FABRIC, INITIAL STATE AND STRESS PATH EFFECTS ON LIQUEFACTION SUSCEPTIBILITY OF SANDS

by

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ABSTRACT

An experimental study aimed at improving the understanding of the mechanics of liquefaction is presented. Two aspects of the undrained behaviour; (i) the influence of initial state, characterized by void ratio, principal stresses and their directions, and fabric under a given undrained stress path, and (ii) the effect of undrained stress path at a given initial state, were studied in a systematic manner. Triaxial, simple shear and hollow cylinder torsional shear devices were used in the experimentation to enforce a range of stress paths that simulate the loading insitu. The effect of membrane penetration was duly addressed in order to confidently measure the truly undrained behaviour of sands.

The influence of soil fabric was determined using triaxial and simple shear tests on specimens reconstituted by different techniques. In addition to specimens reconstituted in the laboratory, undisturbed, in-situ frozen sands from four different sites were tested to assess the relevance of the behaviour of reconstituted specimens to the sand in-situ. It is shown that in-situ sands and those water pluviated in the laboratory are inherently anisotropic, and their undrained behaviour is direction dependent. At a given initial state, they often strain soften to an essential steady state in triaxial extension, but invariably strain harden in compression. Moist tamped sands, on the other hand, strain soften both in compression and extension. The domain of states in void ratio-effective stress space accessible to alluvial in-situ sands is shown to be similar to those that ensue on pluviation. The behaviour of water pluviated sands is similar to that of alluvial in-situ sands, at a given initial stress state and undrained stress path, both under static and cyclic loading

conditions – an indication that they both possess very similar fabric. This opens up the possibility of using water pluviated sand specimens to characterize the behaviour of in-situ sands, an attractive economical alternative.

It is shown that the initial effective stress state plays a dominant role on the subsequent undrained behaviour at a given void ratio. Hollow cylinder torsional shear tests on water pluviated fabric were used to assess the influence of the initial axisymmetric and non-axisymmetric stress states on undrained shear. The behaviour due to an increase in deviator stress alone, and that due to simultaneous changes in deviator stress and the direction of principal stresses is investigated. A larger inclination of major principal stress to vertical results in a softer behaviour at all levels of static shear. An increase in static shear at a given inclination of the major principal stress promotes more strain softening response. Even a small undrained perturbation may trigger flow failure in a sand that is otherwise stable if drained, in the event the initial stress state is highly anisotropic together with larger inclination of major principal stress to the vertical. It is demonstrated that flow failure may be triggered by a mere rotation of principal stress directions only. The sense of principal stress rotation with respect to the initial inclination of the major principal stress also plays a prominent role on the undrained behaviour, and hence on strain development. It is shown that the strain induced anisotropy evolves at an accelerated rate due to rotation of major principal stress direction, compared to an increase in effective stress ratio alone at fixed direction of principal stresses.

It is shown that both steady state and quasi steady state can be treated within the same framework. The friction angle mobilised at these states is independent of fabric, void ratio, effective confining and static shear stress levels, direction of principal stresses, and undrained stress path. This angle appears to be a unique material property. Contrary to the commonly held notion, the minimum undrained strength is dependent on the stress path, initial confining and static shear stress levels, fabric and the directions of principal stresses, in addition to void ratio. This implies that the steady/quasi-steady state is unique in the effective stress space, but not so in the void ratio – stress space. However, the minimum undrained strength normalised by the major principal stress appears to be dependent only on void ratio and the direction of major principal stress (i.e loading mode). The friction angle mobilised at the trigger of strain softening is also dependent on the initial stress state and the loading mode.

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List of Symbols

A_{IM}	Surface area of the specimen covered by the inner membrane
A _{om}	Surface area of the specimen covered by the outer membrane
b	Intermediate principal stress parameter
b _c	Intermediate principal stress parameter at the end of consolidation
D ₅₀	Mean grain size
D _r	Relative density
D _{rc}	Relative density at the end of consolidation
e _c	Void ratio at the end of consolidation
e _{max}	Maximum index void ratio (ASTM D4254-83)
e _{min}	Minimum index void ratio (ASTM D4253-83)
F	Sleeve friction ratio in cone penetration test
I _B	Brittleness Index
K _c	Effective principal stress ratio at the end of consolidation (σ'_1/σ'_3)
\mathbf{S}_{min}	Minimum undrained strength
\mathbf{S}_{peak}	Peak undrained strength
$\mathbf{S}_{\mathrm{SS/QSS}}$	Shear strength at steady/quasi-steady state
S _{SS/QSS/PT}	Shear strength at steady/quasi-steady or phase transformation state
CSR	Critical stress ratio at the instant of peak deviator stress
РТ	Phase transformation
QSS	Quasi steady state
R _i	Internal radius of the hollow cylindrical specimen
R _e	External radius of the hollow cylindrical specimen
\mathbf{q}_{c1}	Cone tip resistance corrected to 100 kPa overburden stress
SS	Steady state
v	Volume of the specimen

V _{IC}	Inner Cavity volume of the hollow cylindrical specimen
V_{s1}	Shear wave velocity corrected to 100 kPa overburden stress
α_{σ}	Inclination of major principal stress to the vertical
$\alpha_{\sigma c}$	Inclination of major principal stress to the vertical at the end of consolidation
χ	Ratio of external to internal radii of the hollow cylindrical specimen
δV_{r}	Recorded volume change of the specimen
δV_{ir}	Recorded volume change of the inner cavity
δV_{IM}	Volume change induced by penetration of inner membrane
$\delta V_{\text{ом}}$	Volume change induced by penetration of outer membrane
$\boldsymbol{\epsilon}_1$.	Major principal strain
ε ₂	Intermediate principal strain
ε ₃	Minor principal strain
€ _a	Axial strain in triaxial
ε _m	Unit membrane penetration (membrane induced volume change per 10-fold
	change in effective confining stress)
Ē m	Normalised Unit membrane penetration (ε_m per unit area of the membrane)
ε _r	Normal strain in the radial direction
ε _z	Normal strain in the vertical direction
ϵ_{θ}	Normal strain in the tangential direction
γ_{max}	Maximum shear strain
$\gamma_{z\theta}$	Shear strain on the horizontal plane in the tangential direction
$\phi_{\rm CSR}$	Friction angle mobilised at the trigger of strain softening
$\phi_{SS/QSS/PT}$	Friction angle mobilised at SS/QSS/PT
σ_1	Major principal stress
σ_2	Intermediate principal stress
σ_3	Minor principal stress
σ_{d}	Deviator stress

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$\sigma_{\text{d,cy}}$	Cyclic deviator stress
σ_{dn}	Deviator stress normalized by effective mean normal stress
σ_{m}	Total mean normal stress
σ_{mc}	Total mean normal stress at the end of consolidation
σ_{r}	Normal stress in the radial direction
σ_{z}	Normal stress in the vertical direction
σ_{θ}	Normal stress in the tangential direction
σ'_{mc}	Effective mean normal stress at the end of consolidation
σ'1	Effective major principal stress
σ'2	Effective intermediate principal stress
σ'3	Effective minor principal stress
σ'_{h}	Horizontal effective stress in triaxial
σ'_{v}	Vertical effective stress in triaxial/simple shear
τ_{cy}	Cyclic shear stress in simple shear
τ_{h}	Horizontal shear stress in simple shear
$\Delta \alpha_{\sigma}$	Change in the direction of major principal stress direction
$\Delta\sigma_{d}$	Change in deviator stress
$\Delta\sigma_{dn}$	Change in normalised deviator stress
Δu	Excess pore pressure

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Chapter 1

Introduction

The term liquefaction embraces all phenomena involving excessive deformation in saturated granular soils. In short duration field events such as an earthquake or a shock, soils get loaded virtually undrained. This causes a reduction in their strength and stiffness due to the development of excess pore pressure. In the field, damage due to liquefaction may express itself as flow slides in sloping ground. Some examples of such flow slides are Ports of Valdez, Alaska, Nice, Lower San Fernando dam, and Chilean tailings dams (Seed at al. 1975, Jeyapalan et al. 1983, Seed et al. 1988, Ishihara et al. 1990, Bardet and Kapuskar 1993, Bardet and Davis 1996, Boulanger et al. 1997, Holzer et al. 1999). Under level ground this damage manifests as cracking or lateral spreading of overlying embankments, and tilting or even collapse of superstructures. Several examples of such type of damage were seen in the earthquakes of Nigaata, Alaska, Mexico City, Northridge, Kobe (Whitman 1987, Barends et al. 1996, Stark and Contreras 1998, Hamada and Wakamatsu 1998) and the recent Turkey and Taiwan events. Deformations due to liquefaction are unidirectional in sloping ground, but they are generally of cyclic nature in level ground.

As in other engineering materials, a fundamental understanding of the physics and mechanics of sand liquefaction has been derived from laboratory element tests. The response of sand under

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undrained triaxial compression loading has been the mainstay of our current knowledge in this field. The mechanical behaviour of soils, however, is very complex. Apart from being nonlinear and inelastic, it depends on the initial state of the soil comprised of density, magnitude and directions of effective principal stresses and any prior stress/strain history, as well as the stress path during undrained shear. For a given initial density and stress state, it further depends on the soil fabric. The objective of laboratory element testing therefore is to assess the influence of both initial state variables and fabric together with that of the stress path during undrained shear.

Most of the databases on element tests consist of triaxial compression tests on reconstituted specimens. In this test, the initial stress state is axisymmetric with principal stress directions aligned in the vertical deposition, and horizontal bedding plane directions. During shear, the directions of the principal stresses and the state of stress (axisymmetric) stay unaltered. Clearly, the behaviour under these conditions only is not sufficient to describe the undrained response under other generalised initial states and variety of stress paths experienced by the soil element in the field. This is because of the complexity of the mechanical response of soils, which is further influenced by inherent anisotropy in natural and hydraulic fill sands (Arthur and Menzies 1972, Vaid and Chern 1985, Vaid et al. 1990a, Riemer and Seed 1997). There is little likelihood of success in developing a general constitutive model that captures all aspects of soil behaviour. Therefore, the field behaviour is commonly assessed using the stress path approach (Lambe 1973). This approach aims at evaluating the behaviour of soils under the initial states and the range of loading paths that duplicate those anticipated in the given field problem.

