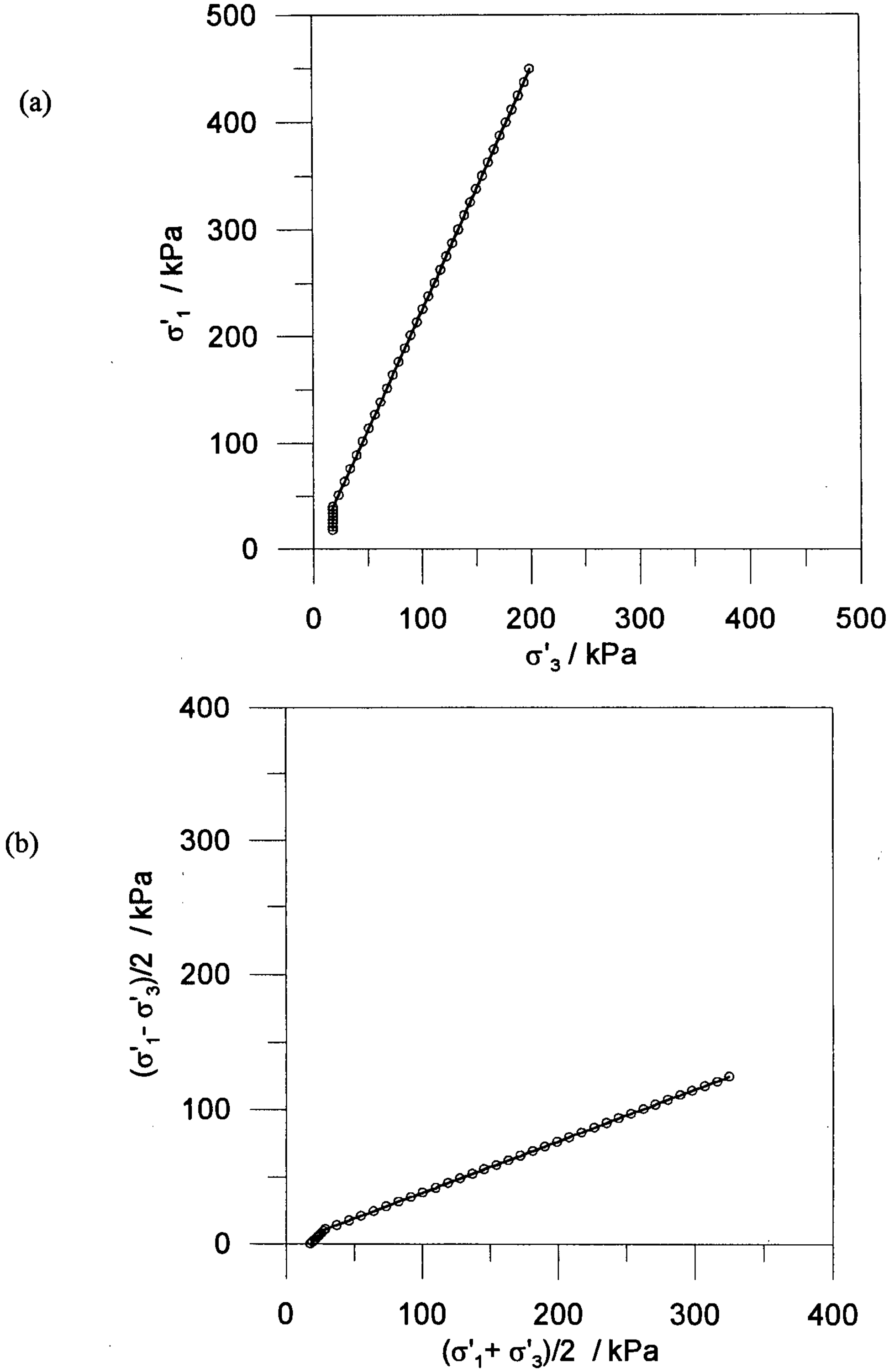


Figure 4.1 Consolidation path followed to reach σ'_{3c} = 200 kPa and σ'_{1c} / σ'_{3c} = 2.26

A total of three series of aging tests were performed to investigate the effect of aging on undrained and partially drained response of Fraser River sand A set of undrained tests and an other set of partially drained tests ($d\epsilon_v/d\epsilon_1$ = -0.05) were performed on specimens consolidated to effective confining stress of 200 kPa and effective stress ratio of 2.26. The third set of tests was performed on specimens with σ_{3c}' = 100 kPa and R = 2.26 under undrained conditions.

4.1.1.1. Aging test results and observations

All specimens exhibited a relative density of approximately 26-27 % at the end of consolidation. Following the consolidation phase, creep volumetric strains were monitored throughout the aging period. Figure 4.2 shows the typical creep strains developed during the aging phase: the strains are less than or equal to 0.1 %. It is apparent that the change in relative density due to volumetric strains of the order of 0.1 % is essentially negligible. Therefore the effect of aging on the shear response of specimens cannot be attributed to the densification of test specimens.

It can also be observed from Figure 4.2 that the rate of development of creep volumetric strains is very high at the beginning, but it diminishes with the time. Most of the creep strains occur over the first 100 mins. Quantitatively, about 75 % of the creep volumetric strains occur over the first 100 minutes.

The shear response of the specimens of different age is plotted in Figures 4.3, 4.4, and 4.5. Figure 4.3 shows the undrained stress-strain response of specimens (σ_{3c}' = 200 kPa, R=2.26) which were aged for 1, 100 and 1000 mins. The curves clearly demonstrate an effect of aging on the stress strain response of sand. The specimens with 100 and 1000 mins age exhibit a stronger and stiffer response compared with the 1 min aged specimen (see Figure 4.3). The PT strength and the CSR strength have increased with the period of aging. It is also apparent that the

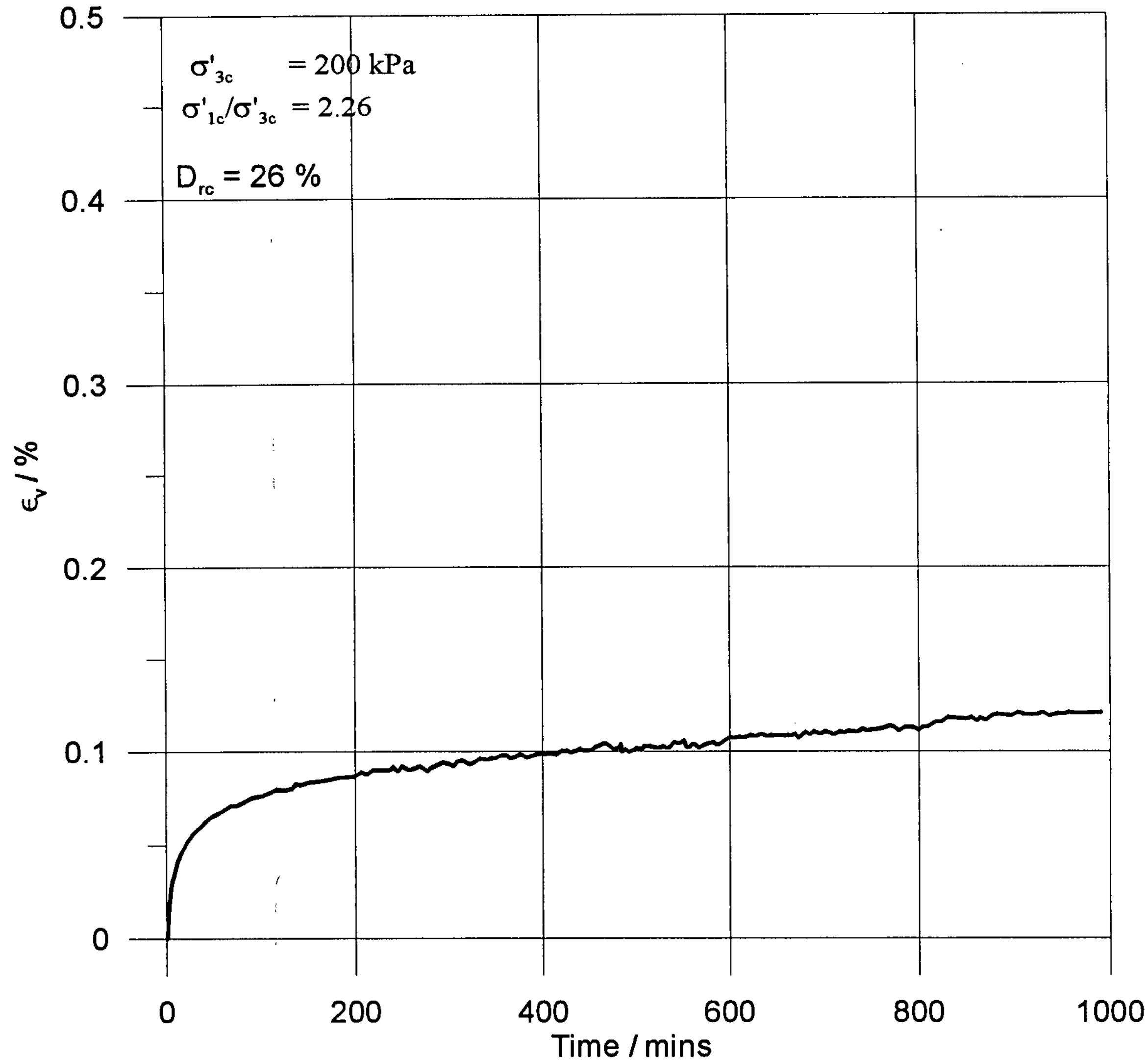


Figure 4.2 Volumetric strain development in Fraser River sand during the aging period

PT and CSR shear strengths of the specimens aged to 100 mins and to 1000 mins are relatively close in comparison to those between specimens aged to 1 min and 100 mins. The other important observation arising from the tests results is that the mobilised friction angle at Maximum Obliquity (ϕ_{MO}), Phase Transformation (ϕ_{PT}) and Critical Stress Ratio (ϕ_{CSR}) is essentially constant, regardless of the aging time.

Figure 4.4 shows the results of undrained tests performed on specimens consolidated to effective confining stress (σ_{3c}') of 100 kPa and effective stress ratio (R) of 2.26. The phenomena that were observed (an invariance of mobilized friction angles at Maximum obliquity, PT and CSR regardless of the aging period and a relatively close response of 100 mins and 1000 mins aged specimens compared with 1 min and 100 mins aged specimens) in the previous set of undrained test could be clearly seen in this case too.

The partially drained responses of specimens (σ_{3c}' = 200 kPa, R = 2.26) with different aging period (1, 100 and 1000 mins) which were subjected to a $d\epsilon_v/d\epsilon_1$ = -0.05 injection rate during shearing are plotted in Figure 4.5. It can be seen that the mobilized friction angles ϕ_{MO}, ϕ_{PT} and ϕ_{CSR} are, again, independent of the aging period, similar to that observed for the undrained cases. It can also be noted, as in the previous cases, that the response of 100 and 1000 mins aged specimens is relatively close compared with the 1 min and 100 mins aged specimens.

The PT and CSR shear strengths observed in all the aging tests were normalised with their corresponding effective confining stresses (σ_{3c}') and plotted in Figure 4.6. The undrained test at σ_{3c}' = 100 kPa and 1000 minutes aging did not exhibit any strain softening (see Figure 4.4 a), therefore the corresponding data point in Figure 4.6 was not available. A linear increase is evident in the PT and CSR shear strengths over the logarithm of aging period. The increase in CSR and PT shear strengths over the first 100 mins of aging is about 9 %, whereas the increase over the next 900 mins is only 3 %.

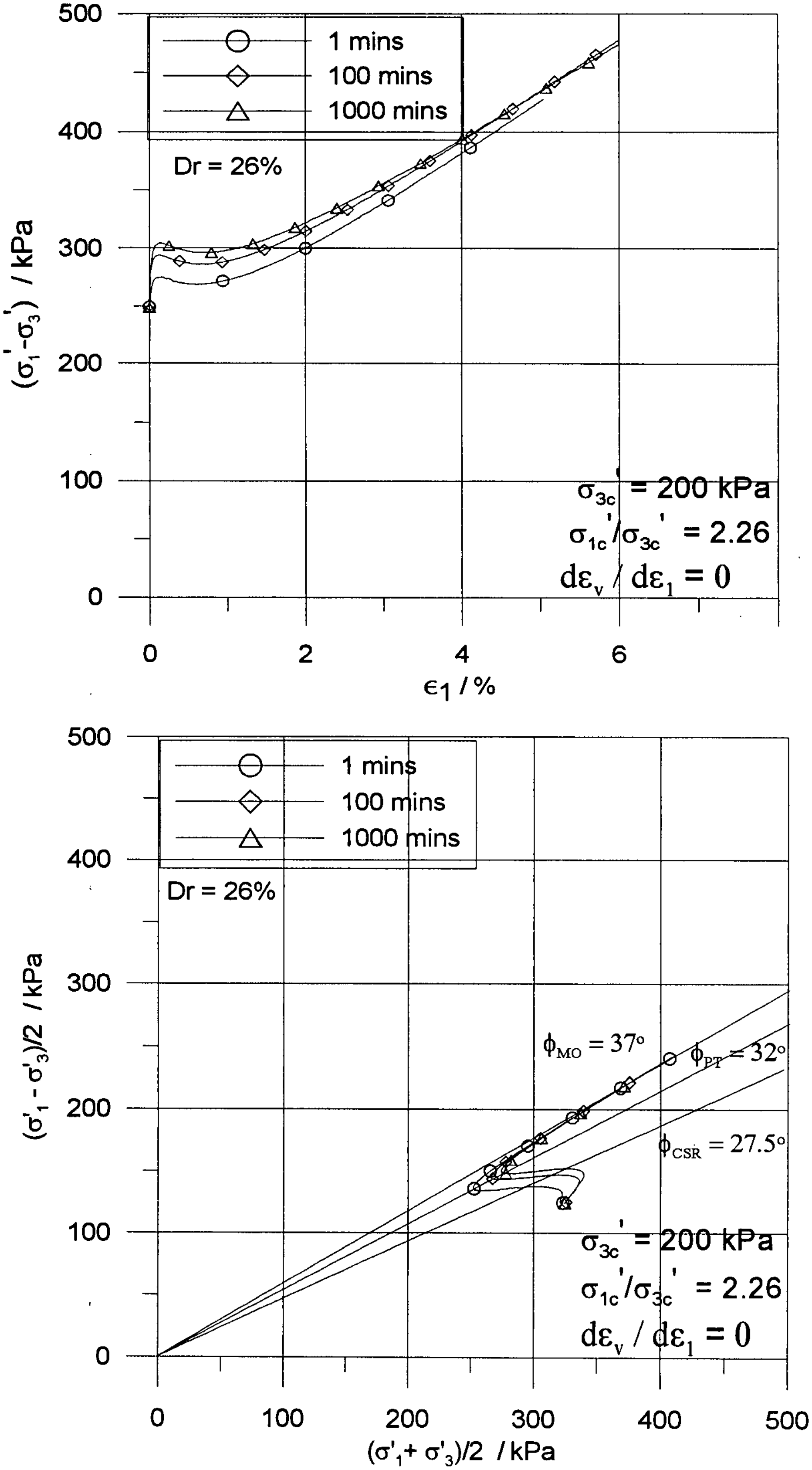


Figure 4.3 Effect of aging on the undrained response of loose Fraser River sand

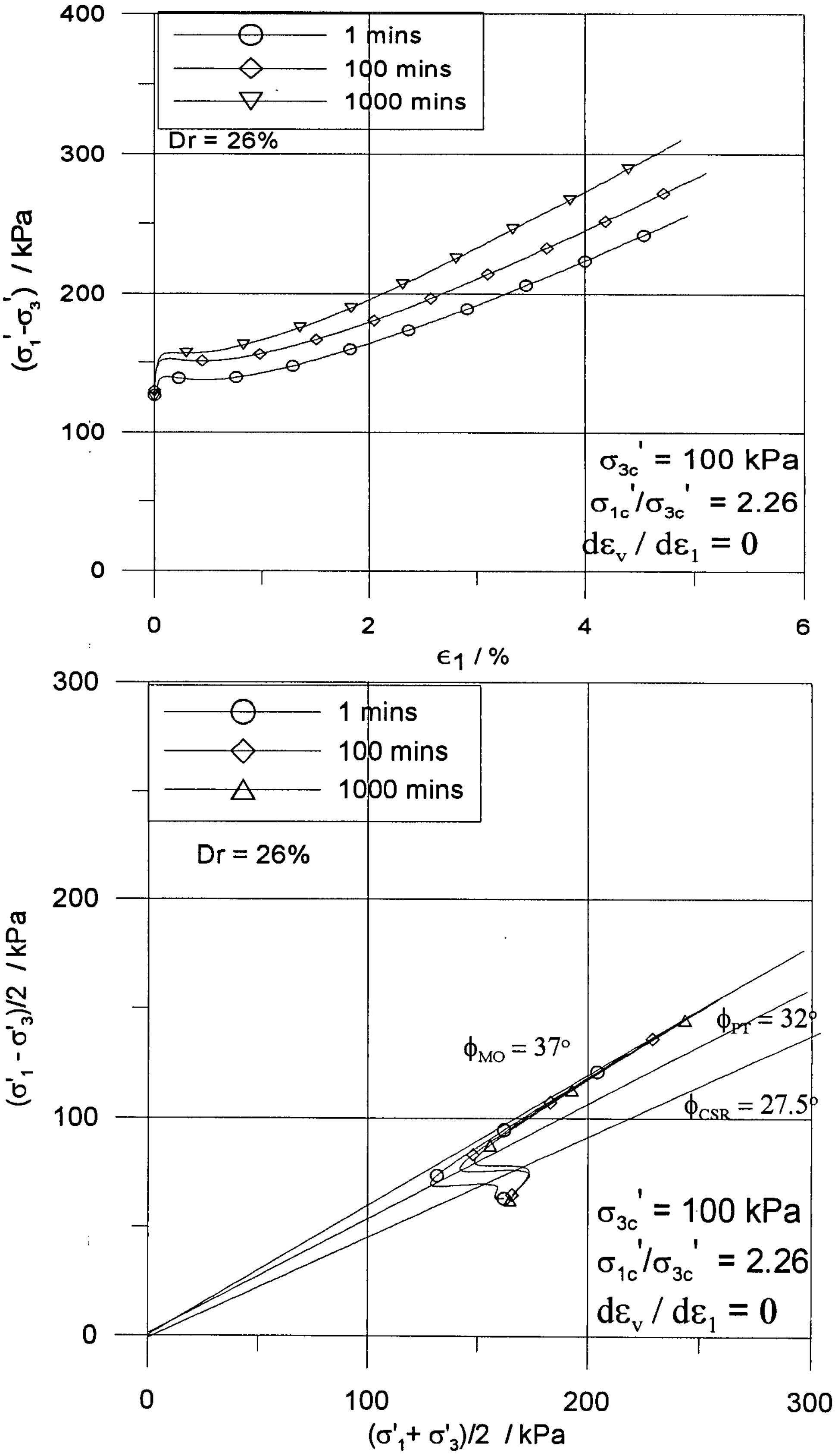


Figure 4.4 Effect of aging on the undrained response of loose Fraser River sand

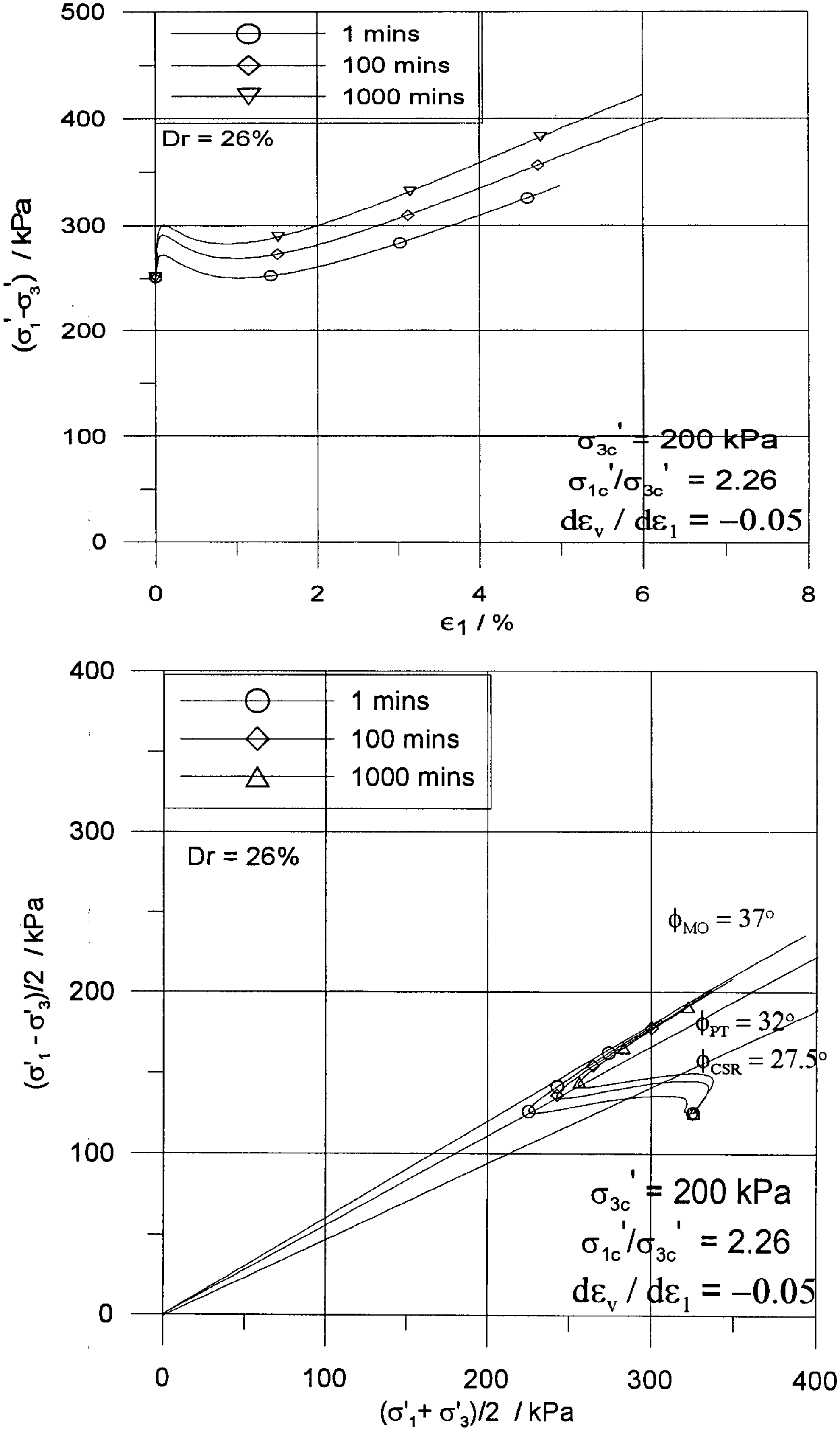


Figure 4.5 Effect of aging on the partially drained response of loose Fraser River sand

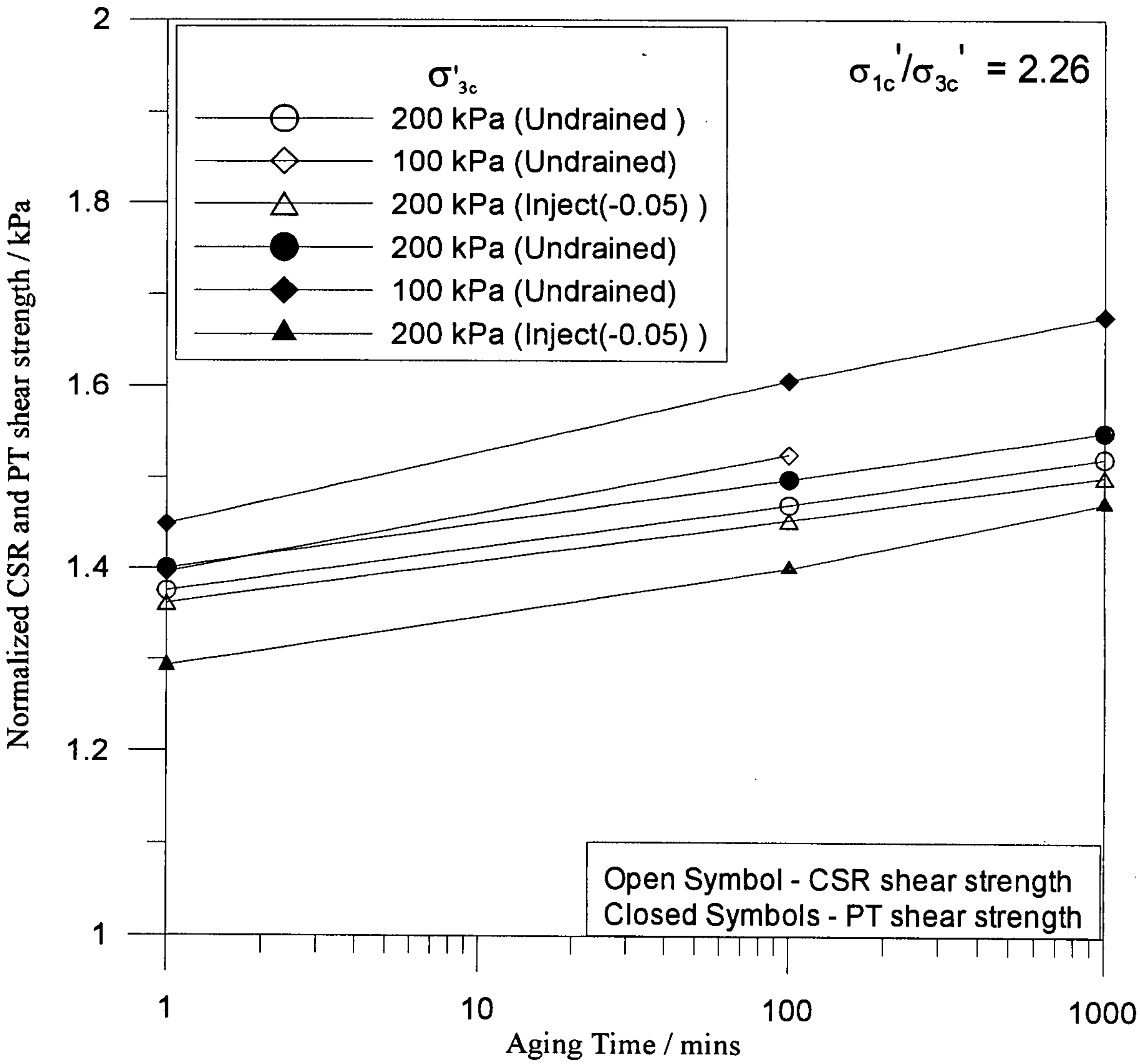


Fig. 4.6 Normalized CSR and PT shear strengths with aging period

4.1.1.2 Summary

- Series of undrained and partially drained tests were performed on reconstituted Fraser River sand specimens to investigate the effect of aging. Tests were carried out on specimens at the loosest state by aging them for different time periods (1, 100 and 1000 mins). The investigation was carried out on specimens consolidated to R = 2.26 and σ_{3c}' = 200, 100 kPa.
- The mobilized friction angle at Maximum Obliquity, Phase Transformation, and Critical Stress Ratio are found to be essentially constant regardless of the age of the specimens.
- Aging period has a significant influence on the undrained and partially drained response of Fraser River sand. Fresh dilative specimens becomes more dilative and contractive specimens become less contractive with aging. These effects may result in transforming a strain softening response into strain hardening response.
- The FRS exhibits a stronger stress strain response with increasing aging time period. Specimens aged for 100 mins and 1000 mins demonstrate a closer response than those aged to the 1 min and 100 mins. The CSR and PT shear strengths increase with aging time period. The gain in CSR and PT shear strengths over the first 100 mins is about 9% whereas the increase over the next 900 mins is only 3%.
- The stronger response of aged specimens cannot be attributed towards densification of the test specimens, because the additional volumetric strains (creep strains) developed during the aging period are in the order of 0.1 % or less.
- As more than 75% of the creep volumetric strains occur over the first 100 mins, and specimens aged to 100 mins and 1000 mins demonstrate a closer response to each other (with only a 3% difference in CSR and PT shear strengths) and given the difficulties in maintaining constant stresses over longer time periods, it judged that the use of a constant aging period of 100 mins for all the FRS test specimens is reasonable.

4.1.2 Undrained response of Fraser River sand

The undrained response of granular material has been investigated by many researchers. Yet the studies have been carried out on specimens prepared by different specimen reconstitution techniques such as water pluviation or moist tamping. It has been established that the fabric of natural alluvial deposits can be closely duplicated by preparing test specimens using the water pluviation technique. Further, most studies have not included control of the aging period prior to shear. Having established a reasonable constant aging period from preliminary tests, a series of undrained tests were carried out to investigate the response of aged Fraser River sand. In an attempt to simulate the possible real undrained behaviour, this study took into account both the effect of aging and the path of consolidation. Test specimens were consolidated along a targeted stress ratio line, as explained in previous chapters. Consolidated specimens were then aged for a nominal period of 100 mins prior to shearing.

4.1.2.1 The effect of confining stress and stress ratio on stress strain response

Undrained tests were carried out to investigate the effect of effective confining stress and effective stress ratio on the stress strain response of aged Fraser River sand. Three undrained tests were carried out on specimens consolidated to different effective confining stresses of 50, 100 and 200 kPa at the same stress ratio of 2.26. The stress strain response and the effective stress paths of the specimens are given in Figures 4.7 and 4.8. Another series of undrained tests were performed to investigate the influence of stress ratio. These tests were performed on specimens with same effective confining stress of 100 kPa, but different stress ratios of 1.0, 1.5, 2.26 and 2.8. The results are plotted in Figures 4.9 and 4.10.

Figure 4.7 shows the undrained stress strain response of specimens with σ'_{3c} of 50, 100 and 200 kPa and constant stress ratio of 2.26. It can be seen from this figure that the specimen with σ'_{3c} of 50 kPa is strain hardening whereas the other two specimens with σ'_{3c} of 100 and 200

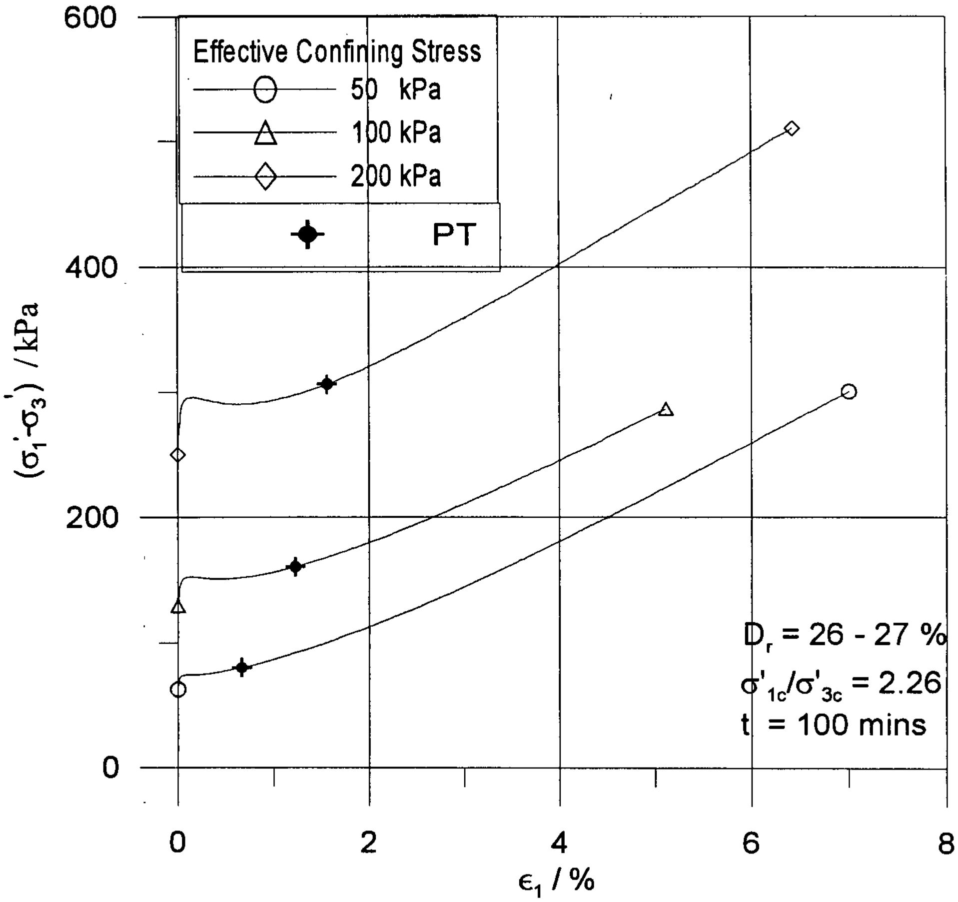


Figure 4.7 Effect of confining stress on undrained stress strain response of FRS

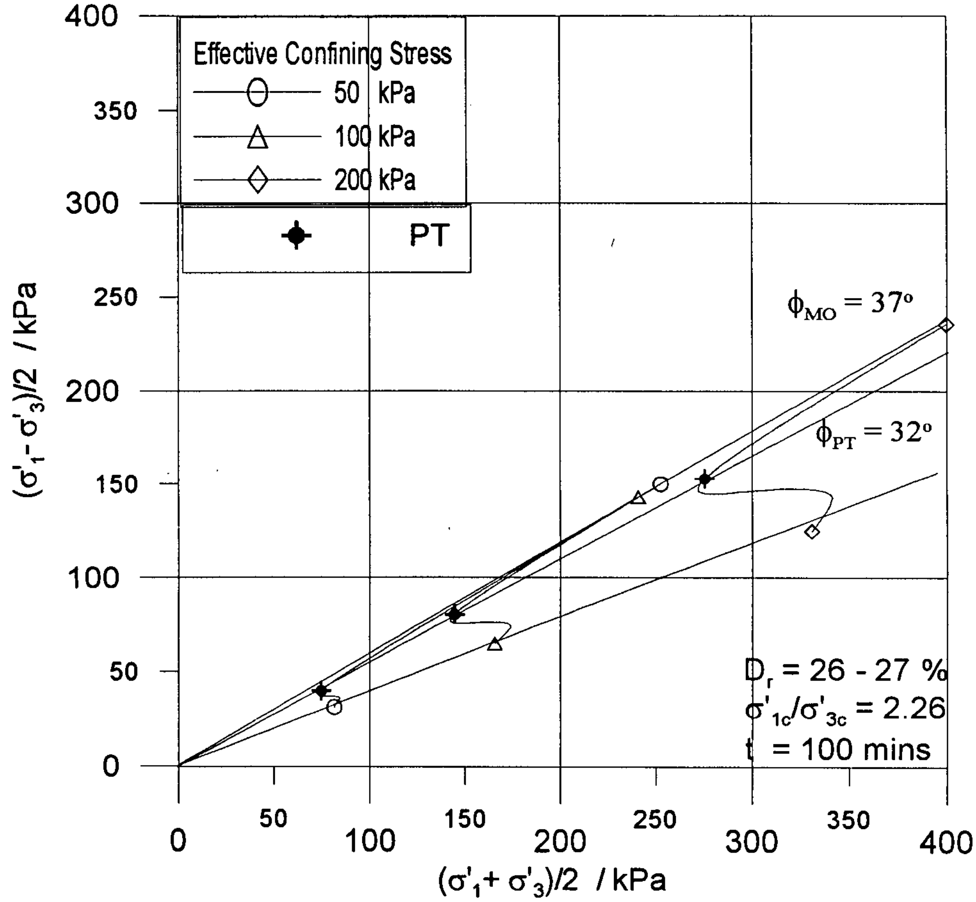


Figure 4.8 Effect of confining stress on undrained stress strain response (effective stress space) of FRS

kPa are mildly strain softening. The strain softening behaviour can be clearly seen in the specimen with σ'_{3c}= 200 kPa. These observations indicate that the aged FRS becomes more contractive with increasing effective confining stress. The point of maximum pore water pressure (PT point) is also marked on the stress strain plots given in Figure 4.7. It is clear from this plot that the phase transformation occurs at larger strain level for larger effective confining stress. The effective stress paths plotted in Figure 4.8 shows the friction angle at Maximum Obliquity (MO) and at Phase Transformation (PT) is not influenced by σ'_{3c}.