Fundamental studies on liquefaction using other testing devices, and hence stress paths, are few. Each of these devices is intended to simulate specific field loading conditions pertinent to

a given problem. The simple shear test, for example, has been used to simulate the behaviour of a sand element under earthquake shaking (Roscoe 1970, Vaid and Finn 1979, Lacasse and Vucetic 1981). This test represents loading of a sand element in horizontal plane strain. It, however, imposes uncontrolled principal stress rotation in the vertical plane. Studies in the hollow cylinder torsional shear device do allow control of not only the magnitude of the three principal stresses, but also their directions in one plane. It thus permits assessment of the response, both under generalised initial state and different stress paths. It is also possible to isolate in a fundamental manner, the effect of each initial state variable or stress path parameter on sand behaviour. Generalised initial stress states, and undrained paths are the norm, rather than exceptions in real field problems.

Three facets of the undrained behaviour of sand have been explored in this thesis. The influence of membrane compliance on the measured undrained behaviour in the laboratory is assessed first. This effect, if not considered, does not yield a measure of the intended true undrained behaviour of sands, but rather the response under a partially drained state. This aspect of research is, therefore, directed towards confident measurement of undrained response. Errors endemic to the penetration of flexible rubber membranes used to enclose the test specimens are reviewed, and a new, more confident method of evaluating membrane penetration is developed for compensation of their compliance effects on undrained tests. Compensation, that depends on soil particle size, if needed, is automatically made by the control system.

The second facet of the investigation is intended to determine the influence of soil fabric, under an otherwise identical initial state and subsequent undrained stress path. Comparative tests are made on undisturbed samples of sand retrieved by in-situ ground freezing, and those reconstituted

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in the laboratory by different techniques. Although considerable evidence exists to indicate that different reconstitution techniques yield different fabrics, and that sand fabric plays a dominant role on the undrained behaviour, little consideration is accorded to it in practice. Further, the extent to which the undisturbed sand behaviour differs from that of its reconstituted counterpart is also not known. This is of crucial importance in applying the results of tests on reconstituted specimens to the undisturbed sands in-situ.

The third facet of the investigation concerns the assessment of the influence of generalised initial stress states and stress paths on undrained behaviour. Triaxial, simple shear and hollow cylinder torsional shear devices are used. The influence of initial stress state variables on undrained behaviour under stress paths simulated by these devices is explored, and compared. The influence of anisotropy of sands on its undrained response is also studied in the results of these tests.

Chapter 2 is devoted to an understanding of the liquefaction phenomena as revealed in fundamental laboratory studies. Results of previous studies, mostly under triaxial compression, and the few under extension, simple shear and generalised stress states are critically reviewed in an attempt to justify the studies undertaken in this thesis. Chapter 3 documents the various testing devices used in this investigation, together with details of the experimental procedures and techniques. Chapter 4 deals with the aspect of membrane compliance. A new, specimen specific method to evaluate the magnitude of membrane penetration is presented, and the influence of membrane compliance on undrained behaviour of sands is highlighted. The effect of fabric on the undrained behaviour of sands is demonstrated in Chapter 5, using reconstituted and undisturbed (in-situ frozen) sand specimens. A comprehensive treatment of the influence of initial effective

stress state and the subsequent undrained stress path on the behaviour of sands is presented in Chapter 6. Specimens reconstituted by water pluviation only are used in these studies so that the fabric is held constant. Chapter 7 finally presents a summary and conclusions arrived at under each of the three facets of the investigations.

Liquefaction of Sands

2.1 Introduction

The current state of understanding of the undrained behaviour and the phenomenon of liquefaction of sands is presented in this chapter. Both static and cyclic liquefaction are considered. As pointed out in the introduction, most of our understanding of undrained behaviour of sands has been derived from fundamental studies of element tests in the laboratory under controlled conditions.

A knowledge of the undrained response of sands is critically important in assessing their susceptibility to liquefaction, and in estimating the liquefaction induced and post liquefaction displacements. Under static loading the susceptibility to liquefaction could trigger flow failure. Under cyclic loading, on the other hand, the susceptibility is for the development of large strains due to a progressive reduction in shear stiffness. The potential for flow failure exists under cyclic loading if initial static shear is present. Post cyclic static behaviour is relevant to the estimation of displacements or even a delayed flow failure that could ensue after the cessation of the earthquake. The former occurs on account of the presence of static shear stresses and the latter due to the redistribution of earthquake induced pore pressures. In such cases, the earthquake

simply initiates a trigger mechanism, and the trigger of the flow failure occurs following the cessation of the seismic event.

The undrained behaviour is first discussed as observed in conventional triaxial compression tests on reconstituted specimens. Such data has been, and still is the mainstay of our understanding of the liquefaction phenomena. The stress conditions simulated by the triaxial test, however, represent loading along a specific stress path under axisymmetric conditions. The soil elements in the field get loaded along many different stress paths and stress systems. A knowledge of the undrained behaviour along such paths and stress systems is therefore of utmost importance to assess the behaviour of sands in the field.

Evidence in support of the inherently anisotropic nature of sands is presented based on data from the literature. The stress path dependent undrained behaviour of sands is a consequence of this inherent anisotropy. This facet of the undrained behaviour has largely been ignored, and only a few studies have attempted to investigate the stress path effects. Results from these studies highlighting the stress path dependency on both static and cyclic response, have been summarised and their implications in practice pointed out.

Soil fabric is an additional state variable that governs its undrained behaviour at identical initial stress states and subsequent undrained paths. A limited number of studies provide evidence in support of this using comparative data on sands with different initial fabrics. The need to duplicate in-situ fabric is then pointed out if laboratory tests on reconstituted sands are to be substituted for the undisturbed sands. The research needs identified from the review of the current state of understanding are detailed at the end of this chapter.

2.2 Undrained behaviour of saturated sands

A progressive reduction in shear stiffness due to the development of excess pore pressure is often the cause of large deformations in the undrained state. Therefore only the behaviour of saturated sands is of concern under undrained shear, and as noted earlier, large deformations can develop either as a result of static or cyclic loading. Terzaghi and Peck (1948) originally used the term "*spontaneous liquefaction*" to explain the sudden deformation of loose sands, in a manner similar to that of a viscous liquid. The term liquefaction has been defined based either on the amount of excess pore pressure, the amount of strain developed or based on the mechanism that is responsible for the deformation. The widely accepted NRC (1985) criterion based on a specified strain level (2.5% single amplitude axial strain or equivalent) without regard to the mechanism responsible for the deformations is adopted in this thesis as the definition of liquefaction.

2.2.1 Static undrained behaviour in triaxial compression

The static undrained behaviour of saturated sands has mostly been studied under triaxial compression loading (e.g. Castro 1969, Castro et al. 1982, Chern 1985, Thomas 1992, Ishihara 1993, and Riemer and Seed 1997), using test specimens reconstituted by water pluviation or moist tamping. Figure 2.1 shows typical triaxial compression undrained response of a saturated sand. The type of response is mainly dependent on its initial state conveniently represented by




Figure 2.1 : Typical undrained response of sands under static loading.

relative density and the associated confining and static shear stress levels prior to undrained shear. At a given initial state the undrained behaviour may change from a type 1 to a type 3 with decreasing void ratio (i.e. with increasing relative density). At loose void ratios, (or at high confining stresses) a sand may develop flow failure type of deformation (type 1) due to strain softening, after mobilizing a peak strength at small strains (typically less than 1%). Type 2 response signifies strain hardening following an initial strain softening. Sands exhibiting strain softening response (types 1 and 2) are often termed contractive. Those that respond in a strain hardening manner (type 3) are termed dilative.

Castro (1969), Casagrande (1975) and Seed (1979) called type 1 strain softening response liquefaction, whilst Chern (1985) termed it as true liquefaction. The characteristic feature of this type of response is continued unlimited unidirectional deformation at constant stresses and void ratio. This is called the steady state (SS) or flow deformation (Castro et al. 1982; Vaid and Chern 1985). The constant shear stress at which the sand undergoes unlimited unidirectional strain is called the undrained steady state strength or residual strength.

The type 2 strain softening response, which occurs over a limited strain range, transforms into strain hardening behaviour with further deformation. This type of response has been termed limited liquefaction (Castro 1969). There exists a small range of strain over which the sand may be considered to deform essentially at constant stresses and void ratio, much in the same manner as during steady state of deformation. The instant of the minimum undrained strength may be viewed as the occurrence of a transient steady state as in type 1 response. This state has been labelled as quasi steady state (QSS) by Ishihara et al. (1975). The term phase transformation

(PT), corresponds to the instant at which the behaviour of the sand transforms from contractive into dilative (i.e the commencement of decrease in excess pore pressure). Several studies have indicated that quasi steady state and phase transformation essentially occur at the same instant in sands that exhibit contractive behaviour (Vaid et al. 1990a, Ishihara 1993, Vaid and Thomas 1995).

Type 3 strain hardening response is associated with dense sands, or even loose sands at low confining stresses. The sand develops large negative pore pressures following the initial positive values. The steady state concepts contend that in addition to type 1 that realises steady state, both type 2 and type 3 responses will ultimately reach a steady state after all dilation is complete. However, this can occur only at very large strains, and only if the back pressure is high enough to accommodate large negative pore pressures so that no pore water cavitation occurs. At a given void ratio, the undrained behaviour of sands has been noted to become more contractive with increasing confining stress. (Vaid and Chern 1985, Vaid and Thomas 1995). A sand that is dilative or exhibiting type 2 response at a lower confining stress may transform into a type 1 response at higher confining stresses.

2.2.2 Cyclic triaxial behaviour

Most of the initial understanding of cyclic liquefaction has come from cyclic triaxial tests. Undrained cyclic loading causes a progressive pore pressure increase that may eventually lead to the development of large cyclic shear strains. Different mechanisms that cause large deformations depending on the initial state of the sands have been identified by Vaid and Chern (1985). Typical mechanisms of strain development are illustrated in Figure 2.2. The cyclic loading resistance in a triaxial test is defined as the uniform cyclic deviator stress amplitude required to cause a specified level of strain in a specified number of cycles, regardless of the mechanism of strain development.