Figures 4.9 and 4.10 show the stress strain response and the corresponding effective stress paths of the specimens having σ'_{3c} of 100 kPa and stress ratio of 1.0, 1.5, 2.26 and 2.8. Specimens with stress ratios of 1.0 and 1.5 are strain hardening, whereas the one with a stress ratio of 2.26 is contractive. The response in the effective stress space indicates that the friction angle at Maximum Obliquity and Phase Transformation is essentially constant regardless of stress ratio (see also Figure 4.11).

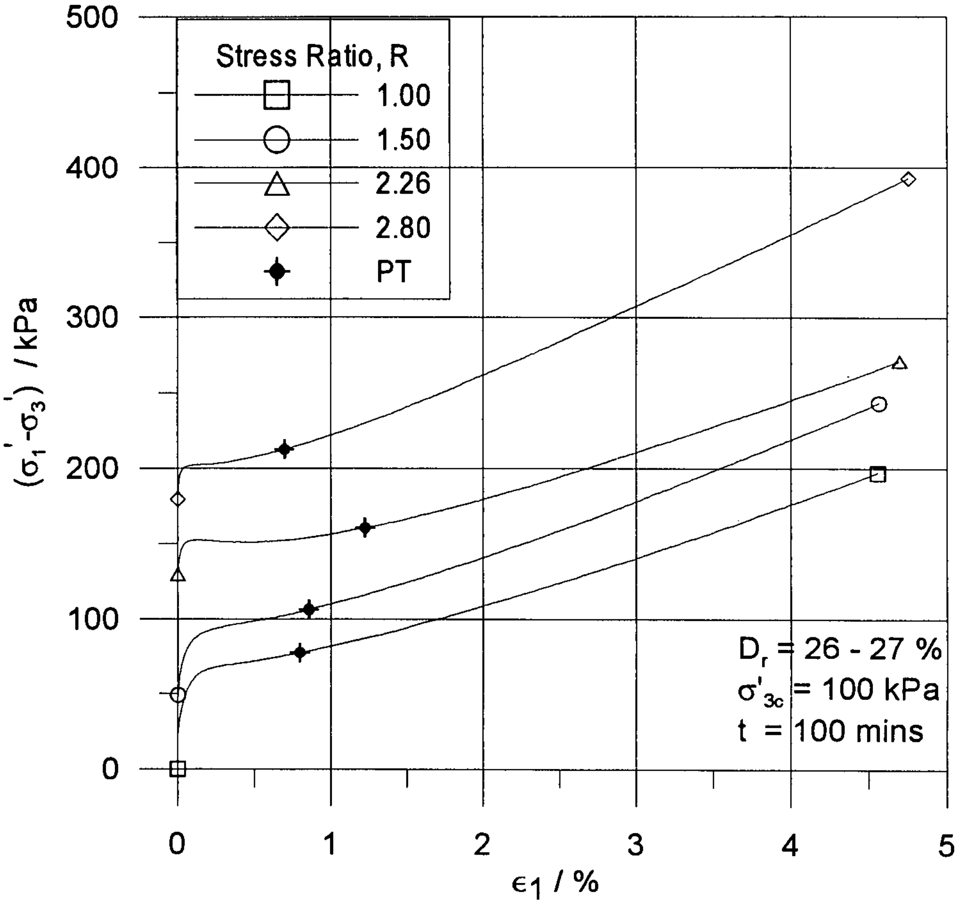


Figure 4.9 Effect of stress ratio on undrained stress strain response of FRS

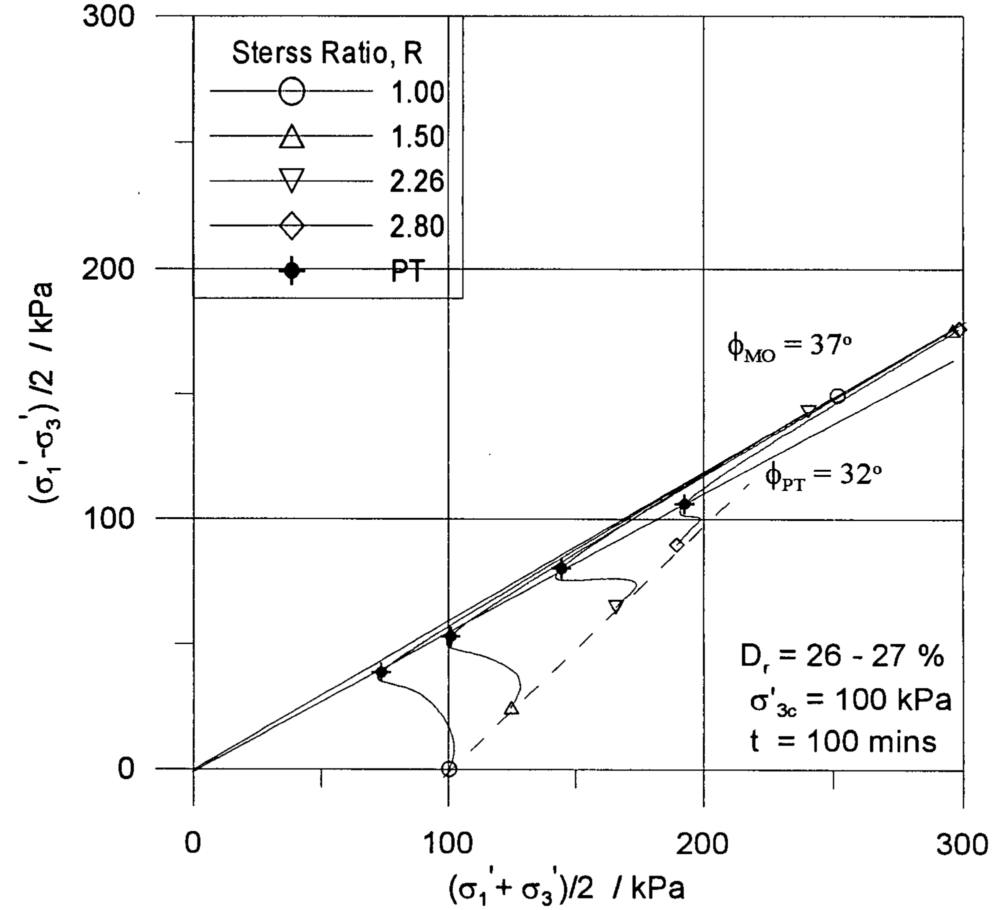


Figure 4.10 Effect of stress ratio on undrained stress strain response (effective stress space) of FRS

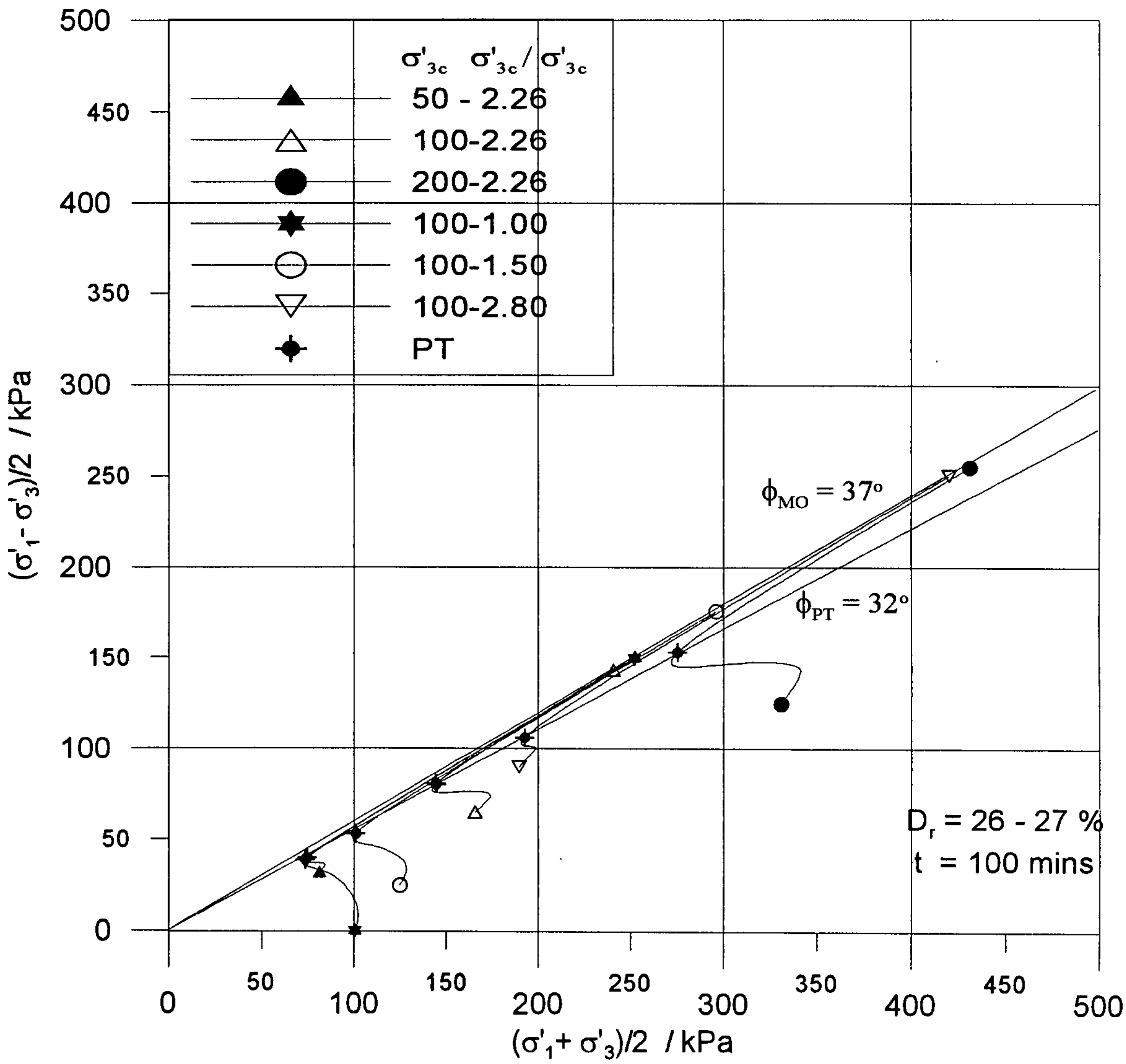


Figure 4.11 Undrained response of FRS in effective stress space

4.1.2.2. Summary

- A series of undrained tests were carried out on specimens with an aging period of 100 mins. Tests were performed at different effective confining stresses (50, 100 and 200 kPa) for a constant stress ratio (2.26), and at different stress ratio (1.0, 1.5, 2.26 and 2.8) with a constant effective confining stress (100 kPa).
- Experimental results indicate that FRS under undrained loading at its loosest density (a relative density of 26%) state is strain hardening at low effective stress (50 kPa). It becomes strain softening with higher effective confining stresses (100 and 200 kPa). These observations indicate that FRS becomes more contractive with higher effective confining stress.
- Undrained tests carried out at different stress ratios indicate that FRS becomes more vulnerable to strain softening with increasing stress ratio.
- The general observation for aged FRS specimens, of a more contractive behaviour with higher effective confining stress and stress ratio, agrees with previous findings of Eliadorani (2000) on unaged specimens.

4.1.3 Partially Drained Response of Fraser River sand

In most studies carried out to investigate the liquefaction potential of saturated granular soils, it has been assumed that there is little time for drainage to occur and therefore the triggering of liquefaction will occur in an undrained fashion. But, field observations and shake table studies show that there will be some degree of drainage and associated volume changes, even during a short duration event. Eliadorani (2000) conducted a comprehensive laboratory test program, and found that a small imposed volumetric strain may change a strain hardening response into a strain softening type of response. His works were mainly on specimens with a few minutes of aging. Further, most of his specimens were first consolidated to the target effective confining stress by increasing the all around pressure and then axially loaded to reach the target stress ratio.

In this study, a series of tests were performed on two sands taking into account the effect of aging and consolidation path. As discussed in the previous sections, specimens were consolidated along the k_0 path and then aged for 100 minutes prior to shearing. Because loose specimens are more susceptible to liquefaction, tests were made on specimens at their loosest state.

The partially drained response of the Fraser River sand was first investigated by imposing a different magnitude of small volumetric strain to each test specimen during shear. The volumetric strain was applied by keeping $d\epsilon_v/d\epsilon_1$ constant throughout the test. A critical magnitude of $d\epsilon_v/d\epsilon_1$ was then selected, and the influence of effective confining stress and stress ratio on the partially drained response subsequently examined.

4.1.3.1 The Effect of Small Imposed Volumetric Strains on the Stress Strain Response

A series of partially drained tests was performed to investigate the effect of small imposed volumetric strains on the stress strain response of Fraser River sand. Tests were carried out on specimens with σ'_{3c} of 100 kPa and stress ratio of 2.26 for $d\epsilon_v/d\epsilon_1$ of +0.10, -0.10 and -0.20. The stress strain response of the specimens and the effective stress paths followed are given in Figures 4.12 and 4.13 together with the undrained response for comparative purposes.

The effect of a small imposed volumetric strain on the stress strain response can be clearly seen in Figure 4.12. The undrained response ($d\epsilon_v/d\epsilon_1 = 0$) exhibits a very slight strain softening response, whereas specimens subjected to $d\epsilon_v/d\epsilon_1$ of -0.1 and -0.2 are apparently strain softening. Further, it can also be seen that the specimen subjected to undrained condition exhibits strain softening for a short period but, then it strain hardens. The specimen subjected to $d\epsilon_v/d\epsilon_1$ of –0.1 exhibits the strain softening response for longer period compared to the one subjected to the undrained response. The specimen subjected to a $d\epsilon_v/d\epsilon_1$ of –0.2 yields a fully strain softening type of response. These observations indicate the magnitude of imposed expansive volumetric strain, though small, dramatically influences the stress strain response of the sand. Figure 4.12 also demonstrates that a small imposed contractive volumetric strain ($d\epsilon_v/d\epsilon_1$ = +0.1) makes the material more strain hardening. A contractive volumetric strain results in a limited development excess pore water pressure, as seen in Figure 4.13.

Figure 4.13 also clearly shows the effect of small imposed volumetric strain in effective stress space. Points corresponding to the critical stress ratio (CSR) and maximum phase transformation (PT) are also indicated. It can be seen from the plot that the points corresponding to critical stress ratio, phase transformation and maximum obliquity are not in line with the origin. This suggests that the mobilised friction angle at the critical stress ratio depends on the degree of drainage. Still, the gradients of lines connecting the origin and the corresponding

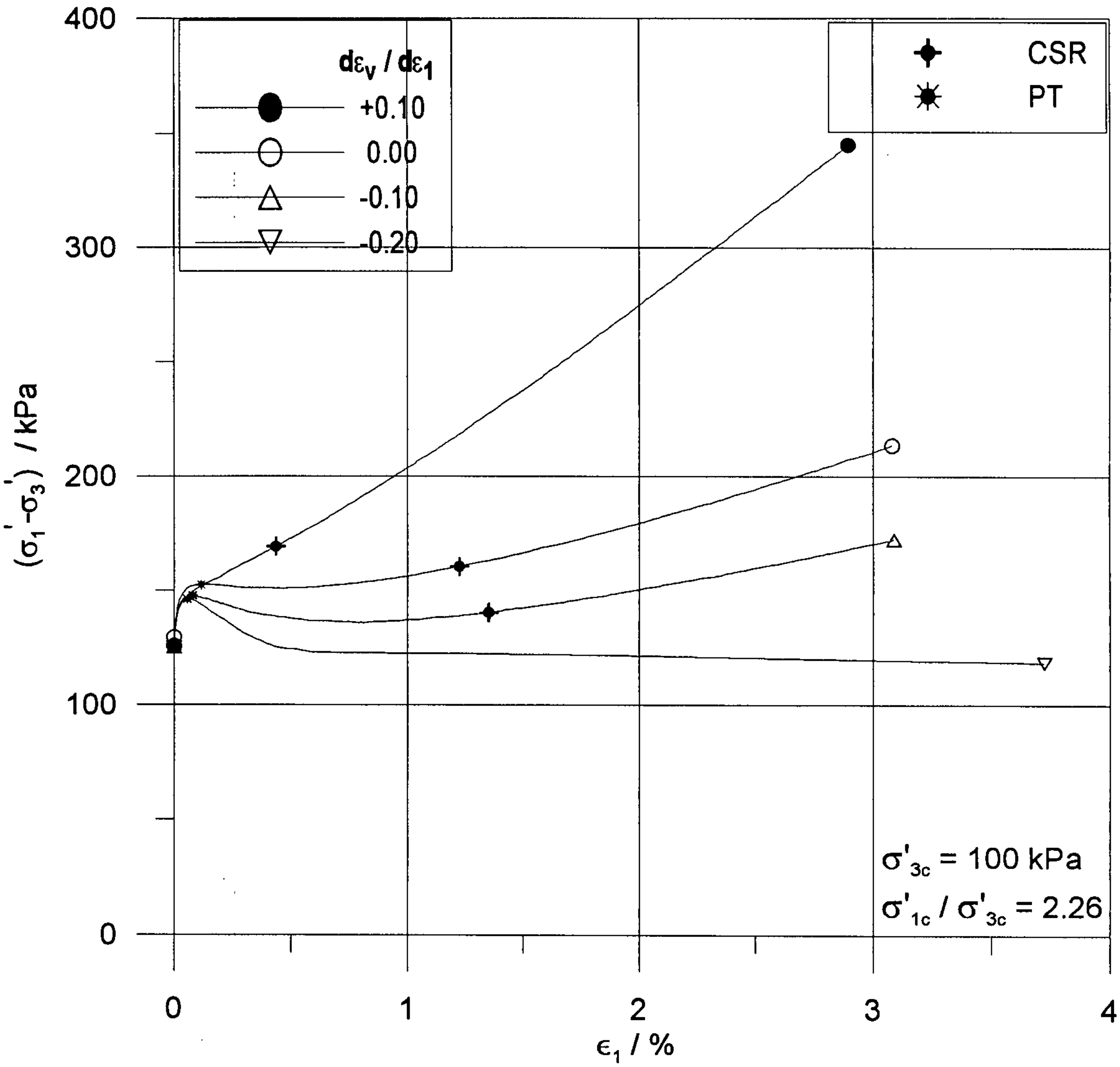


Figure 4.12 Effect of small imposed volumetric strains on stress strain response of FRS

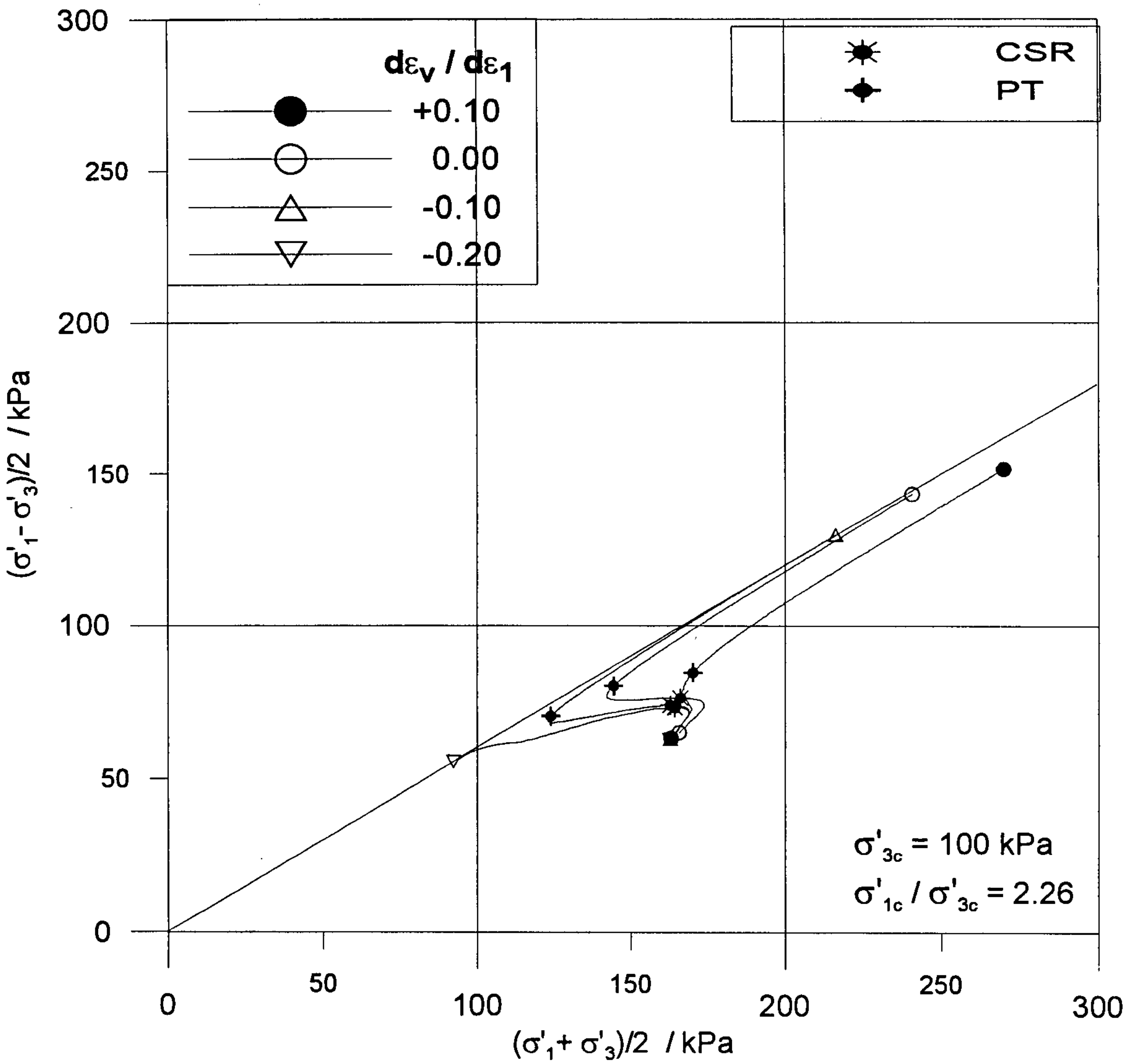


Figure 4.13 Effect of small imposed volumetric strains on stress strain response of (effective stress space) FRS

points are very close. Therefore, the change in mobilised friction angle due to drainage conditions is small.

As seen in Figures 4.12 and 4.13, the specimen subjected to a $d\epsilon_v/d\epsilon_1$ of –0.2 exhibits a fully strain softening response. This type of continuously strain softening response is of concern for flow failures. In an attempt to further explore this crucial partially drained response, additional tests were performed at the same injection rate of $d\epsilon_v/d\epsilon_1$ = –0.2 at which the specimen was fully strain softening.

4.1.3.3 Partially drained response of Fraser River sand, $d\epsilon_v/d\epsilon_1$ = -0.2

A series of partially drained tests was performed at a $d\epsilon_v/d\epsilon_1$ of –0.2 at which FRS exhibits fully strain softening response. The tests were carried out to investigate the influence of effective confining stress and stress ratio on the partially drained response. Therefore, a set of tests was performed at constant stress ratio of 2.26 with different effective confining stresses of 50, 100 and 200 kPa (see Table 3.2). Another set of tests was done to investigate the effect of stress ratio at constant confining stress of 100 kPa. These tests were performed at stress ratios of 1.0, 1.5, 2.26 and 2.8 (See table 3.2). Results were given in Figures 4.14 through 4.15.

Figure 4.14 shows the results of partially drained tests carried out to investigate the influence of effective stress on the stress strain response. Points corresponding to the critical stress ratio are marked on the stress strain curves. Although all specimens are fully strain softening, the effect of confining stress can be seen at small strain levels. At an effective confining stress of 50 kPa, the specimen reaches its critical stress ratio at 0.03 % axial strain, whereas at 100 and 200 kPa the specimens reach their CSR at 0.06 and 0.09 % axial strains respectively.

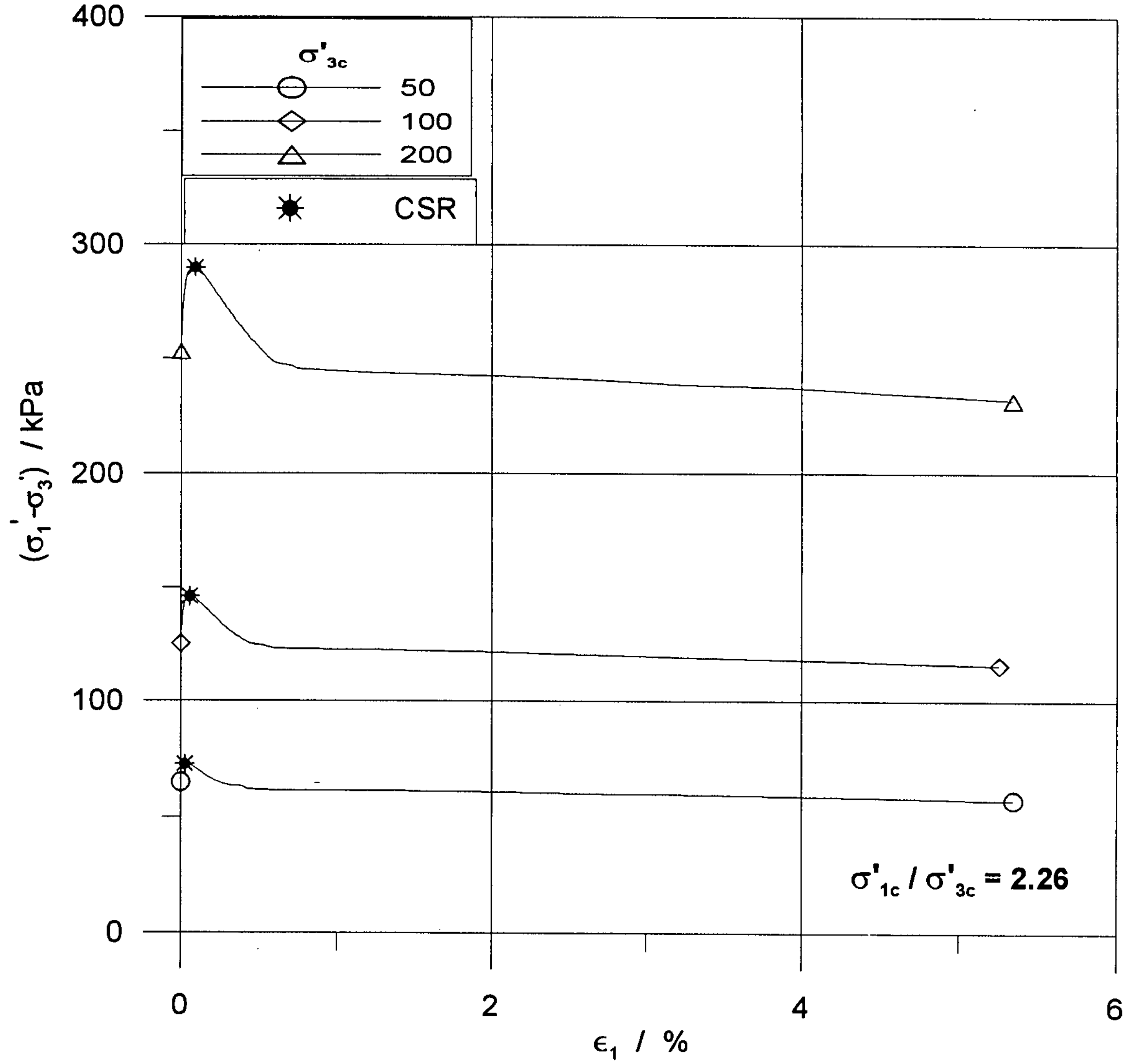


Figure 4.14 Effect of confining stress on stress strain response of FRS subjected to $d\varepsilon_v / d\varepsilon_1 = -0.2$

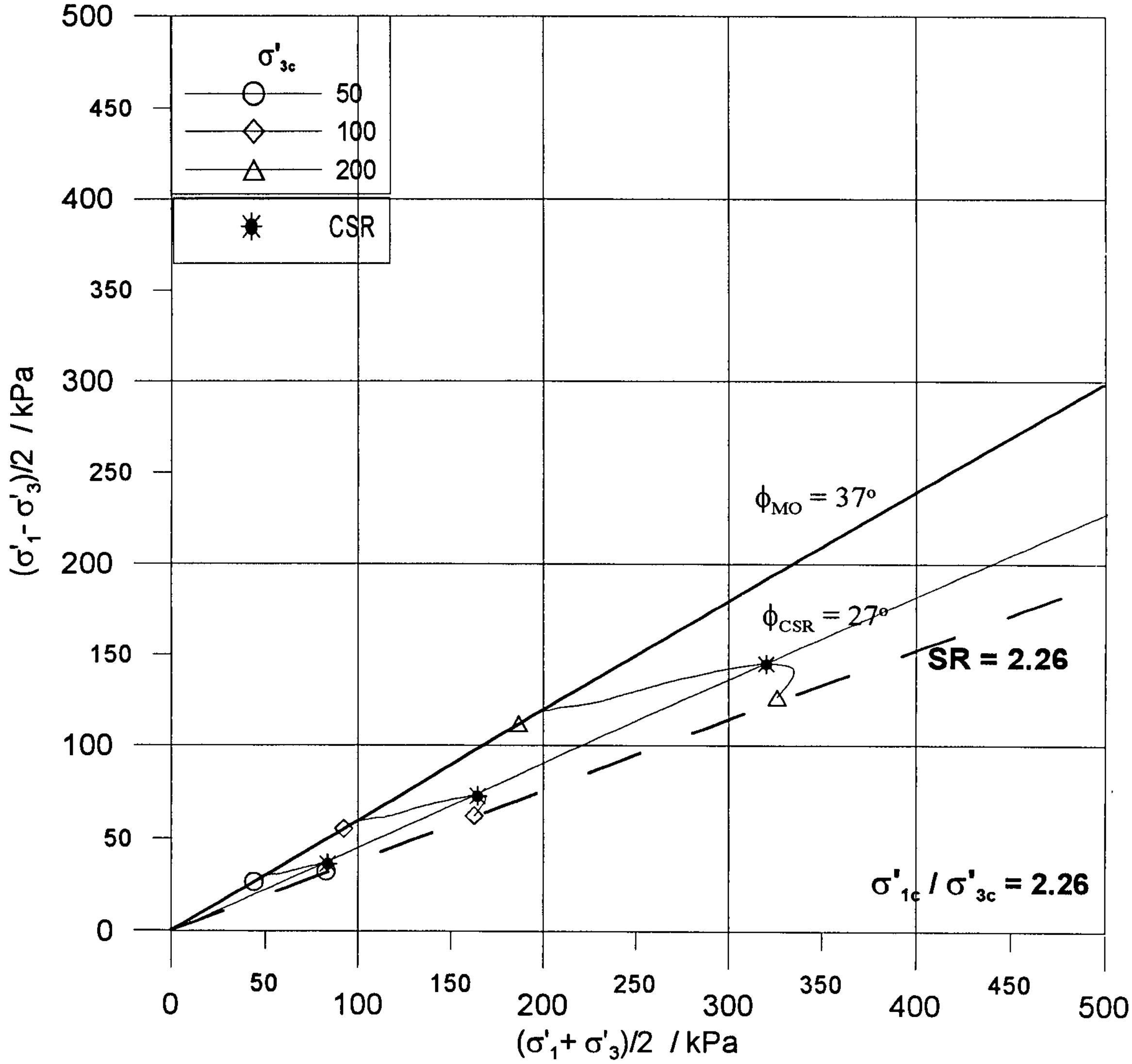


Figure 4.15 Effect of confining stress on stress strain response of FRS (effective stress space) subjected to $d\varepsilon_v / d\varepsilon_1 = - 0.2$

Figure 4.15 shows the response in effective stress space, and the mobilised friction angles at maximum obliquity and critical stress ratio. It clearly indicates that the mobilised friction angles are constant regardless of the effective confining stress under constant $d\epsilon_v/d\epsilon_1$.

Results of partially drained tests carried out at different stress ratios with constant confining stress are plotted in Figures 4.16 and 4.17. Again, points of critical stress ratio are marked on the curves. The stress strain responses given in Figure 4.16 clearly show that all specimens are strain softening regardless of stress ratio. A subtle difference is evident in the response at small strain levels. The specimen with stress ratio of 2.8 reaches its CSR at 0.04 % axial strain, whereas specimens with stress ratios of 2.26, 1.5 and 1.0 reach their CSR at 0.06, 0.13 and 0.18 % axial strains respectively. These observations indicate soil element at higher stress ratio reaches its CSR at smaller strain levels, whereas an element at lower stress ratio must attain larger strain levels to strain soften. Figure 4.17 shows the partially drained response of the specimens in effective stress space. Mobilised friction angles at maximum obliquity and critical stress ratio are established and marked on the curves. Each friction angle is found to be constant regardless of the stress ratio. The specimen with $\sigma_{1c}'/\sigma_{3c}'=2.8$ is in the region of strain softening, given that its effective stress ratio is higher than the stress ratio corresponding to the PT state. Therefore, the specimen started to strain soften almost immediately upon shearing (see Figure 4.16). The influence of aging (100 mins) may be the reason for the small gain in the deviator stress at the start of the test.