2.2.2.1 Mechanisms of strain development

Large strains can develop during cyclic loading, either due to strain softening response or cyclic mobility (Castro 1969; Vaid and Chern 1985). Deformation on account of liquefaction or limited liquefaction is characteristic of statically strain softening sands only. On the other hand, large strains in a dilative sand develop only on account of cyclic mobility. The mechanism that is responsible for the deformation depends on the initial state (void ratio and the effective stresses among others) of the sand and the characteristics of the dynamic event. The initial development of strains is associated with stiffness degradation due to decreasing normal effective stress with cycles of loading. Strains are small until the excess pore pressure ratio, defined as the ratio between the excess pore pressure developed and the initial effective confining stress, exceeds at least about 60% (Seed, 1979).

True liquefaction can develop during cyclic loading in the same manner as during static loading for sands that exhibit type 1 response in static loading (Castro 1969, Vaid and Chern 1985). This type of deformation is illustrated in Figure 2.2(a).



Figure 2.2(a): Steady state of deformation following strain softening under cyclic loading (After Vaid and Chern, 1985).



Figure 2.2(b) : Strain softening followed by cyclic mobility under cyclic undrained shear (After Vaid and Chern, 1985).







Figure 2.2(d): Cyclic mobility without transient states of zero effective stress (After Vaid et al., 2000b)

A state of zero effective stress (i.e, 100% excess pore pressure) is never realised (not withstanding any subsequent unloading pulses following flow deformation), but unlimited unidirectional deformation occurs at a finite level of stresses. It is important to recognize that if the definition of liquefaction is based on 100% excess pore pressure criteria (as adopted by some researchers), then sands exhibiting this flow failure type of response would deemed to have not liquefied, even though this type of response is the most damaging.

Large strains induced by limited flow deformation, as illustrated in Figure 2.2(b), are characteristic of sands exhibiting type 2 response in static loading. Following the occurrence of limited deformation due to strain softening within a certain cycle, continued cyclic loading causes further strain due to cyclic mobility. The cause of liquefaction in this type of response can therefore be both due to limited flow deformation and cyclic mobility. However, the cyclic deformation mechanism in sands, that is dilative in static loading, is always cyclic mobility (Figure 2.2c). The occurrence of transient states of zero effective stress is a must for the development of large deformations due to cyclic mobility. Whether it occurs or not, depends on the relative magnitudes of the of the static and cyclic shear stress amplitudes. Cyclic stress amplitudes that cause shear stress reversal are necessary for realizing transient states of zero effective stress. These occur at the instance when the net shear stress during the cyclic shearing becomes zero. If the initial static shear stress is larger then the cyclic shear stress amplitude, then deformation progresses due to cyclic mobility, but without transient states of zero effective stress as schematically illustrated in Figure 2.2(d). Strain development is gradual in this type of loading as there are no excursions through transient states of zero effective stress.

2.2.2.2 Strain softening in cyclic loading

The deformation due to strain softening in cyclic loading is triggered in the same manner as in static loading, upon mobilizing the critical stress ratio noted under static loading. The sand strain softens until it reaches the steady/quasi steady state. If the response is of steady state type, unlimited unidirectional flow deformation occurs while the stress state stays stationary. On the other hand, the stresses would start to increase as the stress state moves above the PT line, if the response is of the limited liquefaction type. Subsequent unloading from the peak imposed shear stress amplitude causes a large increase in pore pressure and generally brings the sand close to a state of transient zero effective stress. Further load-unload cycles take the sand through these transient states of zero effective stress, which is responsible for additional accumulation of strain. Castro and Poulos (1977) and Vaid and Chern (1985) have put forward the following criteria for the occurrence of contractive deformation during cyclic loading.

- 1. Sand must be contractive in static loading.
- 2. The maximum shear stress must be greater than the steady state or quasi steady state undrained strength.

$$\tau_{static} + \tau_{cyclic} \ge S_{u,min}$$

3. There must be sufficient number of load cycles to lead the effective stress path to the CSR line.

2.3 Anisotropy in sand deposits

In-situ sand masses are formed by the deposition of discrete sand particles, often through water under gravity. The internal structure that ensues on such deposition has been found to be anisotropic on account of both the formation process and the shape of individual soil grains. A microscopic study of the internal structure of the sand assembly would be the direct means of assessing the existence of any anisotropy. This, however, is not often attempted, as it will be quite tedious to quantify the results except on a statistical basis. The existence of anisotropy is easily revealed by direction dependent mechanical behaviour of sands.

The anisotropy that is attributed to the formation process of the sand deposit is termed "inherent anisotropy". The nature of this anisotropy changes as the sand deforms under shear, and this has been termed "induced anisotropy" by Casagrande and Carrillo (1944). These definitions provide a convenient means of classifying anisotropy, and have been further extended by several researchers (Arthur and Menzies 1972, Wong and Arthur 1985, Symes et al. 1988, Rothenburg and Barthurst 1989).

Oda (1972) found that the preferred alignment of the longitudinal axis of non spherical sand particles is often horizontal, and that the majority of the contact normals are in the vertical direction of deposition. These observations explain the fundamental reason for the anisotropic nature of sand deposits. The existence of anisotropy in non-spherical particles is in fact due both to the preferred orientation of the longitudinal axis and the directions of the contact normals. Anisotropy in spherical particles is attributed to directions of contact normals only (Oda 1981, Haruyama 1981, Shibuya and Hight 1987). The earliest revelations of anisotropy in sands is based on consolidation of specimens under a hydrostatic state of stress. Both cubical (Kjellman 1936) and cylindrical triaxial (El-Sohby and Andrawes 1972) sand specimens have been used for this purpose.

Deposition of sand particles under gravity results in a cross anisotropic structure, with a horizontal plane of isotropy. Existence of this cross anisotropy has been demonstrated by Lade and Wasif (1988) and Yamada and Ishihara (1979) using the true triaxial device. In-situ seismic wave velocity measurements in the horizontal and vertical planes by Stokoe et al. (1991) further reveal that the in-situ sand deposits are cross anisotropic. Larger radial strains than axial under hydrostatic compression is the most compelling evidence of anisotropy (El-Sohby and Andrawes 1972, Ladd et al. 1977, Yamada and Ishihara 1979, Sayao 1989).

Induced anisotropy results from the changes in the spatial arrangement of sand particles due to the applied stresses and the resulting deformations. Oda et al. (1985) found that the spatial arrangement of sand grains and the void space within the structure of the sand deposit undergo progressive changes when subjected to shearing stresses. They found that the concentration of contact normals tends to increase in the direction of the major principal stress, and that sand particles align with their longitudinal axis along the minor principal stress. The fabric changes associated with stress/strain induced anisotropy is reported to be significantly influenced by the initial anisotropic fabric (Oda et al. 1978, El-Sohby et al. 1972). Arthur et al (1980) demonstrated that the anisotropy induced by drained pre-shear significantly influences the stiffness of the sand.

The undrained behaviour of sands under different loading modes also demonstrates the existence of inherent anisotropy (Symes et al. 1984, Kuerbis et al. 1988, Vaid et al. 1990a). The simplest manifestation of its existence is the radically different response in undrained triaxial compression and extension for identical initial states. Induced anisotropy signifies that the fabric of the sand continually evolves as the deformation progresses. It has been hypothesised that the superposition of inherent and induced anisotropy is what eventually leads to the presumed steady state of deformation, discussed later in this chapter.

2.3.1 Stress path dependent behaviour

Undrained behaviour of sands is direction dependent on account of the inherent anisotropy. However, as noted earlier, most of our understanding of the phenomenon of liquefaction has stemmed from triaxial compression test results. This test represents the behaviour of sand along one specified stress path only. A sand, at a given initial state may be contractive in triaxial extension, yet much less contractive or even dilative in compression (Bishop 1971, Miura and Toki 1982, Chung 1985, Vaid et al. 1989). Several studies have shown that dilative strain hardening response may ensue even in the loosest state (Vaid et al. 1990a, Vaid and Thomas 1995) for sands deposited in water. The dramatic differences noted in Figure 2.3 are a clear reflection of the stress path dependent undrained behaviour of sands.

Sands manifest different behaviour in triaxial compression and extension on account of the differences in major principal stress direction and the magnitude of intermediate principal stress



Figure 2.3 : Comparative triaxial compression and extension undrained behaviour of sands (After Vaid and Thomas, 1995)

parameter. The major principal stress acts vertically, perpendicular to the bedding planes, in triaxial compression ($\alpha_{\sigma} = 0^{\circ}$) but horizontally in triaxial extension ($\alpha_{\sigma} = 90^{\circ}$), and the intermediate principal stress is equal to the minor in compression, but major in extension. Hollow cylinder torsional shear tests, where the inclination of the major principal stress to vertical α_{σ} can

be independently controlled, reveal a systematic change in the undrained behaviour, from dilative to contractive, as α_{σ} changes from 0° to 90° (Symes et al. 1984, Uthayakumar and Vaid 1998, Yoshimine et al. 1998). A systematic softening of the loosest deposited Fraser River sand having identical intermediate stress parameter (b = 0) but different principal stress directions is shown in Figure 2.4. Similar stress path dependent volumetric strain behaviour is noted under drained shear (Symes et al. 1984, Vaid et al. 1990b). Stronger dilative response when $\alpha_{\sigma} = 0^{\circ}$ and weaker response when $\alpha_{\sigma}=90^{\circ}$ are consistent with the findings of Oda (1972) with regard to the contact normal densities.



Figure 2.4 : Systematic softening in undrained behaviour with increasing α_{σ} (After Uthayakumar and Vaid, 1998).

Arthur et al. (1980) pointed out that principal stress rotations occur in several in-situ loading paths. But, not much attention has been paid to the effects of principal stress rotation on the deformation characteristics of the sand. Rotation of principal stresses in the vertical plane will result in deformations on account of the cross anisotropic nature of sand deposits. Arthur et al (1979) reported that cyclic rotation of principal stresses resulted in continued strain accumulation. Shibuya and Hight (1989) have shown that a progressive increase in excess pore pressure would occur under constant magnitudes of principal stresses due to their mere rotation in an undrained state. The influence of stress rotation has, however, been assessed at relatively high initial stress ratio, and contrasting observations have been reported in the literature (Symes et al. 1985, Miura et al. 1986b). Sayao (1989) and Wijewickreme (1990) have performed an extensive study on the influence of principal stress rotation under drained conditions.