Figure 4.18 shows the drained, undrained and partially drained response of identical specimens with σ_{3c}' = 100 kPa and $\sigma_{1c}'/\sigma_{3c}'=2.26$. Inspections shows that the partially drained condition of $d\epsilon_v/d\epsilon_1 = -0.2$ results in a significantly different response compared with the stronger response of other two conditions. The deviator stress in the drained condition increases at a diminishing rate with strain, to attain a constant value. In contrast, the undrained condition

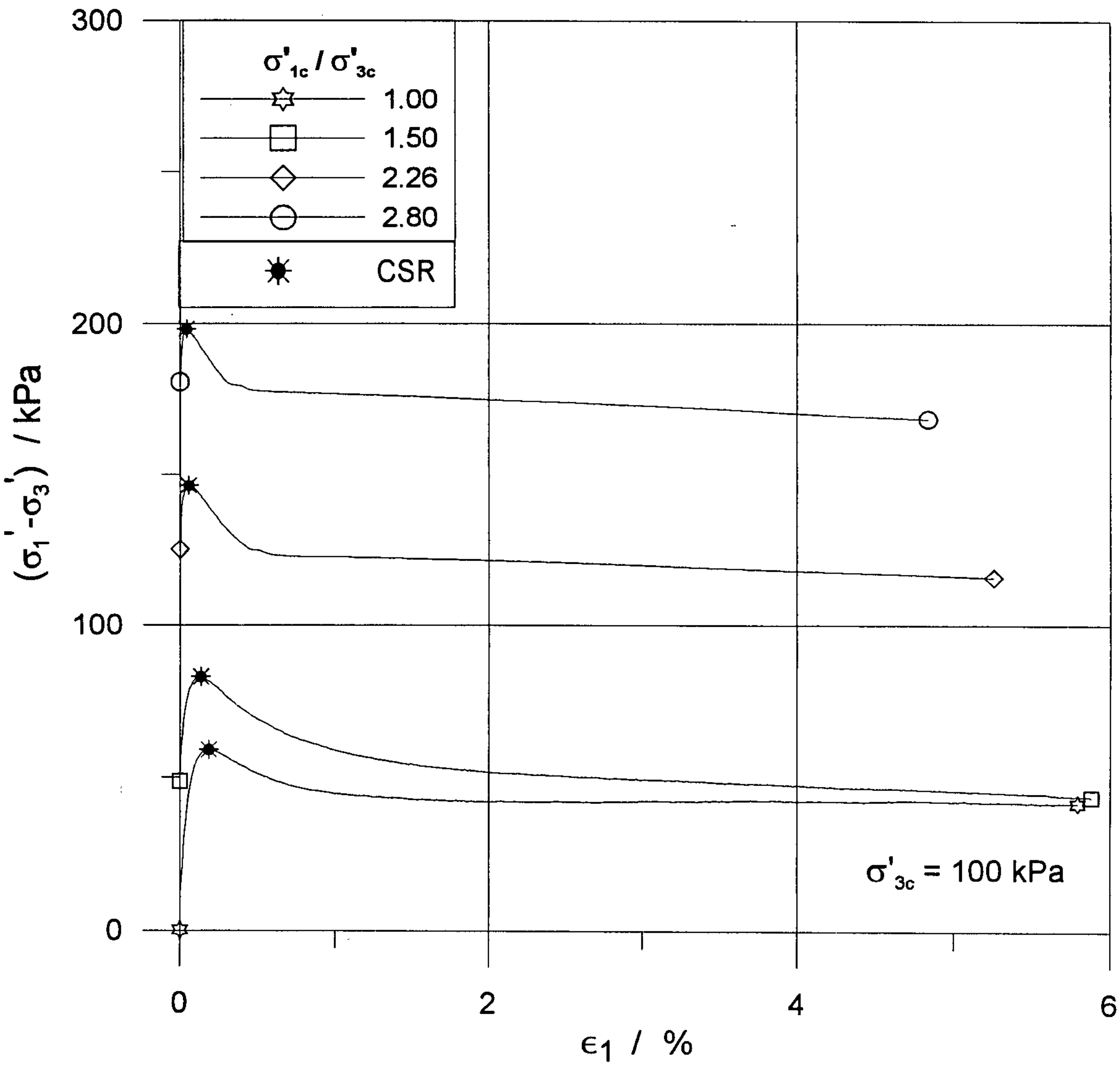


Figure 4.16 Effect of stress ratio on stress strain response of FRS subjected to $d\varepsilon_v / d\varepsilon_1 = -0.2$

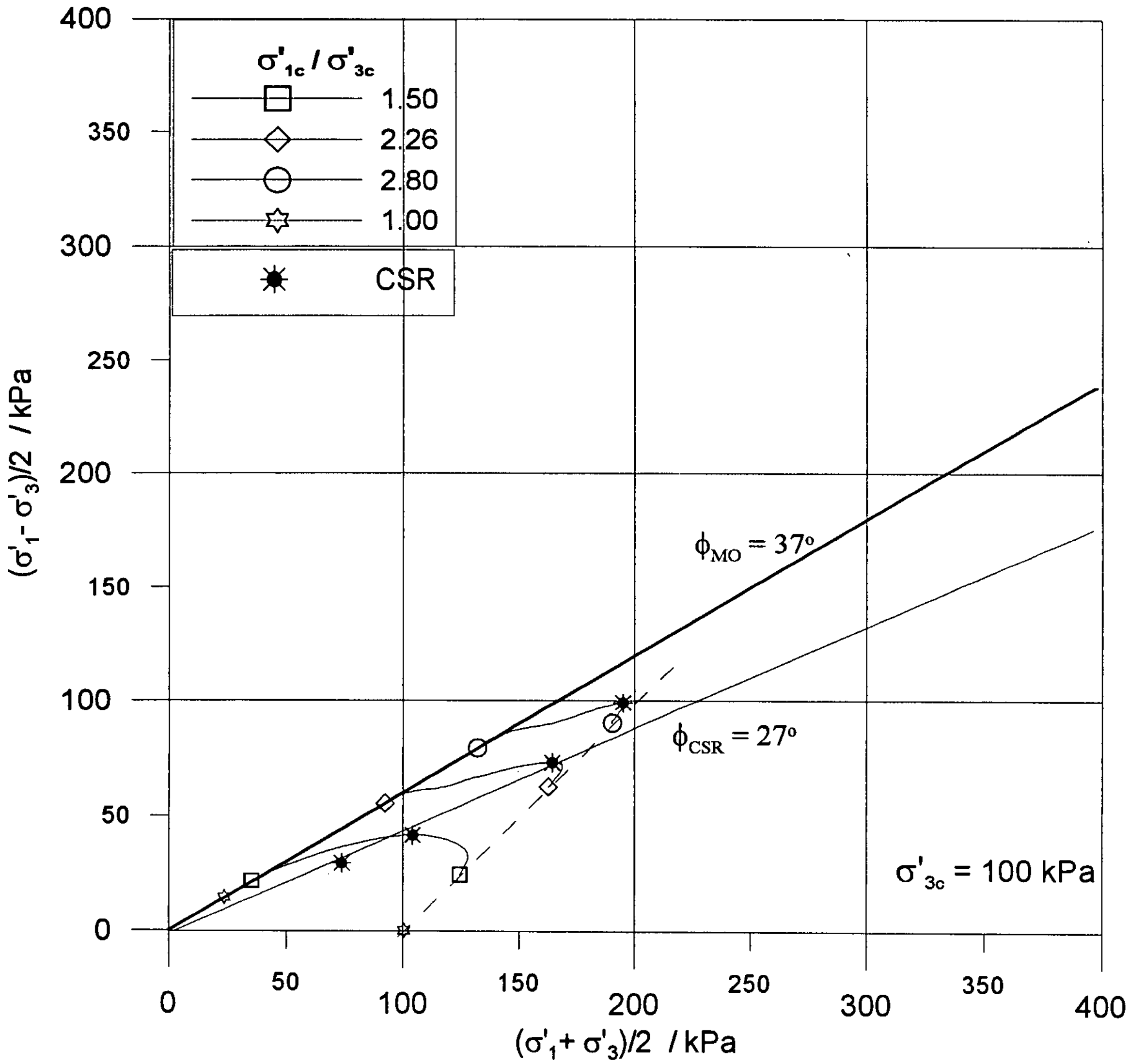


Figure 4.17 Effect of stress ratio on stress strain response of FRS (effective stress space) subjected to $d\varepsilon_v / d\varepsilon_1 = -0.2$

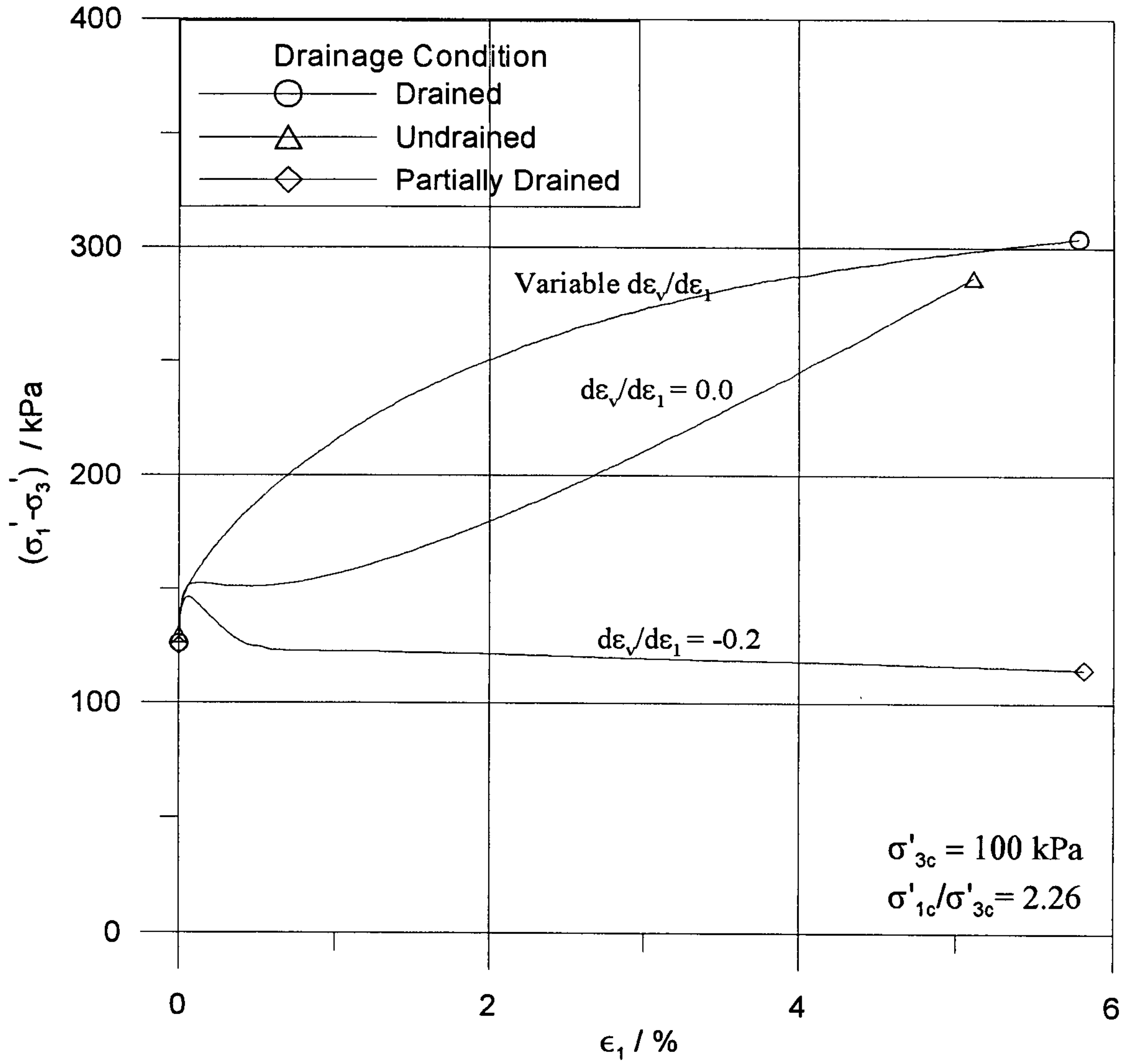


Figure 4.18 Comparison of Drained , Undrained and Partially drained response of FRS

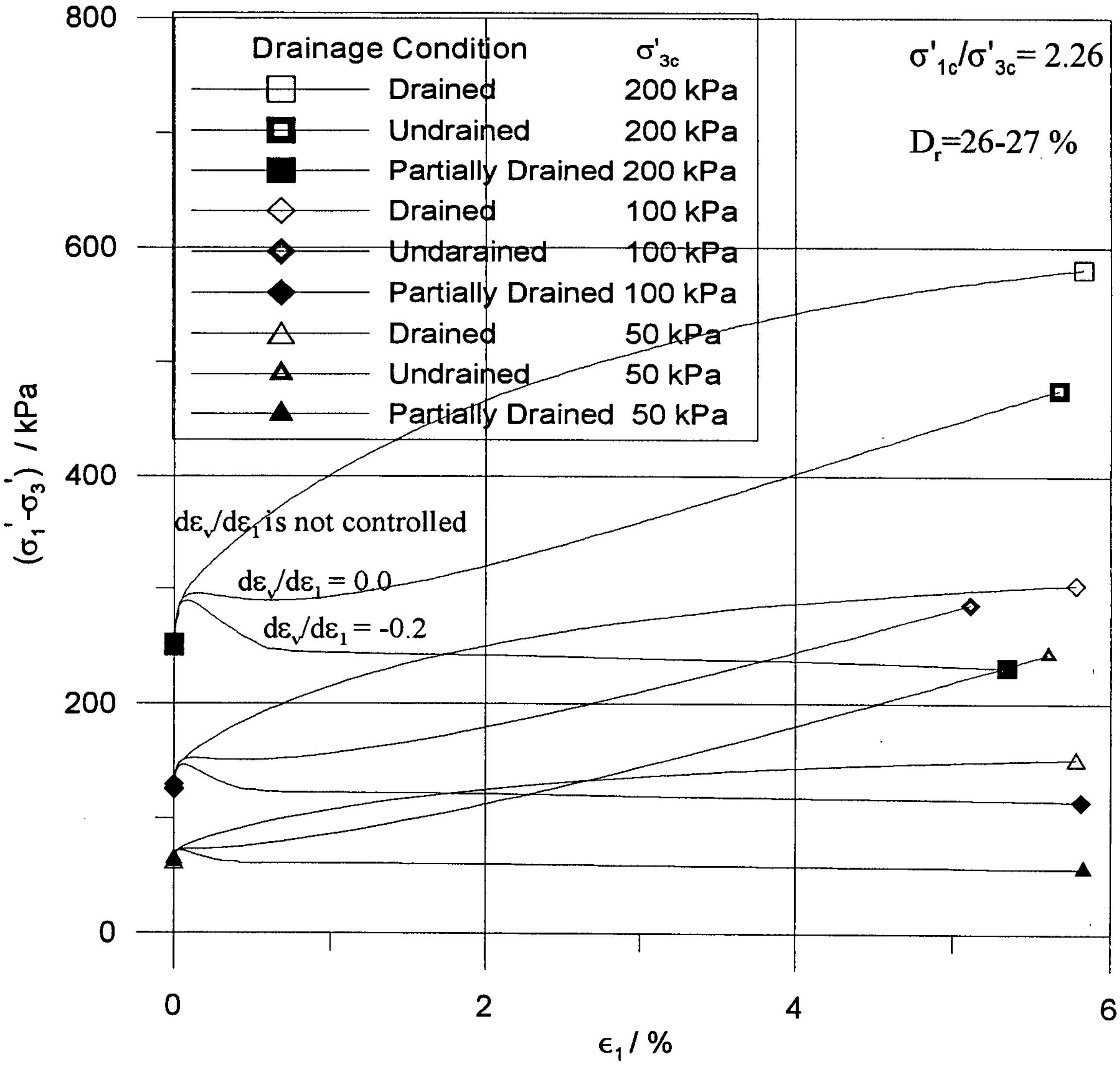


Figure 4.19 Comparison of Drained , Undrained and Partially drained response of FRS at different confining stress

exhibits a marginal strain softening and continuous strain hardening thereafter. The partially drained condition yields a continuously strain softening response following the peak strength (CSR). The curves show that the partially drained response is not bounded by drained and undrained responses. Further, it suggests that certain partially drained conditions, and in particular, expansive volumetric strains, would generate a more damaging response.

Figure 4.19 shows three sets of drained, undrained and partially drained responses observed at different effective confining stress levels (50, 100 and 200 kPa). The behaviour noted previously at σ_{3c}'=100 kPa is also evident at a σ_{3c}' of 50 and 200 kPa. Further, an effect of confining stress on the stress strain response may be observed in this plot too. Although all undrained responses exhibit strain hardening, the specimen with σ_{3c}' = 200 kPa requires more than 6% strain to exceed the drained strength, whereas the specimen with σ_{3c}' = 50 kPa exceeds the drained strength at less than 3% of axial strain. These observations confirm that sand to be more dilative at low confining stress.