In addition to the direction of the principal stresses, the intermediate stress parameter has been shown to influence the behaviour of sands (Symes et al. 1984, Wijewickreme 1990, Uthayakumar and Vaid 1998, Sayao and Vaid 1996). The behaviour is found to be stronger when b=0 (equal minor and intermediate principal stresses) than with b = 1 (equal major and intermediate principal stresses).

Anisotropic nature of sand deposits has been recognized for a long time. However, sands are often treated as isotropic in deformation or stability analyses to gain simplicity. Even though the recent attempts to model the sand as a cross anisotropic material is a significant improvement, most of these methods still do not consider the influence of principal stress rotation.

2.3.2 The strain softening response

The peak shear stress state represents the onset of strain softening response. The effective stress ratio at the instant of peak shear stress has been termed as critical stress ratio (CSR) by Vaid and Chern (1983). The critical stress ratio is unique for a given water deposited sand in triaxial compression (Chern 1985, Chung 1985, and Kuerbis 1989). However, it depends on the deposition void ratio under triaxial extension (Vaid and Thomas 1995) and simple shear (Vaid and Sivathayalan 1996a). Strain softening in simple shear and triaxial extension modes is triggered at smaller values of critical stress ratio of moist tamped sand varies with void ratio and confining stress level in triaxial compression. Further, the dependency of critical stress ratio on the sample preparation technique has been reported by Kuerbis (1989) for a given triaxial loading mode. The strain softening deformation in cyclic loading has been shown to trigger at the same critical stress ratio noted under static loading (Vaid and Thomas 1995, Vaid et al. 2000b).

Strain softening leading to true liquefaction type of response results in unlimited deformation at "steady state" represented by constant void ratio and stresses. However, strain softening is only temporary in the limited liquefaction type of response, and the sand strain hardens following the quasi steady state due to the development of negative excess pore pressure on account of dilation.

2.3.3 The phase transformation and quasi-steady state

The state at which the response of the sand transforms from contractive to dilative is termed phase transformation by Ishihara et al. (1975). This state coincides with the instant of maximum excess pore pressure. The effective stress ratio at phase transformation, and thus the mobilised friction angle ϕ_{PT} has been shown to be independent of the void ratio, confining stress level, and mode of loading (Vaid and Chern 1985, Vaid and Thomas 1995, Vaid and Sivathayalan 1996a) for water deposited sands. The minimum undrained strength in limited liquefaction type of response occurs at or just prior to the phase transformation state (Vaid and Thomas 1995, Sukumaran et al. 1996).

Chern (1985) has shown that the friction angle mobilised at steady state (in true liquefaction type of response) is equal to the friction angle at phase transformation. The stress state in such a response remains stationary on the PT/SS line as the deformation progresses. Based on extensive test data on water deposited sands, Vaid and Chern (1985) have shown that both true liquefaction and limited liquefaction states can be treated within the same framework on the basis of the uniqueness of the friction angle and the fact that they both correspond to the minimum undrained strength of the strain softening sand. In addition to being unique for a given sand, ϕ_{PT} has been found equal to the constant volume friction angle ϕ_{CV} under drained shear (Chern 1985, Negussey et al. 1988). Negussey et al. (1988) have shown that $\phi_{PT/SS}$ is only dependent on soil mineralogy, and is independent of any other state variables whatsoever.

2.3.4 Angle of maximum obliquity & Ultimate/Failure state

On straining beyond the PT state, the effective stress path rapidly approaches the line of maximum obliquity (i.e. the ultimate failure envelope; Figure 2.1). This line has also been shown to be unique for a given water deposited sand (Vaid and Chern 1985, Vaid and Thomas 1995, Vaid and Sivathayalan 1996a). The corresponding mobilized friction angle ϕ_f is about 3° to 4° larger than $\phi_{PT/SS}$ for most sands. Miura and Toki (1982), however, show that this angle increases slightly with increasing density and is soil fabric dependent. Even though this angle remains essentially constant beyond the PT state, the shear strength of the sand increases continually with increasing strain, on account of the negative pore pressures due to dilation. The recent treatment of the undrained response of sand in which the stress state at the end of the test is regarded as the steady state (Konrad and Pouliet 1997, Ishihara et al. 1998) is incorrect. It is necessary to recognize that end of test state is arbitrary (Sivathayalan et al. 2000) and cannot be used in lieu of the ultimate steady state that a sand is expected to attain after all dilation is complete.

2.3.5 Cyclic simple shear and triaxial behaviour

Because of the inherent anisotropy in sands, its undrained behaviour is influenced both by the stress path and the loading system. The cyclic resistance of sands is often assessed using cyclic triaxial, and to a lesser extent by the cyclic simple shear tests even though the latter closely simulates the loading conditions imposed by a vertically propagating shear wave due to an

earthquake. Undrained cyclic triaxial tests have often been substituted for simple shear tests primarily because of their relative simplicity and widespread availability. The cyclic resistance measured in the triaxial test is then adjusted to the simple shear equivalent by utilizing empirical reduction factors (Seed and Peacock 1971, Vaid and Sivathayalan 1996a).

The effective stress state both prior to and during shear loading differs between cyclic simple shear and cyclic triaxial tests (Figure 2.5). Initial stress state in simple shear is always K_o and cyclic shear stresses are applied on the horizontal plane. Triaxial specimens can be either isotropically or anisotropically consolidated prior to undrained shear. The cyclic stress is imposed in the vertical direction, and the principal stresses remain fixed in the vertical and horizontal directions in a triaxial test. An alternating triaxial compression and extension loading takes place every half cycle in specimens consolidated to an isotropic stress state, or to an anisotropic stress state with cyclic shear stress amplitude exceeding static shear. This results in 90° jump rotations in the direction of major and minor principal stresses, and a jump between 0 and 1 in the intermediate principal stress parameter. Such jump rotations will not occur, and intermediate principal stress will stay equal to the minor, if the initial static shear stress is greater than the cyclic.

Simple shear loading in contrast causes continuous rotation of principal stress directions symmetrically about the vertical axis. The maximum shear stress in simple shear occurs on the horizontal plane (Roscoe 1970) whereas the maximum shear stress in the triaxial test occurs on planes oriented at 45° to vertical.





Cyclic Triaxial

Figure 2.5: Stress conditions prior to and during undrained shear in triaxial and simple shear tests.

The measure of cyclic resistance in simple shear is the ratio (τ_{cy}/σ'_{vc}) between the applied cyclic shear stress amplitude and the initial normal effective stress σ'_{vc} . In triaxial the corresponding resistance is taken as $\sigma_{d,cy}/(2\sigma'_{3c})$, i.e. the maximum cyclic shear stress normalized by the initial effective minor consolidation stress. Peacock and Seed (1968) reported that for loose sands the (τ_{cy}/σ'_{vc}) required to cause liquefaction in simple shear was merely 35% of $\sigma_{d,cy}/(2\sigma'_{3c})$ required in a triaxial test. Seed and Peacock (1971) introduced a factor C_r to be applied to the measured triaxial cyclic resistance for arriving at the cyclic stress ratio that would cause liquefaction under simple shear conditions in field. Additional studies by Finn et al. (1971) led to a suggestion for the adoption of C_r values of 0.6 to 0.7. Figure 2.6 indicates that the conversion from cyclic triaxial resistance to cyclic simple shear resistance is not unique for a given sand, but depends on the confining stress level and the relative density of the sands (Vaid and Sivathayalan 1996a). The C_r at loose density states changes little with effective confining stress, and is as high as 0.80. However, for denser states it tends to increase with increasing confining stress, and is smaller than that corresponding to loose sands.

2.3.6 Fabric dependence of behaviour

The term "fabric" is used to refer to the structure of the sand deposit that is represented by the spatial arrangement of the solid particles and the associated void space (Brewer 1964). The mechanical behaviour of sands is highly dependent on the fabric that ensues upon the deposition process. Oda (1972) was one of the first researchers to observe that specimens formed by



Figure 2.6 : Ratio of cyclic resistance in simple shear to that in triaxial (After Vaid and Sivathayalan, 1996a).

different techniques yield different mechanical behaviour. He noted that specimens prepared at identical initial void ratios by a raining technique exhibited higher strength and stiffness than those formed by a rodding technique. Different fabrics ensue depending upon the specimen reconstitution technique adopted to prepare the sand specimen. This has been illustrated by Oda (1972) using microscopical analysis of the structure of the sand specimens reconstituted using different methods. The dependency of the static undrained behaviour on the fabric of the sand has been demonstrated by Ladd (1974) and Miura and Toki (1982). Ladd (1977) and Mulilis et

al (1977) have shown that the cyclic resistance of sands is also highly dependent on the method of specimen reconstitution.

The anisotropic nature of natural in-situ sands has been studied by Oda and Koishikawa (1977) and Oda et al. (1978). They studied the particle arrangements within the soils structure using optical microscope. Their results indicate that sand grains in the natural sand deposits have preferred particle alignment and contact normal density much in the same manner as described by Oda (1972) for reconstituted sands. Oda et al. (1978) have demonstrated that the "characteristics of the initial fabric of the granular sand are determined not only by the shape of constituting grain particles, but also by the manner in which they are deposited". The anisotropic properties of in-situ sand deposits have rarely been demonstrated using its mechanical behaviour, because of the difficulties involved in obtaining undisturbed sand specimens from in-situ sand deposits.

One of the main objectives of laboratory tests is to characterize the in-situ response of sands. Since different reconstitution techniques may yield drastically different mechanical behaviour in the laboratory, adoption of a proper reconstitution method is critical. If laboratory studies are to be meaningful in modelling the behaviour of in-situ sands, then the objective of the specimen reconstitution technique should be to duplicate the in-situ fabric of the sand deposits. This can only be achieved by duplicating the process and environment in which the sand deposits are formed in-situ. Pluviation of sand particles in water simulates the alluvial deposition process and sands placed in hydraulic fills. Moist tamping techniques may simulate sands which have been compacted in such a manner while being placed in a given man made earth structure. However, it is necessary to recognize that soils are usually compacted at an optimum moisture content while being placed in an earth structure unlike in moist tamping. Moist tamping techniques often compact the sands at a lower water content in order to produce loose void ratios on account of the capillary forces. This often results in a collapsible "honeycomb" like structure (Casagrande 1975) in the moist tamped specimen. This casts serious doubts in the applicability of the results on moist tamped specimens even in compacted fills.