4.2 RESULTS AND DISCUSSION OF NORTH SEA SAND

4.2.1 Effect of aging on the stress-strain response

As discussed in Chapter 3, a set of aging tests was performed to investigate the effect of aging on the stress strain response of North Sea sand (NSS) to complement the detailed investigation on Fraser River sand (FRS). A set of partially drained tests were performed to investigate the effect of aging on specimen with identical initial states (stress state and relative density) but with different aging time periods of 1, 100 and 1000 minutes (see Table 3.3).

The aging tests were performed on specimens at their loosest possible states (Dr=22%) under partially drained conditions ($d\epsilon_v/d\epsilon_1$=-0.05). Results of the aging tests are plotted in Figures 4.20 and 4.21. The effect of aging on the stress strain response can be clearly seen in both stress strain space as well as in the effective stress space. As was observed with the Fraser River sand, the North Sea sand also becomes more dilative with a longer aging period. Fig.4.20 shows the PT shear strength increase with aging period. Quantitatively, the PT shear strength increases by 6% over the first 100 mins and by a further 3.5% over the next 900 mins.

Typical creep volumetric strains developed during the aging period are in the order of 0.02 % (see Figure 4.22), which are smaller than observed with FRS. The smaller creep strains is attributed to the finer and more rounded particles of the North Sea sand. Almost all the creep strain occurs over the first 100 mins, compared to 75 % over the first 100 mins for the FRS. Again, the very small creep volumetric strains indicate there will be no significant densification, eliminating this action as a reason for the stronger response of the aged NSS.

Results of the aging tests are also plotted in the effective stress space in Fig.4.21. The effective stress paths indicate the mobilised friction angle at maximum obliquity and phase transformation is constant regardless of the aging period. Considering the development of creep strain and gain in phase transformation shear strength over the first 100 mins, together with difficulties in maintaining constant stresses over longer time periods, and recognizing the benefit

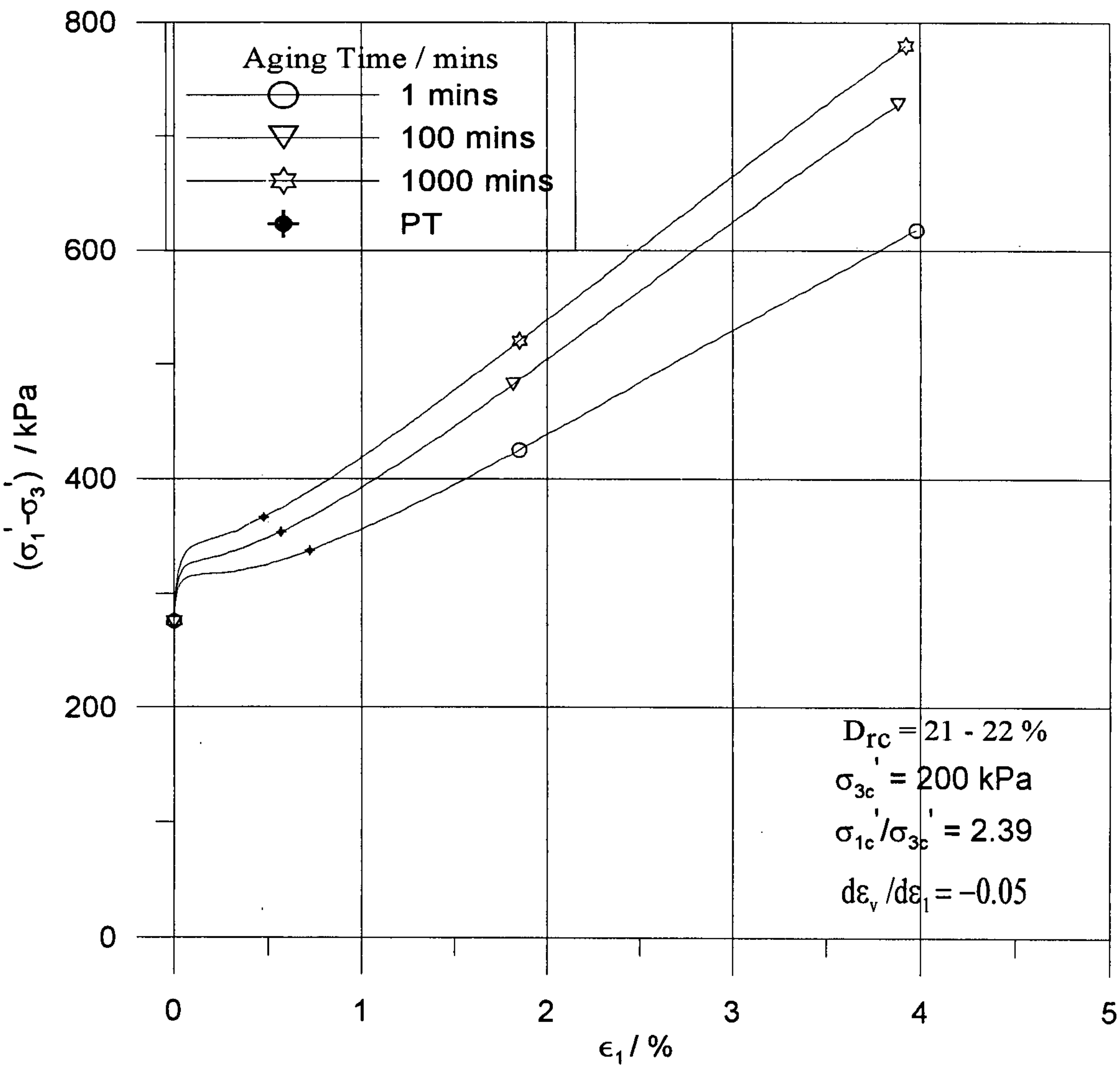


Figure 4.20 Effect of aging on the stress-strain response of North Sea sand

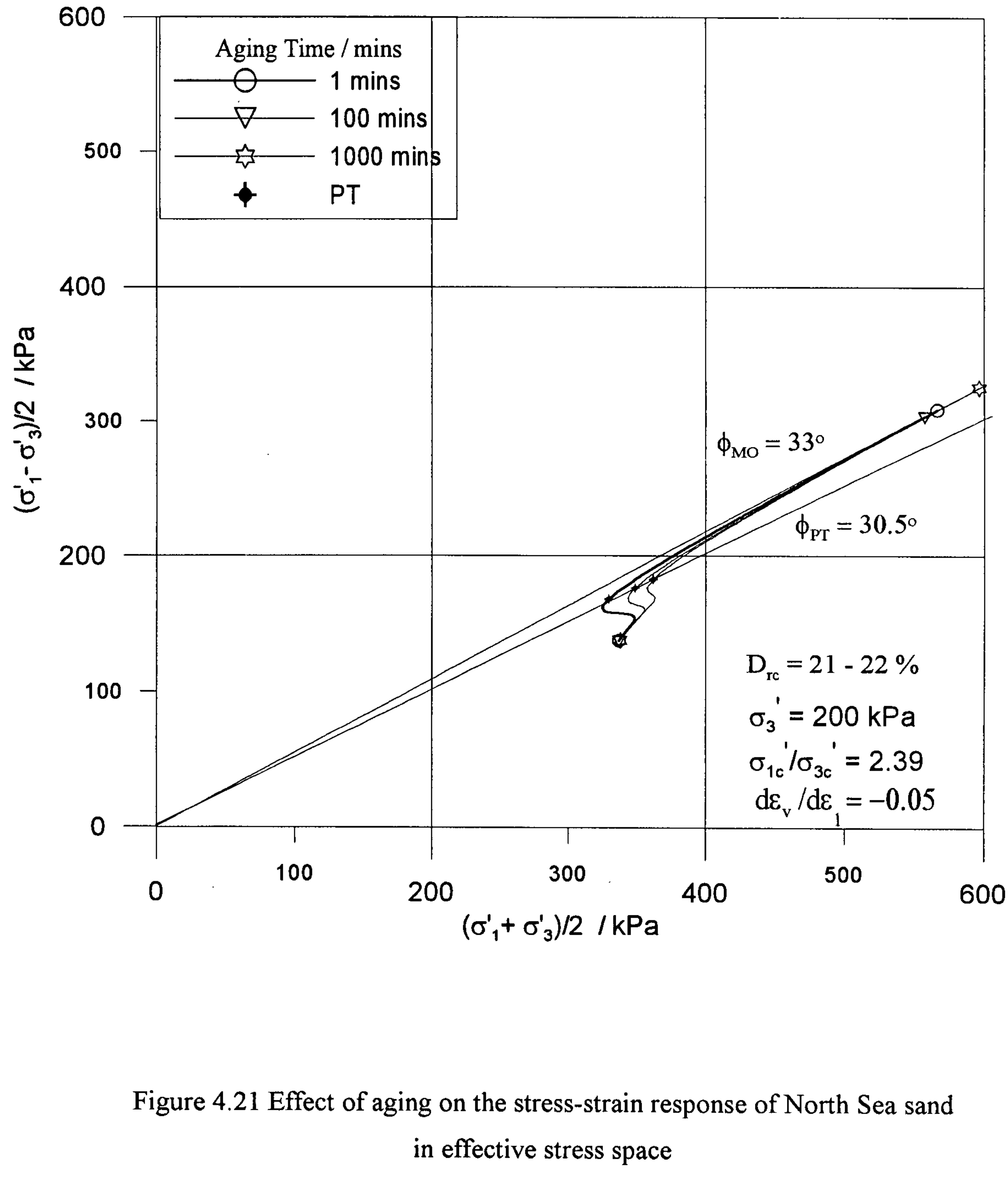


Figure 4.21 Effect of aging on the stress-strain response of North Sea sand in effective stress space

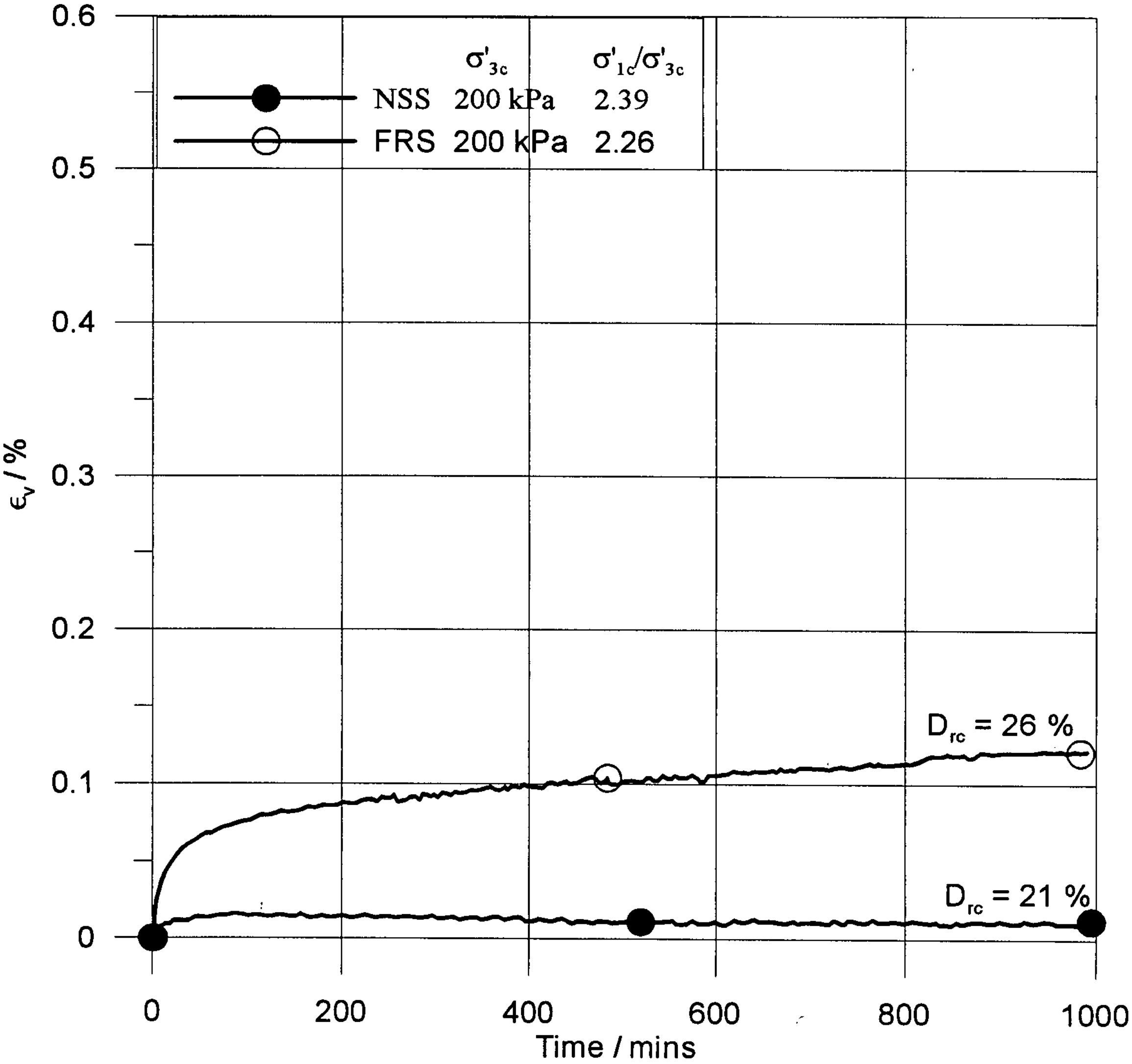


Figure 4.22 Volumetric strain development in North Sea sand during the aging period

of consistency with FRS tests, it was decided to age all NSS specimens for 100 mins prior to shearing.

As it was observed with the Fraser River sand, the North Sea sand also becomes more dilative with longer aging period. The phase transformation shear strength increases by 6% over the first 100 mins compared with a 9% increase for the Fraser River sand. The North Sea sand further gains another 3.5% over the next 900 mins, compared with a 3% gain strength for the Fraser River sand.

4.1.2 Stress strain response of North Sea sand.

A set undrained tests was performed to establish the mobilised friction angle at maximum obliquity and phase transformation, and to investigate the influence of effective confining stress on the stress strain response. Undrained tests were carried out at different effective confining stresses of 50, 100 and 200 kPa with a constant stress ratio of 2.39 (see Table 3.4). The stress strain responses are given in Figure 4.23, together with the corresponding FRS tests. The effect of confining stress can be seen in effective stress space in Figure 4.24. The specimens with 50 kPa confining stress merely had any excess pore water pressure developed whereas the other two specimens with 100 and 200 kPa confining stresses developed more excess pore water pressures. It indicates that the aged NSS specimens too become more contractive with higher effective confining stresses. The mobilised friction angles at maximum obliquity are established and shown in Figure 4.24. As it was observed with aged FRS, the mobilised friction angle of aged NSS at maximum obliquity is constant regardless of effective confining stress.

In an attempt to further extend the characterization of the partially drained response of sands, tests were performed on NSS specimens under constant $d\epsilon_v/d\epsilon_1$ conditions. Two partially drained tests were performed at a $d\epsilon_v/d\epsilon_1$ of -0.05 and –0.2, on specimens with an effective