2.4 Critical/Steady state concepts

Poulos (1981) defined steady state as "the state of deformation for a mass of particles in which the mass is continuously deforming at constant volume, constant normal effective stress, constant shear stress and constant velocity". A special soil structure exists at the steady state that allows the soil to deform continuously at its minimum shear resistance with no volumetric or stress changes. Casagrande (1975) and Castro (1969) called such a structure as the "flow structure" for cohesionless soils. Normally, the steady state can be reached only after large strains (Poulos, 1981). Fabric changes during steady state are a special case of induced anisotropy. During flow deformation, new particle contacts are formed and existing contacts are lost in such a manner so as to maintain a statistically constant fabric. The critical state of sands is defined by Roscoe et al. (1958) as the state at which "the soil continues to deform at constant stress and constant void ratio" and is often determined from drained tests. Steady state is determined from undrained tests. Poulous et al. (1988), Been et al (1991) show that critical and steady states are

coincident, whereas test results by Castro (1969), Alarcon-Guzman et al (1988) show significant differences in the two states.

Based on triaxial compression tests on moist tamped specimens, the normal effective stress and shear stress during steady state deformation have been shown to be uniquely related to the void ratio regardless of the initial stress state (Castro 1969, Castro et al. 1982). The locus of all possible steady states is the steady state line in three dimensional space comprising void ratio, normal effective stress and shear stress (Castro et al. 1982). It is often represented by its trace in two dimensional plots as shown in Figure 2.7. The behaviour of sand at a given initial state is considered to be governed by its position relative to the steady state line (Castro et al. 1982, Been et al. 1991).

Triaxial compression tests on water deposited sands by Vaid and Chern (1985) show that the steady state line also encompasses the strain softening response of the limited liquefaction type. They contend that steady state and quasi steady state lines are identical in triaxial compression loading. Uniqueness of steady and quasi steady states in the effective stress space has been demonstrated by Vaid and co-workers and other researchers for several sands under a variety of loading conditions (Vaid and Chern 1985, Vaid et al. 1990a, Vaid and Thomas 1995, Vaid and Sivathayalan 1996a, Vaid and Uthayakumar 1998, Mooney et al. 1998). Ishihara (1993), however, has shown that the quasi steady state line always lies below the steady state line but they come closer to each other as the confining stress decreases.

The concept that a unique steady state dependent only upon the void ratio exists is quite attractive because it provides a fundamental framework for modelling sand behaviour. The





notion of a unique steady state line is so appealing that it has been considered an axiom in critical state numerical models (Graham and Hovan 1986, Jefferies 1993, Manzari and Dafalias 1997). Several researchers have promoted the idea of the existence of a unique steady state (Castro 1969, Castro et al. 1982, Ishihara et al. 1975, Been and Jefferies 1985, Been et al. 1991, Ishihara 1993, Ishihara et al. 1998). However, there is an extensive body of evidence in the literature that contradicts the notion of a unique steady state (Hanzawa 1980, Miura and Toki 1982, Vaid et al. 1990a, DeGregorio 1990, Negussey and Islam 1994, Vaid and Thomas 1995, Riemer and Seed 1997, Mooney et al. 1998, Vaid et al. 1999).

Vaid et al (1990a) have shown that stress path plays an important role in the mobilised steady state strength. They illustrated that sand at a given void ratio may strain soften and reach steady state in triaxial extension, but yet may strain harden in compression and will mobilize steady state at much higher stresses. Test results in triaxial compression and extension (Vaid et al. 1990a, Vaid and Thomas 1995), simple shear (Vaid and Sivathayalan 1996a), hollow cylinder torsional shear (Uthayakumar and Vaid 1998), and plane strain compression (Mooney et al. 1998) clearly illustrate that steady state is not unique. These studies show that the steady state strength depends on the confining stress level and the loading mode at identical void ratio. The influence of the initial anisotropic consolidation stress state on the steady state strength is not well documented in the literature even though in-situ stress states are almost always anisotropic.

Steady state in strain softening specimens is normally realised within about 10% shear strain. However, dilative specimens will not reach steady state until "all dilation is complete" (Poulous 1981). Several studies show that dilative specimens do not realise steady state even at strains as high as 25 to 30% (Vaid et al. 1990a, Ishihara 1993, Konrad and Pouliet 1997). No laboratory apparatus is capable of straining the sand to the very high strain levels that are required to realise steady state in dilative sands, without causing excessive non uniformities. Further, attempting to treat the end of test state as a characteristic state of the sand (Konrad and Pouliet 1997, Ishihara et al. 1998, Tsukamoto et al. 1998) is fallacious. End of test state is arbitrary and has no physical meaning whatsoever.

In recent studies, Tsukamoto et al. (1998), Ishihara et al. (1998) and Konrad and Pouliet (1998) suggested that fabric effects disappear at about 20% axial strain. Sivathayalan and Vaid (1999) pointed out that a careful examination of the data presented by Konrad and Pouliet would in fact indicate that fabric effects are dominant even after 20% strain. The very large scatter (by a factor of about four in steady state strength) in the data of Tsukamoto et al. (1998) and Ishihara et al. (1998) arise on account of the influence of the loading mode (Sivathayalan et al. 2000).

The minimum undrained strength and not the steady state strength is of concern in Engineering design. These two cases coincide for strain softening specimens of type 1. However, type 2 strain softening specimens will have a much smaller minimum strength (at quasi steady state) than their steady state strength. The observed non-uniqueness of the steady state line has serious implications in the steady state concepts of design wherein a unique steady state line is assumed based on results of triaxial compression tests only. Such an approach ignores the dependence of the minimum undrained strength on the confining stress, loading mode, initial static shear and fabric. In a given field problem, variation of the minimum undrained strength, depending upon

the effective stress state and loading mode must be taken into account for a rational assessment of the potential for a flow slide on a sloping ground.

2.5 The stress state in-situ

The preceding discussion clearly indicates that the response of sands is direction dependent, and is influenced by the initial stress state. In-situ sands in nature are subject to a variety of loading modes, superimposed on a generalised three dimensional initial stress state. Figure 2.8 illustrates the stress states to which a sand element in the field may be subjected prior to and during construction and performance. There will be no shear stresses on the horizontal plane, and hence the major principal stress will act vertically downwards in a sand element under flat ground (Figure 2.8a). The construction of an earth structure close by, (e.g., buildings, dams etc.), may induce shear stresses on the horizontal plane, ultimately resulting in a new stress state as shown in Figure 2.8(b). The effect of shear stresses on the horizontal plane is to rotate the direction of principal stresses. The inclination of the major principal stresses to vertical depends on the relative values of the shear stress on the horizontal plane and the vertical stress. Any undrained stress increment, due either to static or dynamic loading, will now be superimposed on this stress state (Figure 2.8c). This often will result in rotation of principal stress directions in addition to an increase in deviator stress. This is illustrated in the Figure only in one plane for simplicity, but such generalised loading may occur in all three planes.



(a) : Initial stress state





(c): stress state during undrained shear



Figure 2.8 : Stress state of soil elements in-situ.

Further, the direction of principal stresses and the initial stress state will possibly vary along a potential failure surface in-situ. The direction of major principal stress will often vary from about 0° at the top of the failure surface to about 90° at the bottom. This type of generalised stress state will often be the rule rather than the exception in-situ. Numerical analysis of the effective stress states and deformations of the J-pit embankment in the CANLEX project indicated that stress rotation as much as 60° will be encountered in the dam during construction and performance (Puebla, 1998). It also indicated that depending on the initial effective stress state, the stress rotation due to subsequent loading may either increase α_{σ} or decrease it.

It is evident that most of the in-situ stress states are three dimensional and subject to stress rotation. Nonetheless, most of our understanding of the behaviour of sands has been derived from tests on isotropically consolidated triaxial specimens under fixed principal stress directions. This was on account of the earlier presumption, based on the steady state concepts, that the response of sands is dependent only on the effective confining stress. However, as noted earlier several researchers have since then published results to the contrary. Triaxial compression, extension and simple shear represent specific loading modes and thus cannot be regarded as representative of all the possible spectrum of responses. The undrained behaviour of sands has invariably been assessed under hydrostatic, and to a lesser extent under axisymmetric consolidation stresses. Even though the influence of initial static shear (K_0 consolidation) has been recognized by several researchers (Vaid and Chern 1983, Seed and Harder 1990, Vaid et al 2000b), no attempts have been made so far to assess the influence of non axisymmetrical initial static stress state.

The initial effective stress state of the sand, characterized by the three principal stresses and the direction of the major principal stress $\alpha_{\sigma c}$ is represented by the effective mean normal stress σ'_{mc} , the intermediate stress parameter b_c , and the ratio of major to minor principal stress K_c and $\alpha_{\sigma c}$ in this thesis for convenience. The hollow cylinder torsional shear device allows independent control of each of these four parameters.

2.6 Research Needs

The stress path dependency of the undrained behaviour of sands is widely recognized, primarily from comparative undrained triaxial compression and extension behaviour of water deposited sands (Vaid et al. 1990a, Riemer and Seed 1997). This occurs on account of the inherent anisotropy present in sand deposits. Our understanding is based on specimens reconstituted in the laboratory, and no direct evidence of direction dependent mechanical behaviour of natural in-situ sands is available in the literature. In addition to the stress path during undrained shear, the initial effective stress state prior to undrained shear plays an important role in the subsequent undrained response of the sand. The effect of initial effective stress state has been studied using anisotropically consolidated sands, but only in triaxial compression. To the knowledge of the author, the influence of initial non axisymmetric effective stress states has not been studied, despite the fact that the effective stress conditions in embankments, and slopes prior to undrained shear is non axisymmetric, and these are the structures that are most prone to flow failure.