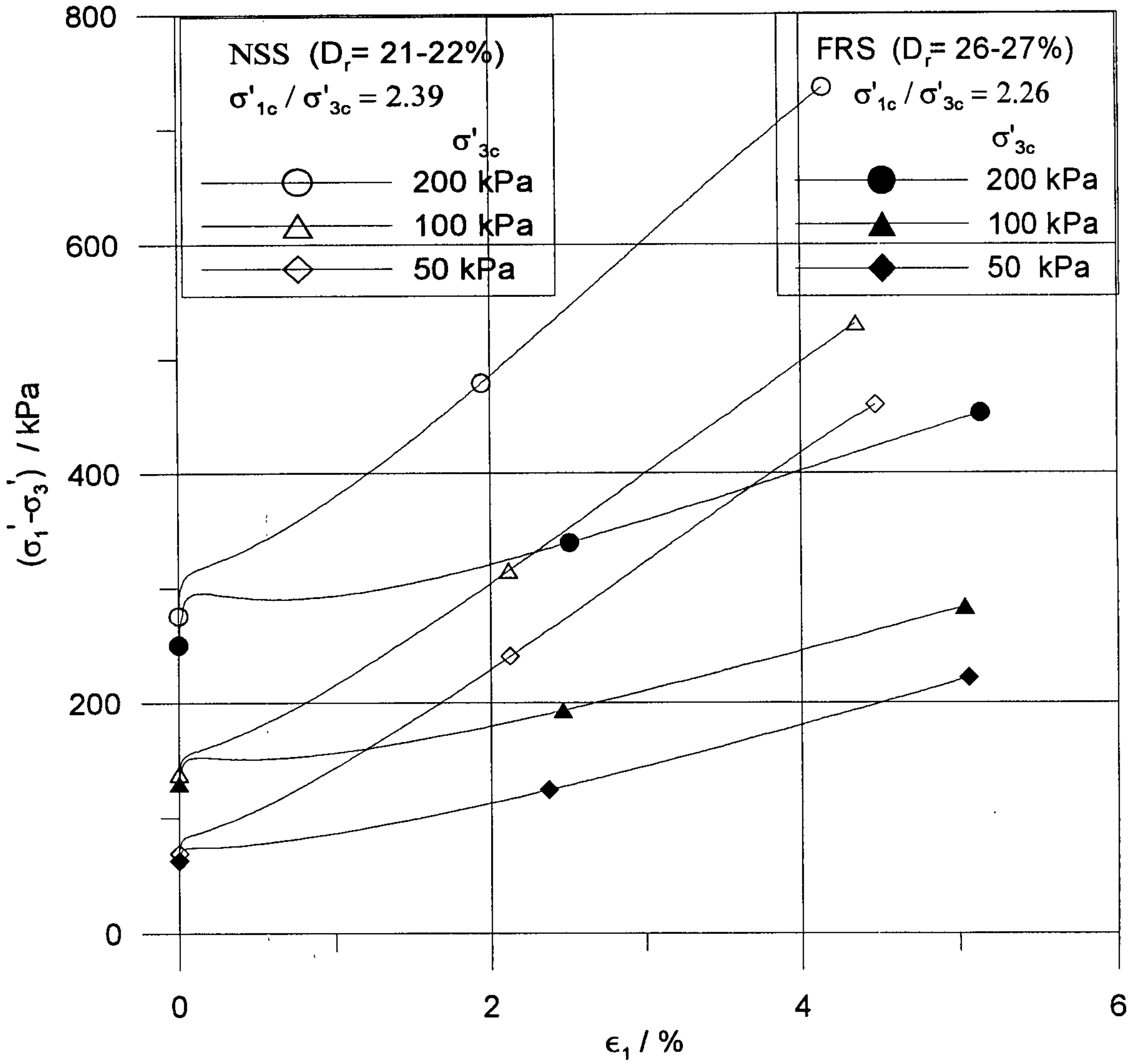


Figure 4.23 Undrained stress strain behavior of North Sea sand

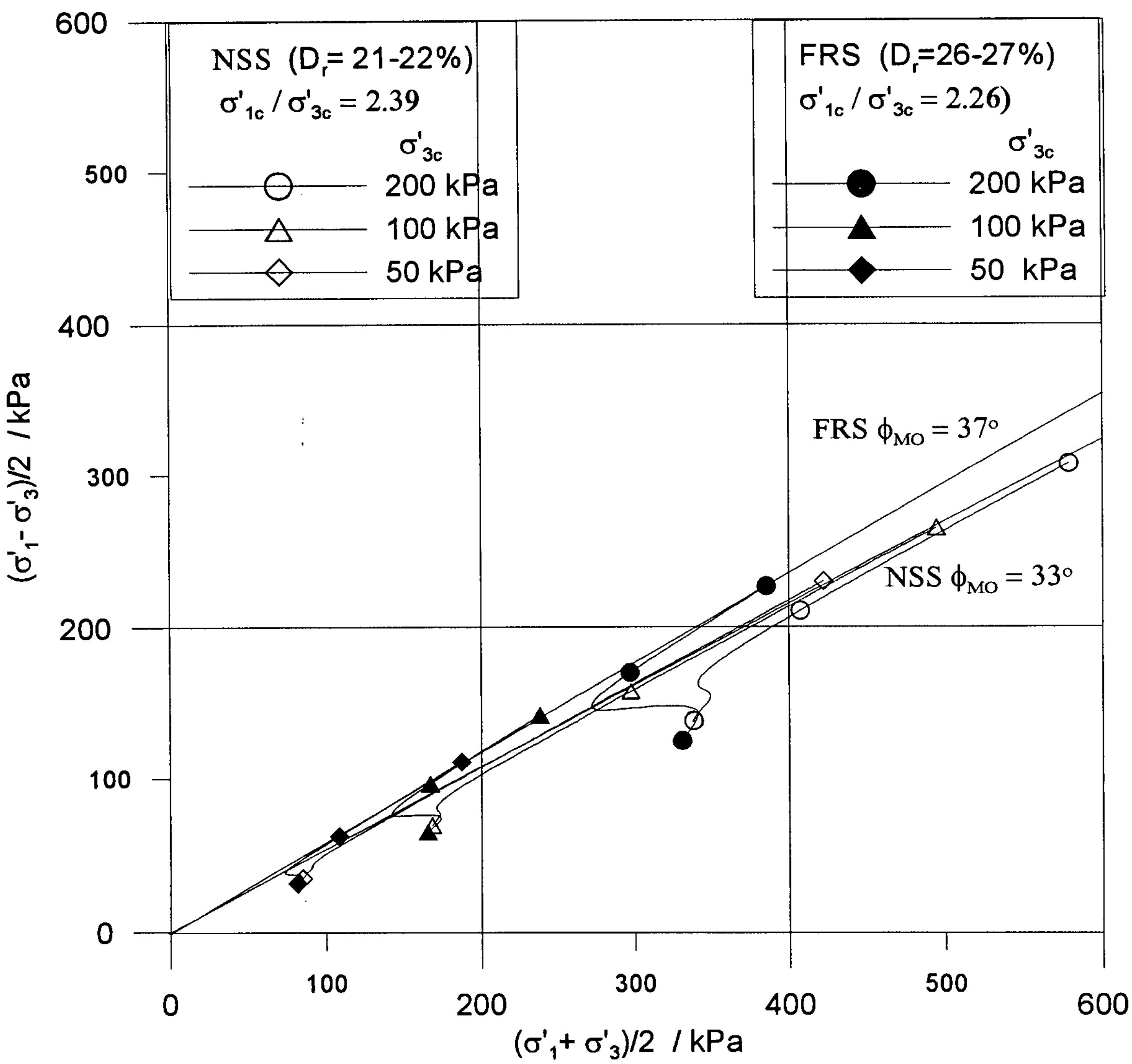


Figure 4.24 Undrained stress strain behavior of North Sea sand in effective stress space

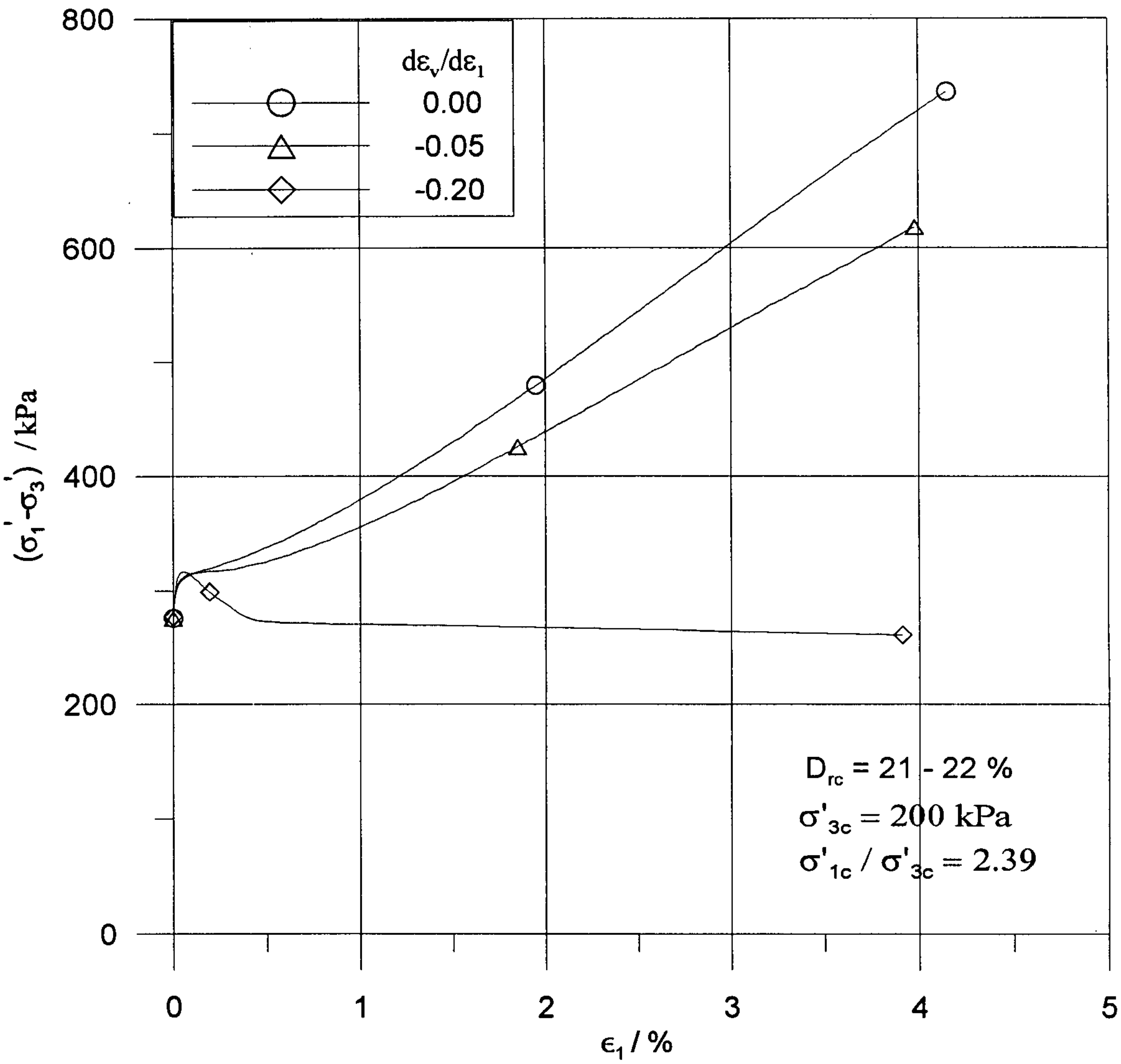


Figure 4.25 Effect of small imposed volumetric strains on the stress strain response of North Sea sand

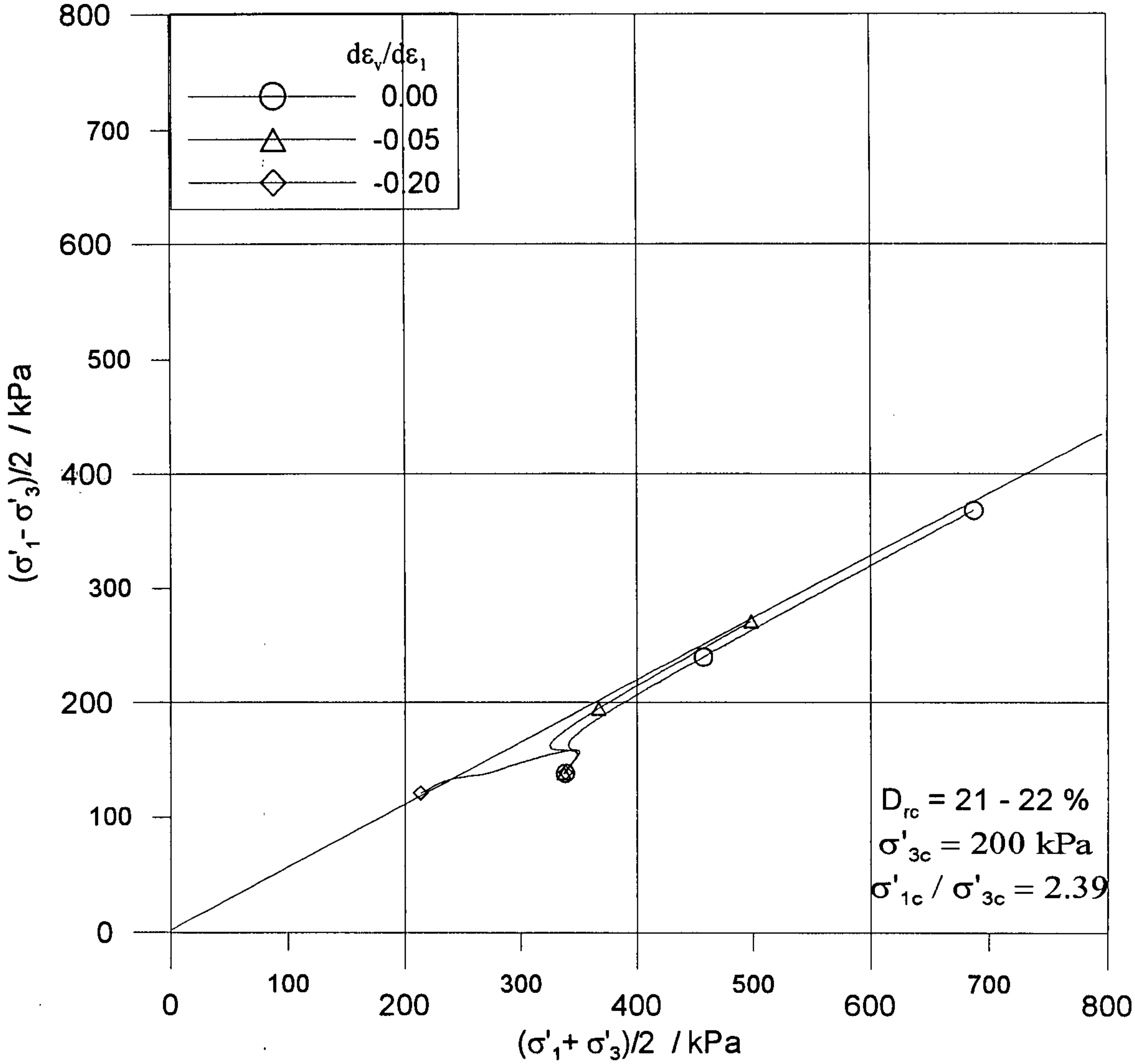


Figure 4.26 Effect of small imposed volumetric strains on the stress strain response of North Sea sand in effective stress space

confining stress of 200 kPa and a stress ratio of 2.39. Results of these tests and the corresponding undrained response are plotted in Figures 4.25 and 4.26. The effect of small imposed expansive volumetric strain (injection) on the stress strain response of NSS can be clearly seen in both plots. The specimen sheared at $d\epsilon_v/d\epsilon_1$ of -0.05 yielded a softer response compared with the undrained response of the identical specimen. But, the specimen with a $d\epsilon_v/d\epsilon_1$ of –0.2 shows a dramatic change in the form of strain softening. Given the magnitude of imposed volumetric strain is very small, the dramatic change in the stress-strain response observed with $d\epsilon_v / d\epsilon_1$ of -0.2 cannot be attributed to a physical loosening of the specimen. All these findings agree with observation and conclusions made with FRS.

The continuously strain softening partially drained response observed under a $d\epsilon_v/d\epsilon_1$ of –0.2 is potentially of significant concern in term of flow failures. Therefore, additional tests were performed to further explore the response at different effective confining stress levels (50, 100 and 200 kPa). Results of these tests are provided in Figures 4.27 and 4.28.

All the specimens are continuously strain softening regardless of the effective confining stress. The effect of confining stress can be seen at small strain, especially in reaching the critical stress ratio. The specimen with 50 kPa confining stress reaches its CSR at 0.03 % axial strain, whereas those with a confining stress of 100 and 200 kPa reach the CSR at 0.04% and 0.05 % axial strain respectively. That is, specimens at lower confining stress attain the CSR at smaller strain magnitudes.

A comparison of the drained, undrained and partially drained response of identical specimens with σ_{3c}' = 200 kPa and $\sigma_{1c}'/\sigma_{3c}'$=2.39 is given in Fig.4.29. As it was observed with FRS (see Figs.4.18 and 4.19), the partially drained conditions (expansive volumetric strains) result in a more damaging and continuously strain softening type of response. Further, it can also be seen that, as for the FRS, the partially drained response is not bounded by the drained and undrained responses.

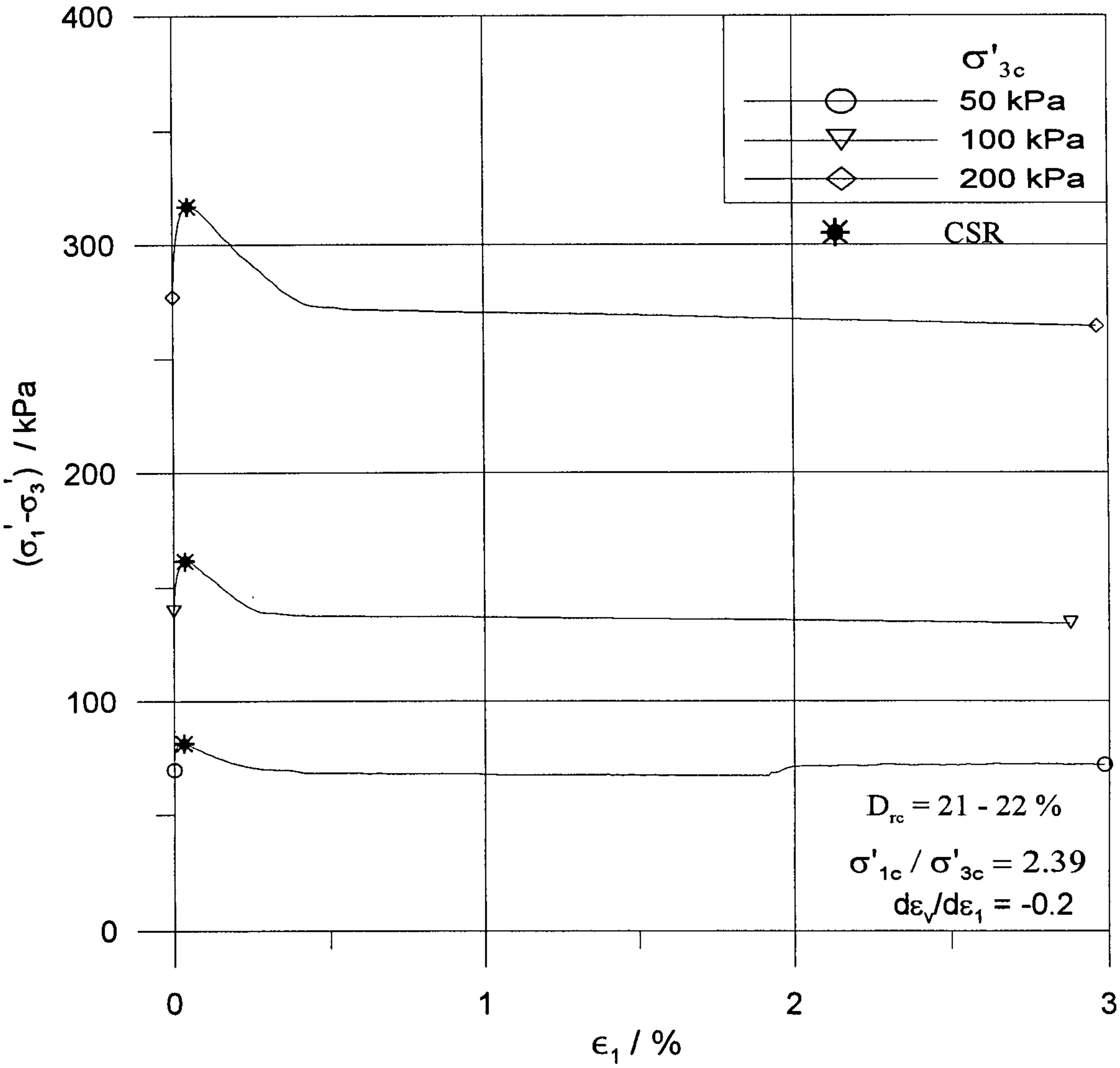


Figure 4.27 Effect of confining stress on the partially drained response of North Sea sand

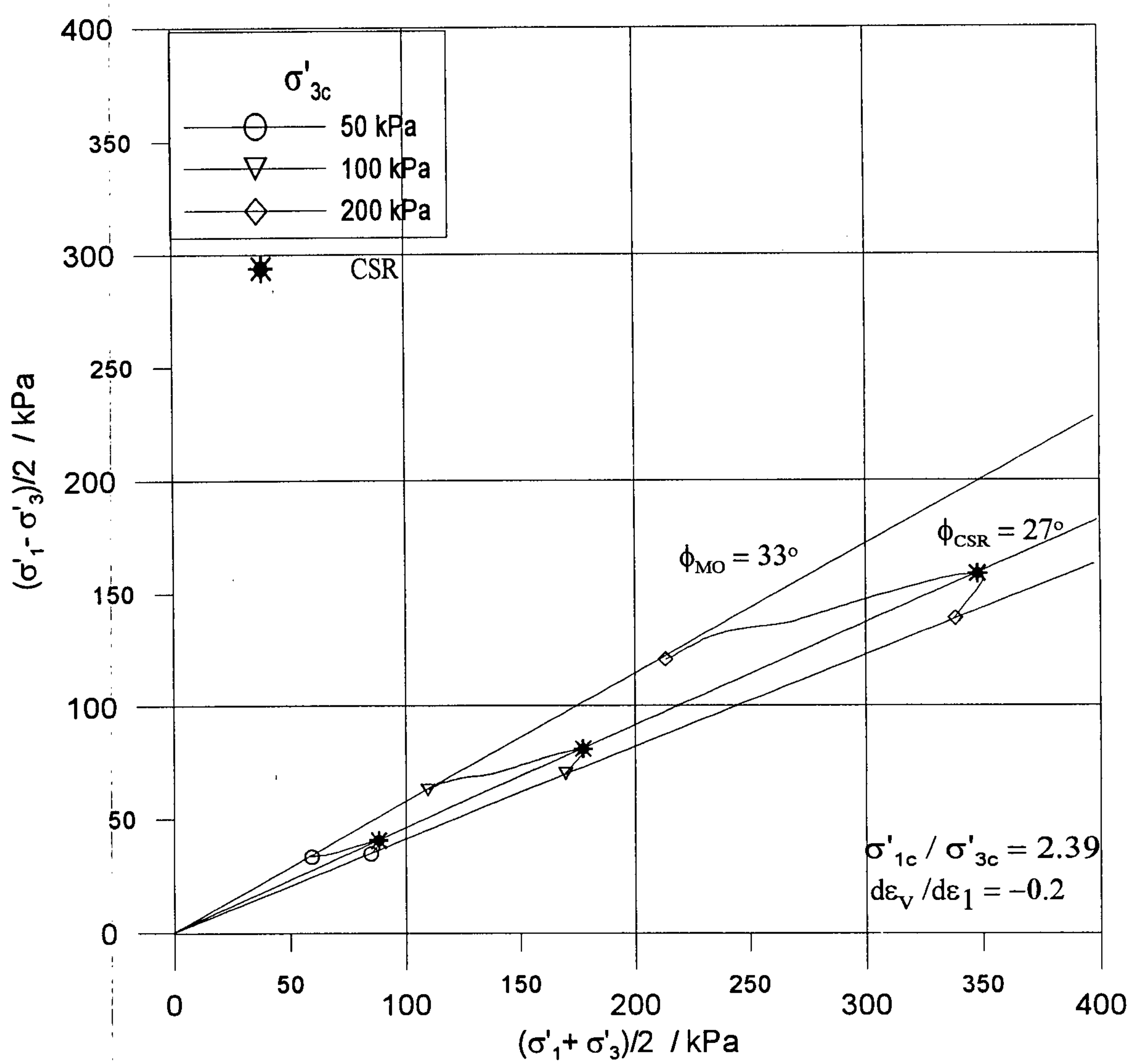


Figure 4.28 Effect of confining stress on the partially drained response of North Sea sand in effective stress space

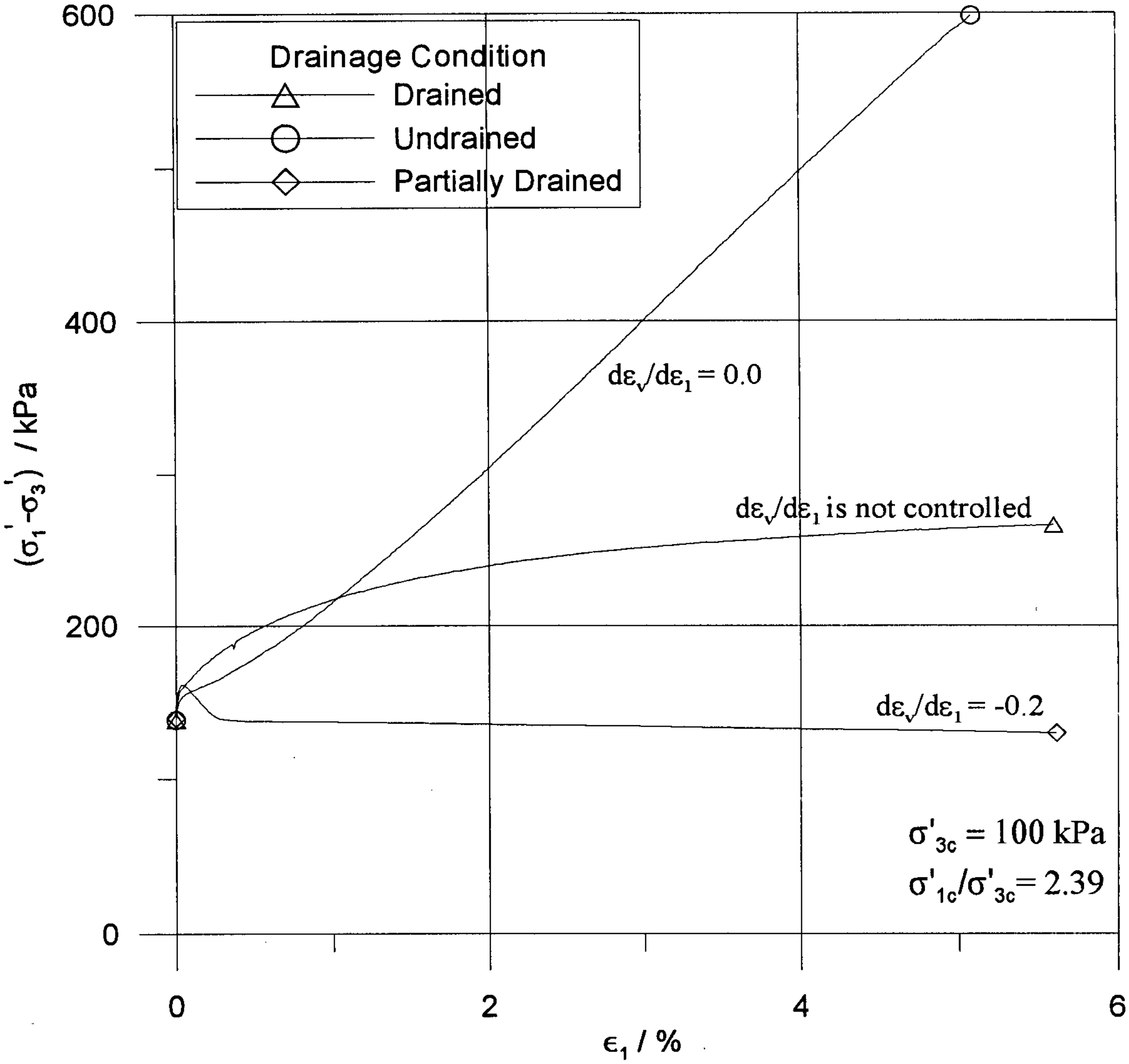


Figure 4.29 Comparison of Drained, Undrained and Partially drained response of NSS

CHAPTER 5

SUMMARY

An experimental investigation has been carried out on Fraser River sand and North Sea sand which differ in mineralogy, grain shape and grain size. The study mainly focused on the partially drained response of sands, considering the effect of aging on the stress strain response and consolidation history. The effect of aging on both sands was first studied to establish a constant aging period for all laboratory specimens. A series of undrained tests was performed to investigate the undrained response of aged sands, and to assess whether observations reported by previous researchers on specimens with no or few minutes of age generally applicable to for aged sands. The partially drained response was investigated in detail. Partially drained tests were performed on identical specimens, but with different degrees of drainage conditions. Knowing the drainage conditions that result in continuous strain softening, additional tests were performed to investigate the influence of effective confining stress and stress ratio on the partially drained response of both sands.

5.1 COMMON FEATURES OF FRASER RIVER SAND AND NORTH SEA SAND

Both Fraser River sand and North Sea sand specimens were reconstituted at their loosest possible densities and consolidated along a constant stress ratio line (see Chapter 3.6.2). During the aging period, both sands developed additional volumetric and axial strains. The magnitude of these creep strains was very small compared with the consolidation strains. Typically the creep volumetric strains were less than approximately 0.1 %. Further, most of the creep strain was developed over the initial 100 mins. Therefore, it is argued that any change in the shear response of sand due to aging cannot be attributed to densification of the test specimens.

Undrained and partially drained shear tests performed on identical specimens, but with a different period of aging, show aging to influence the shear response of a sand. As an indication of the influence, the shear strength at Phase Transformation (PT) and Critical Stress Ratio (CSR) increases with longer period of aging. The PT and CSR shear strengths of Fraser River sand increase by 9% due to 100 minutes of aging. These shear strengths only increase by 3 % for an additional aging period of 900 minutes. The corresponding gain in PT and CSR shear strength for the North Sea sand over the first 100 minutes is about 6 %, whereas it is about 3.5 % over the next 900 minutes.

Given that most of the creep strains occur over the first 100 minutes, that specimens aged to 100 minutes and 1000 minutes demonstrate a more closer response to each other, and that difficulties are encountered in maintaining constant stresses over longer time periods, it was judged that the use of an aging period of 100 mins is reasonable for all the Fraser River sand and North Sea sand specimens.

Undrained tests performed on both sands at their loosest possible states indicate that the general observations reported by previous researchers on the effect of confining stress and stress ratio, using unaged specimens, are true for aged specimens too. It was clearly observed with the Fraser River sand that it becomes more susceptible to strain softening at higher stress ratios. Additionally, a higher confining stress results in more contractive response.

The partially drained response may not be bounded by the drained and undrained responses to shearing. Instead, there is an infinite number of candidate partially drained responses, some of which can fall between drained and undrained curves and some of which do not. Further, certain partially drained conditions, particularly the case of expansive volumetric strains, can result in a more damaging response compared to the undrained and drained conditions.

The partially drained tests performed on both sands demonstrate that small imposed volumetric strains significantly change the stress strain response of sands. These constant strain paths impose only very small volumetric strains on the specimens. Therefore, the significant change in the shear response can not be attributed to a physical densification (or loosening) of the test specimen. The imposition of expansive volumetric strains on a sand specimen yields a more contractive response in loose specimens or, conversely a less dilative response in dense specimens. Imposed contractive volumetric strains result in the opposite way, i.e., make the specimens more strain hardening. Further, it is also evident that the degree of drainage influences the mobilized friction angles at maximum obliquity (MO), phase transformation or the point of maximum pore water pressure (PT), and critical stress ratio (CSR).

Both sands were found to be continuously strain softening at $d\epsilon_v/d\epsilon_1 = -0.2$ irrespective of confining stress and stress ratio. The mobilised friction angle at maximum obliquity and at the critical stress ratio is constant irrespective of confining stress and stress ratio, under constant $d\epsilon_v/d\epsilon_1$ conditions. Still, the influence of confining stress and stress ratio can be seen at small strain levels. Specimens at lower confining stresses behave in a more brittle manner, and start to strain soften at smaller strain levels. Specimens at higher stress ratios start to strain soften at relatively small strain levels, whereas those at lower stress ratios would reach their CSR at larger strain levels.

5.2 SIGNIFICANTLY DIFFERENT FEATURES OF FRASER RIVER SAND AND NORTH SEA SAND

The sands used in this program of testing have a distinctly different mean grain size. The Fraser River sand (FRS) has a D_{50} of 0.35 mm, whereas that for North Sea sand (NSS) is 0.183 mm. However the coefficients of uniformity is very similar (1.7 and 2.0, respectively). The North Sea sand particles are more rounded than the Fraser River sand particles (see Figure 3.3).

In the consolidation phase, the Fraser River sand developed more axial and associated volumetric strain compared to the North Sea sand. More specifically, the Fraser River sand developed about 2% of consolidation volumetric strains whereas North Sea sand developed only 0.5% for the range between 50 and 200 kPa effective confining stresses. This is attributed to the influence of grain size and angularity.

In the aging phase, at the same confining stress and k_0 stress ratio, FRS developed creep volumetric strains of about 0.1% whereas the NSS developed even smaller creep volumetric strains of 0.02 %. Further, almost all the creep strains of the NSS were developed within the first 100 minutes, compared with 75% for the FRS. The relatively small magnitude of creep strains, and their faster development in the NSS, is again attributed to the smaller and more rounded grain size of this sand.

The undrained shear response of both sands at their loosest possible densities indicates that the NSS, which is finer, is more dilative compared with the FRS. This more dilative nature of the NSS was also observed under the partially drained condition of $d\epsilon_v/d\epsilon_1 = -0.05$. An imposed volumetric strain of $d\epsilon_v/d\epsilon_1 = -0.05$ on the FRS specimens transformed the otherwise strain hardening response in undrained shear to a strain softening response; the NSS specimens did not strain soften at the same injection rate. The moblized friction angles (ϕ_{MO}, ϕ_{CSR} and ϕ_{PT}) of Fraser River sand are found to be slightly larger than that of North Sea sand as expected.

CHAPTER 6

CONCLUSIONS

A comprehensive laboratory study has been carried out on two different sands to investigate the partially drained response of sands. The program was also addressed the effect of aging on the shear response of sands.

Results suggest that the effect of aging on the stress strain response of sand is considerable, and should be taken into account when predicting the likely behaviour of sands in nature. Aged specimens demonstrate a stronger and stiffer stress strain response compared with unaged (or fresh) specimens. Therefore, predictions based on the response of an unaged specimen should tend to be conservative and underestimate the available soil strength in nature.

In most cases the laboratory tests were performed on reconstituted specimens. This study suggests that a consistent period of aging has to be adopted for such reconstituted specimens. For the two sands tested in the program of study, a period of 100 minutes of aging was found to reasonable.

The undrained response of aged and unaged sand specimens demonstrates the same general behaviour. Essentially, the qualitative effect of confining stress and stress ratio on the stress strain response is similar.

It is generally believed that the drained and undrained responses to shear represents two extreme conditions and any partially drained response will fall within these bounds. However, the study clearly showed that the drained and undrained conditions are just two cases of drainage conditions. There are an infinite number of possible drainage conditions which, in turn, yield a large spectrum of responses.

Small imposed volumetric strains significantly change the response of aged saturated sands. Expansive volumetric strains render the material susceptible to strain softening, whereas contractive volumetric strains increase the dilative response of sands. Beyond a certain rate of expansive volumetric strains, a sand will exhibit a continuous strain softening response. Differences may seen in a continuously strain softening responses at small strain levels. The sand reaches its critical stress ratio, and starts to strain soften, at different strain levels depending on its physical properties, confining stress and stress ratio. Specimens at higher confining stress and lower stress ratio will start to strain soften at comparatively larger strains compared with specimens at lower confining stress and higher stress ratio.

All these findings suggests that the partially drained conditions and the influence of aging must be taken into account when predicting the response of a natural soil deposit. In effect, the soil structure under consideration should be analyzed to account for pore pressure redistribution and any resulting volumetric strains. Attention must be given to soil elements which experience expansive volumetric strains. Elements with low confining stress and a higher stress ratio must be treated carefully.

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