Undrained principal stress rotation at constant magnitudes of principal stresses induces excess pore pressure, and eventually large deformations. The liquefaction susceptibility due to stress rotation is dependent on the initial stress state and the sense of loading under non axisymmetric initial stress states; upstream loading decreases α_{σ} and downstream loading increases α_{σ} . Little systematic research has been carried out to assess the influence of stress rotation on the undrained behaviour of sands, and no data exist on the effects of the sense of loading.

It has been recognized that not all laboratory reconstitution techniques produce the same fabric. As a result, the undrained behaviour of sands vary widely depending on the method of specimen reconstitution in the laboratory (Mulilis et al 1977, Ladd 1977, Miura and Toki 1982), and this often has been cited to discourage the direct application of laboratory test data to sands in-situ. However, no attempts have been made to assess the comparative behaviour of undisturbed in-situ sands, with reconstituted specimens, in order to evaluate the applicability of reconstitution methods, except for the microscopical studies of Oda (1972).

Post liquefaction stress-strain behaviour is the primary input in assessing liquefaction induced displacements. Current analytical methods, often rely on empirical data to determine the post liquefaction stiffness and strength of a sand. The little systematic research carried out on the post liquefaction behaviour of sands, indicates that the stiffness increases with strain, and that the virgin minimum undrained strength is not a limiting strength on post liquefaction loading. However, no data on the post liquefaction behaviour of in-situ sand deposits is available

The research reported herein is an attempt to address some of the above concerns. The static undrained behaviour of specimens reconstituted by different techniques were compared in order to demonstrate the influence of fabric. These tests were carried out in triaxial and simple shear, and it is shown the domain of accessible states is dependent on the method of reconstitution. A loose void ratio at a given confining stress that is accessible in the laboratory by a certain method of reconstitution may not even be accessible to the natural sand in-situ. Therefore, the relevance of the behaviour of such specimens to the field sands is highly questionable.

Extreme care was taken in the laboratory testing to minimize any apparatus specimen interactions, in order to ensure high quality and reliability of the data. The effects of membrane penetration, on the drained and undrained response was comprehensively assessed and eliminated if necessary. The test techniques yield excellent repeatability and therefore very high confidence in the measured data.

Undisturbed in-situ frozen sands are tested in triaxial compression, extension and simple shear in order to explore the stress path dependent undrained behaviour of natural in-situ sands. These sands are tested under static, cyclic and post liquefaction loading, and their response is compared with reconstituted counterparts. A comparison of the behaviour of water pluviated and undisturbed sands under is made in static, cyclic and post liquefaction loading. This is attempted to evaluate the applicability of laboratory tests on reconstituted specimens, to the sand in-situ.

The influence of initial effective stress state prior to undrained shear, on the subsequent behaviour of sands is demonstrated using specimens consolidated to non axisymmetric effective stress states, with varying α_{σ} and K_c . Water pluviation was chosen as the method of specimen reconstitution in these tests, because it duplicates the deposition process in an alluvial environment. In-situ stress states are often non axisymmetric, and therefore an understanding of

the undrained behaviour under these conditions is essential for successfully characterizing the behaviour of in-situ sands. The effect of principal stress rotation leading to liquefaction and its dependency on the initial stress state are assessed. These tests explore the uniqueness, or lack thereof, of the steady state line. This thesis comprehensively addresses these critical issues, and hopefully, the findings will inspire more rational methods of analysis and design, and better our understanding of the liquefaction phenomena.
Chapter 3

Experimentation and Test Apparatus

3.1 Introduction

Triaxial, simple shear and hollow cylinder torsional shear devices have been used in this study to assess the undrained behaviour of sands. Because of the non linear and anisotropic nature of soils, the usage of different test apparatus becomes essential in order to assess their response under different stress systems and loading paths anticipated in field problems. The triaxial apparatus is limited only to axisymmetric stress states in compression and extension loading. The simple shear apparatus applies plane strain loading in one horizontal direction, and enforces zero extension in the other perpendicular horizontal direction. It also imposes uncontrolled principal stress rotation during loading. The hollow cylinder torsional (HCT) device is more versatile because it can apply a variety of loading paths, including controlled principal stress rotation in one plane. The HCT device, however, has seen only a limited use in soil testing, primarily because of the complexities of the apparatus and test procedures.

The behaviour of soils is traditionally assessed using cylindrical specimens in the triaxial devices. The simple shear device, though more complex than triaxial, is most suitable to assess the cyclic resistance of sands, because it closely simulates the loading caused by the vertically

propagating shear waves in an earthquake. The configuration and capabilities of each testing device used in this study are briefly described in the following sections. A detailed description can be seen elsewhere (Sayao 1989, Sivathayalan 1994, Thomas 1992).

3.2 Laboratory shear testing of soils

The objective of laboratory shear tests is to characterize the behaviour of the soil element insitu. Any parameter that affects the response of the soil in-situ should be appropriately represented in the laboratory test. The entire soil specimen in the laboratory represents a single point in the soil mass in-situ and thus all laboratory tests must be element tests. Laboratory test results can be properly interpreted only if the test remains an element test, and this demands a homogeneous specimen subjected to uniform stress state. The attempts to characterize different in-situ conditions have led to the use of several soil testing devices, that can often be conveniently categorized as axisymmetric, plane strain or multi axial.

A typical triaxial device uses a cylindrical specimen, and only the vertical and horizontal normal stresses can be controlled in this device. The axisymmetric geometry forces two of the three principal stresses to be equal at all times. Bishop and Henkel (1962) excellently detail all aspects of the triaxial test. Plane strain devices have gained considerable prominence because field conditions are often of the plane strain type. The simple shear device (Kjellman 1951, Roscoe 1953, Bjerrum and Landva 1966) is one of the most commonly used plane strain apparatus. Normal and shear stresses are applied in the horizontal plane of the laterally confined

simple shear specimen. The stress conditions lead to principal stresses inclined at 45° to the vertical, and the direction of the principal stresses cannot be independently controlled in a simple shear device. The directional shear cell (Arthur et al. 1981) is a plane strain device that allows controlled rotation of principal stress directions.

Annular and hollow cylindrical specimens have been used in shear testing of soils since the early days (Cooling and Smith 1936, Hvorslev 1939). The currently used hollow cylinder torsional shear device is an evolution of these early devices, and it allows independent control of the magnitudes of the principal stresses and their directions. This enables the device to follow any complex stress path and makes this one of the most versatile soil testing devices. Saada and Townsend (1981) provide a detailed discussion of the advantages and limitations of various soil testing devices. Triaxial, simple shear and hollow cylinder torsional shear devices were used in the investigations undertaken herein.

3.2.1 The triaxial test

Most of our understanding of soil behaviour has come from controlled laboratory tests in the conventional triaxial test. Cylindrical specimens, with a height to diameter ratio of at least two are subjected to axisymmetric loading in this test. Only compression tests where the major principal stress acts vertically and extension loading with major principal stress acting horizontally can be simulated in a triaxial test. The major and minor principal stress magnitudes can be controlled, and the intermediate principal stress equals either the minor principal stress in a

compression test, or the major principal stress in an extension test. The principal stresses in this test remain fixed in the vertical and horizontal directions at all times.

A true triaxial device enables independent control of the magnitude of each of the three principal stresses (Ko and Scott 1967, Arthur and Menzies 1968, Pearce 1971, Green 1971). However, their direction cannot be arbitrarily controlled. Principal stress directions are aligned with the specimen axis and only 90° jump rotations can be simulated. This is because only normal stresses can be applied on the prismatic boundaries of the specimen.

End restraint due to frictional platens in the conventional triaxial test causes progressive stress and strain non-uniformities within the specimen with increasing deformation. This then compromises the presumed equality of radial and tangential stresses and strains at larger strains (Casbarian and Jamal 1963, Frydman et al 1971), and introduces uncertainty in the interpretation of the results as an element test. Lubricated end platens and free ends (Rowe and Barden 1964) have been used as means of reducing the end restraint effects. Their usage, however, introduces bedding errors in deformation measurements.

3.2.1.1 The UBC triaxial apparatus

A schematic diagram of the UBC triaxial apparatus is shown in Figure 3.1. Cylindrical specimens of 60mm nominal diameter and 120mm nominal height were used. An external load cell is used to measure the axial deviatoric load, together with an essentially frictionless, continually air leaking seal on the loading ram. The cell pressure and the pore pressure are



Figure 3.1 : Schematic diagram of the UBC static/cyclic triaxial device

measured with pressure transducers, the volume changes of the specimen by a differential pressure transducer, and the axial displacement by a displacement transducer. The deviatoric stress to the specimen can be applied either stress controlled by the double acting pneumatic piston or with displacement control by the variable speed motor drive connected in series with the piston. Thus, this apparatus can perform loading compression tests or unloading extension tests. The electro-pneumatic transducer connected to one chamber of the double acting piston is controlled by the computer. This enables automated anisotropic consolidation prior to static or cyclic shear loading.

3.2.2 The simple shear test

The specimen geometry of the simple shear specimen is either cubical or cylindrical. The horizontal normal strain components ε_x and ε_y are zero in this test on account of the enforced parallel lateral boundaries. Consolidation prior to shear is one dimensional, and corresponds to the K_o condition. Constant volume conditions are enforced to simulate undrained shear by arresting the vertical displacement and thus forcing the vertical strain ε_z to be zero. The reduction in vertical stress σ_v in such a constant volume test then equals the excess pore pressure developed in an equivalent undrained test (Dyvik et al. 1987). The average boundary normal and shear stresses are the only measured stress components in a simple shear test. These are not sufficient to completely prescribe the state of stress within the specimen. Studies in hollow cylinder torsional shear (Uthayakumar 1996) and highly instrumented Cambridge cuboidal simple shear

device (Roscoe, 1970) have shown that the horizontal plane in simple shear is the plane of maximum shear stress. Thus, the principal stresses are inclined at 45° to the vertical axis of symmetry. The measured normal and shear stresses together with the knowledge that the principal stresses are inclined at 45° to the axis of symmetry, enables a complete definition of the plane strain state of stress.

It is recognized that the absence of complementary shear stresses on the vertical boundaries, results in non uniform normal and shear stress distribution on the horizontal plane in the simple shear specimen. However, these non uniformities are severe only at the edges (Duncan and Dunlop 1969, Cole 1967, Finn et al. 1978, DeGroot et al. 1994). In fact, detailed measurements of the normal and shear stresses using several contact transducers on the specimen boundaries at Cambridge has shown that the stress conditions in the middle third of the specimen are fairly uniform and their magnitude approximately equals the average boundary stresses (Cole 1967, Airey et al. 1985). Airey et al. (1985) found that a low specimen height to diameter ratio reduces stress non uniformities considerably. Further Duncan and Dunlop (1969) reported that "*the stress non uniformities have little effect on the ultimate shear resistance of soils, and it is sufficiently accurate to use the average stress conditions for test interpretations*". Notwithstanding the concerns of stress uniformities, the simple shear test is very attractive in practical applications on account of its ability to closely model the upward propagation of shear waves during an earthquake.

3.2.2.1 The UBC simple shear apparatus

The simple shear apparatus used in this study was of the NGI type (Bjerrum and Landva, 1966). A'cylindrical soil specimen is laterally confined by a steel reinforced rubber membrane in this apparatus (Figure 3.2). Since the reinforcement enforces an essentially constant cross sectional area, arresting the vertical displacement simulates constant volume conditions. A schematic diagram of the simple shear apparatus is shown in Figure 3.3. A single acting air piston applies the vertical load to the sample. The horizontal shear stress can be applied either by the double acting pneumatic piston for stress controlled or by the variable speed motor drive for strain controlled loading. The loading mechanism is essentially identical to that in the triaxial apparatus and permits switching to displacement controlled from stress controlled loading during



Figure 3.2 : Specimen and loading arrangement in UBC simple shear apparatus





a test. The normal and shear stresses are measured by the vertical and horizontal load cell. Vertical strain, which equals the volumetric strain is monitored by a vertical displacement transducer. The shear strain is measured by horizontally positioned displacement transducers. Two LVDTs were used in the horizontal direction in order to increase the available displacement range and yet to have high measurement resolution at low strain levels.

Cyclic loading was applied by changing the pressure in one of the two chambers of the double acting piston, using an electro-pneumatic transducer. The electro pneumatic transducer is controlled by a computer. An air volume booster is connected to the chambers of the double acting piston in order to maintain a constant amplitude of cyclic shear stress when large deformations occur due to liquefaction. Both undisturbed in-situ frozen and reconstituted sand specimens were tested in order to assess their monotonic, cyclic and post liquefaction response. Pre and post liquefaction monotonic tests were performed under the displacement control whereas cyclic loading performed under stress control.

3.2.3 The Hollow Cylinder Torsion (HCT) test

Thin walled hollow cylindrical soils specimens subject to confining pressure on both inside and outside the cylindrical boundaries, together with torsional shear stress on the horizontal plane allow controlled shear loading including principal stress rotation under multi-axial stress states. The HCT device can impose deviatoric stresses on the specimen by changing the vertical stress, the torsional shear stress on the horizontal plane or by a combination of these two. The control of internal and external confining pressures, vertical stress and torsional shear stress allows independent control of the principal stresses and their rotation. Earlier HCT devices used equal internal and external pressure for specimen confinement. However, the ability to apply different values of internal and external pressures is required to independently control the direction of principal stresses.

The HCT test is frequently subject to criticism on account of the non uniform stresses across the wall of the specimen when different internal and external pressures are used. The uniformity of the stresses depends on the specimen geometry, and the HCT devices are designed to minimize the stress non-uniformities. Thin walled specimens, with large radius and a height to external diameter ratio of about two result in minimal stress non uniformities (Hight et al. 1983, Saada 1988, Sayao 1989, Vaid et al. 1990b).

3.2.3.1 The UBC hollow cylinder torsion device

A schematic diagram of the UBC HCT device is shown in Figure 3.4. The specimen dimensions were adopted so as to minimize the stress non uniformities (Sayao 1989). Several modifications have been made to the original device designed in 1986 in order to facilitate strain controlled testing (Wijewickreme 1990) and combined stress and strain controlled loading (Uthayakumar 1996). Detailed description of this device is given in Sayao (1989) and Vaid et al (1990b), and only the modifications made during the course of this research are detailed following a description of the apparatus.





The nominal internal and external diameters of the specimen are 10 cm and 15 cm respectively, and the height to diameter ratio of the specimen is two. The domain of stresses that can be explored without causing excessive stress non-uniformities were delineated by considering appropriate stress strain characteristics of sands (Wijewickreme and Vaid 1991).

The stresses are applied by stepper motor driven precision regulators and/or stepper motor driven water saturated pistons called digital volume-pressure controllers (or DVPCs). Independent control of four of these stresses yield independent control of the principal stresses $(\sigma_1, \sigma_2, \sigma_3)$ and their direction (α_{σ}) in one plane. For testing purposes, these four independent parameters are alternatively expressed in terms of the derived stress parameters σ'_m , α , K_c and b. The usage of DVPCs in soil testing has been pioneered by Menzies (1988). It enables predetermined volumes of water to be injected into or withdrawn out of the loading piston, and thus provides a convenient means of applying displacement controlled loading. Such a system does not suffer from the large compliance that is endemic in traditional motor driven gear systems.

The specimen is fixed at the top loading cap, and the vertical load is applied by a double acting piston mounted at the bottom of the loading frame. The water saturated bottom chamber of the piston is connected to the vertical loading arrangement as shown in the inset in Figure 3.4. This enables the application of stress controlled loading during consolidation, by the stepper motor driven regulators and subsequent strain controlled loading by the DVPCs.

Torsional shear is applied by means of four single acting pistons. Two of these pistons are water saturated and connected to a loading arrangement similar to that of the vertical piston and the other two are used for balancing. The torque is generated by the differential pressures in the piston pairs and transmitted to the specimen by cables and pulleys. The entire torsional loading system is mounted on the axial loading shaft and moves up or down with the sample deformation. This eliminates any vertical or horizontal forces on the loading shaft compared to the case of a fixed system. The torsional shear is transmitted to the specimen by twelve 1mm thick and 2mm high radial ribs that are equally spaced on the loading platen.

Six equally spaced porous stones that cover only about 8% of the specimen area provide drainage to the specimen. The radial restraint is minimized by using polished anodized aluminum end platens together with porous stones that cover only a small fraction of the area of the specimen. The positioning of the porous stones and the radial ribs is shown in Figure 3.5. The bottom drainage line is connected to a burette and the top drainage to a burette and a DVPC, as illustrated in Figure 3.6. This enables application of back pressure with a regulator, or alternatively the control of the volumetric strain of the specimen by the DVPC. These are necessary in strain path controlled tests, as well as for compensation of membrane penetration induced volume changes in undrained tests.

Internal and external cell pressures and the back pressure to the membrane lined, soil specimen are applied through air water interfaces. Air diffusion into soil specimen or cell is minimized by using long narrow bore diffusion coils (Chern 1985, Sayao 1989). The internal and external cell pressures are controlled by stepper motor controlled precision regulators, and the back pressure is applied manually using a precision regulator. The pressure transducers are mounted close to the cell and specimen in order to minimize compliance effects. The vertical load on the specimen





Figure 3.5 : Arrangement of porous stones, shear ribs and drainage passage in HCT loading cap



Figure 3.6 : Schematic arrangement of pore water drainage in an HCT specimen

is measured by the external load cell placed at the bottom of the loading shaft and is appropriately corrected for shaft friction. The torque is measured using a torque cell placed above the torque pulley. Careful design of the torque cell has resulted in negligible amount of cross talk between the vertical load and torque.

Four deformation components are measured in the UBC HCT device. A linear variable displacement transformer (LVDT) measures the axial strain, and a rotating arm connected to the loading shaft converts the rotation of the base platen into linear tangential displacement, and enables another LVDT to measure the angular displacement and hence the shear strain. The latter LVDT is mounted at a sufficiently large distance away from the loading shaft in order to avoid

necessitating corrections in the measured tangential displacement due to the vertical movement of the rotating arm (Figure 3.7). The volume change of the specimen is recorded using a differential pressure transducer (DPT) and a pipette, and of the inner chamber by another DPT. All volume measurements are corrected for membrane penetration effects, if applicable, before computing the strain components.

3.2.3.2 Stresses and strains in an HCT specimen

The average stress state in an HCT specimen is computed from the measured vertical load (F_z), Torque in the horizontal plane (T_h), internal cell pressure (P_i) and the external cell pressure (P_e). Figure 3.8 shows the boundary forces acting on an HCT specimen together with the stresses in the wall element. The stress state in the element is expressed in terms of vertical (σ_z), radial (σ_r) and tangential (σ_{θ}) normal stresses and shear stress $\tau_{z\theta}$. The test interpretation is made by considering the entire specimen as an element deforming as a right cylinder. Since the magnitudes of σ_r , σ_{θ} and $\tau_{z\theta}$ vary across the thickness of the wall, average representative values are calculated.

The stress state in a thick cylinder subject to internal and external pressures, axial and torsional shear loads cannot be completely solved by equilibrium considerations alone. It, in addition, requires a knowledge of the constitutive behaviour of the cylinder material. The vertical stress σ_z , however, is independent of the material constitutive behaviour, and can be directly obtained from

the measured vertical load and the cross sectional area of the specimen, with appropriate corrections.



Figure 3.7: Schematic arrangement of the torsional displacement measuring system in the UBC hollow cylinder torsional shear device.





Different approaches of solving the mechanics of the thick cylinder geometry have resulted in different equations for σ_r , σ_{θ} and $\tau_{z\theta}$. These stresses were calculated by Symes et al. (1985) and Vaid et al. (1990b) by considering the material to be linearly elastic. The radial and tangential stresses at any radius r are then given by

$$\sigma_{r (at r=R)} = \frac{P_{e} \cdot r_{e}^{2} - P_{i} \cdot r_{i}^{2}}{R_{e}^{2} - R_{i}^{2}} - \frac{(P_{e} - P_{i}) \cdot R_{e}^{2} \cdot R_{i}^{2}}{(R_{e}^{2} - R_{i}^{2})} \cdot \frac{1}{R^{2}}$$

$$\sigma_{\theta (at r=R)} = \frac{P_{e} \cdot r_{e}^{2} - P_{i} \cdot r_{i}^{2}}{R_{e}^{2} - R_{i}^{2}} + \frac{(P_{e} - P_{i}) \cdot R_{e}^{2} \cdot R_{i}^{2}}{(R_{e}^{2} - R_{i}^{2})} \cdot \frac{1}{R^{2}}$$
(3.1)

where R_i , R_e are the internal and external radii respectively of the specimen. Vaid et al. (1990b) consider the equilibrium of forces to arrive at the average σ_r and σ_{θ} , whereas Symes et al. (1985) use the average of the individual stresses as shown in equation 3.2.

Symes et al. (1988):
$$\sigma_r = \frac{\int \sigma \, dr}{\int dr}$$

Vaid et al. (1990b): $\sigma_r = \frac{\int \int \sigma \, r \, d\theta \, dr}{\int \int r \, d\theta \, dr}$
(3.2)

Miura et al. (1986a) calculated the average stresses assuming a linear variation across the walls. Figure 3.9 shows that differences among the average values of stresses calculated by different methods are relatively minor. The following set of expressions proposed by Vaid et al. (1990b), based on their merit of satisfying the force equilibrium are used in this thesis to calculate the stresses.

$$\sigma_{z} = \frac{F_{z} + \pi \cdot (P_{e} \cdot R_{e} + P_{i} \cdot R_{i})}{\pi (R_{e}^{2} - R_{i}^{2})}$$

$$\sigma_{r} = \frac{(P_{e} \cdot R_{e}^{2} - P_{i} \cdot R_{i}^{2})}{(R_{e}^{2} - R_{i}^{2})} - \frac{2 \cdot (P_{e} - P_{i}) R_{e}^{2} R_{i}^{2} \ln(R_{e}/R_{i}))}{(R_{e}^{2} - R_{i}^{2})^{2}}$$

$$\sigma_{\theta} = \frac{(P_{e} \cdot R_{e}^{2} - P_{i} \cdot R_{i}^{2})}{(R_{e}^{2} - R_{i}^{2})} + \frac{2 \cdot (P_{e} - P_{i}) R_{e}^{2} R_{i}^{2} \ln(R_{e}/R_{i}))}{(R_{e}^{2} - R_{i}^{2})^{2}}$$

$$\tau_{z\theta} = \frac{4 \cdot T_{h} \cdot (R_{e}^{3} - R_{i}^{3})}{3 \pi (R_{e}^{4} - R_{i}^{4}) \cdot (R_{e}^{2} - R_{i}^{2})}$$
(3.3)

The average strain components are calculated assuming linear variation of displacement across the wall of the specimen. This assumption is consistent with the presumed linear elastic response in calculating the stresses, and yields the following expressions.

$$\epsilon_{z} = -\frac{\Delta h}{h}$$

$$\epsilon_{r} = -\left(\frac{\Delta R_{e} - \Delta R_{i}}{R_{e} - R_{i}}\right)$$

$$\epsilon_{\theta} = -\left(\frac{\Delta R_{e} + \Delta R_{i}}{R_{e} + R_{i}}\right)$$

$$\gamma_{z\theta} = \left(\frac{2\Delta \theta (R_{e}^{3} - R_{i}^{3})}{3h(R_{e}^{2} - R_{i}^{2})}\right)$$
(3.4)

Where Δh and $\Delta \theta$ are the axial displacement and the angular rotation,

h is the height,

and ΔR_i and ΔR_e are the change in the internal and external radii respectively.



Figure 3.9 : Comparison of different approaches to find representative stresses in a hollow cylindrical soil element.

The change in the inner chamber radius ΔR_i is calculated from the measured volume change of the inner chamber and the axial deformation Δh . The change in the external radius ΔR_e is then calculated using ΔR_i , Δh and the volume change of the specimen.

The absence of shear stresses in the $r\theta$ and rz planes makes the radial stress σ_r a principal stress. Often σ_r is the intermediate principal stress. The major and minor principal stresses can then be computed from σ_z , σ_{θ} and $\tau_{z\theta}$. The principal strains are also computed in the similar manner.

3.3 Data acquisition and control systems

All three shear testing devices used in this study use sophisticated, high speed data acquisition and control systems. Data in simple shear tests is gathered using a *PCL718x*, 12-bit high speed data acquisition card. This data acquisition card has eight analog to digital (A/D) input channels and two digital to analog (D/A) output channels. Five of the A/D channels are used in simple shear for transducer input; one each for vertical and horizontal load cells, one for vertical displacement transducer (LVDT) and two for horizontal displacement transducers. Two LVDTs are used to measure the horizontal displacement in order to have a larger range, and still retain higher measurement resolution at small strain levels. One of the D/A channels is used to control the electro pneumatic transducer and the other the motor drive. The data acquisition system is capable of gathering about 6000 data sets per second. The gathered data is averaged over sixty readings by the data acquisition program in order to reduce the electrical noise that is inherently present in the 60Hz a.c electrical supply.

Both the triaxial and hollow cylinder torsional shear devices use "*National Instrument AT-MIO16x*" 16-bit high speed data acquisition card for signal input. This data acquisition card in the triaxial apparatus is set up to use eight double-ended A/D input channels and two D/A output channels. Five of the A/D channels are used for transducer input in the triaxial test; two for pressure transducers and one each for the axial load, axial displacement and the specimen volume change. One D/A channel of this card is used in the triaxial test to control the electro pneumatic transducer to facilitate anisotropic consolidation, and cyclic loading. The full scale range of each

of the channels in this card is dynamically controlled in real time using software in order to improve the measurement resolution. This system scans about 3000 data sets per second, and sixty readings are averaged to filter out the noise in the input signals. The horizontal effective stress in the triaxial test is calculated from the pressure transducer readings, and the deviatoric stress from the load cell reading. Axial and volumetric strains are calculated from the LVDT and DPT respectively. The radial strains are calculated from the volumetric strain (zero during undrained shear) and the axial strain, since axisymmetric conditions prevail in a triaxial test.

In addition to the "National Instrument "*AT-MIO-16X*" data acquisition card for A/D input, a 16 bit three channel D/A "Vexta Indexer" card for controlling the DVPCs and a 15 bitequivalent, four channel D/A card for controlling the stepper motor regulators are used in a hollow cylinder test. All three of these cards are used in conjunction in real time to control the surface tractions and/or the strains in order to follow the prescribed stress/strain path. Nine of the sixteen single ended A/D channels in the AT-MIO10x card are used to read input from the three pressure transducers, two differential pressure transducers, two LVDTs, a load cell and a torque cell.

All transducer signals are filtered using a signal conditioner before being fed into the A/D card. The signal conditioners are designed and built in house in the electronics workshop in the department. About fifty sets of data can be gathered by the software after averaging sixty scans in order to enhance the resolution of the measurements.

3.3.1 Measurement resolutions

The high resolution, high speed data acquisition system enabled confident measurement of stresses and strains in all tests. The radial and axial stresses in the triaxial test are measured with a resolution of about 0.25 kPa and both the axial and volumetric strains are measured with an accuracy of about 10⁻⁵. The measured stresses are appropriately corrected for membrane stiffness, loading ram friction, loading cap weight and the uplift force on the ram, and the volumetric strain is corrected for membrane penetration effects.

No corrections are necessary for the measured vertical load in the simple shear test, on account of the positioning of the vertical load cell. The measured shear load was corrected for loading shaft friction and the stiffness of the reinforced membrane in simple shear test. The normal and shear stresses in simple shear have a resolution of better than 0.25 kPa and the vertical strain has a resolution of about 0.01%. The resolution of shear strain measurements is better than 0.01% for shear strains up to 10%, and is about 0.05% for shear strains in excess of 10%. The electro pneumatic transducer that was used to apply cyclic shear stress in simple shear or K_c consolidation in triaxial has a resolution of about 0.10 kPa.

The high precision pressure transducers used in the hollow cylinder torsional shear apparatus enable the internal, external and pore pressures be measured with a resolution of better than 0.05 kPa. The resolution of the vertical and the torsional shear stresses are about 0.2kPa. The DPTs enable volume measurement accurate to about 2 mm³, that translates to a volumetric strain resolution of better than 10^{-5} . The LVDTs can detect a displacement of about 5 µm. This is equivalent to an axial and shear strain resolution of about 10^{-5} . LVDTs that can detect up to 1 μ m displacement were used when finer measurement resolutions were required in certain tests. The range of the A/D data acquisition card was dynamically adjusted using software control to further enhance the resolutions at low transducer output levels (i.e., at low stress and strain levels). The stress/strain path control was achieved by using a feedback loop to apply the required stresses/strains. A window of 0.25 kPa was used in all stress path controlled tests. The controller cards use square pulses to drive both the DVPCs and the stepper motor regulators. The stepper motor regulators are controlled with an accuracy of about 0.1 kPa. These stepper motor regulators can apply pressure gradients from about 4 kPa per second to 40 kPa per second. The stepper motor connected to the DVPC requires 400 pulses per revolution. This is equivalent to an accuracy of about 4 mm³ of volume. The area of the vertical and torsional shear load pistons are sufficiently large to yield better than 10^{-5} accuracy in axial and shear strains.

3.4 Test procedures

Triaxial and simple shear tests were performed on both undisturbed and reconstituted sand specimens, and hollow cylinder torsional shear tests were carried out only on specimens reconstituted by water pluviation (Vaid and Negussey 1988). Undisturbed sand specimens were obtained using in-situ ground freezing as part of the CANLEX laboratory testing program. Reconstituted specimens were formed using Fraser River sand, that is similar to the undisturbed Kidd and Massey sands.

Frozen undisturbed specimens retrieved after in-situ ground freezing were machined to the prescribed dimensions (60 mm diameter x 125 mm height for triaxial and 69.8mm diameter x 20mm height for simple shear) at the University of Alberta, Edmonton and shipped at regular intervals to UBC in cold boxes with dry ice. These specimens were stored in the freezer at -20°C until needed for testing. Details of setting up a frozen specimen on a triaxial or simple shear apparatus and thawing it to the room temperature are given in Vaid et al. (1996). All undisturbed specimens were consolidated to the in-situ effective stress states, except a few in Phase III, where the specimens were consolidated to the anticipated stress levels after the construction of the test embankment (Byrne et al. 2000).

3.4.1 Specimen reconstitution

Studies aimed at providing a fundamental understanding of the mechanical behaviour of sands require sand specimens that are homogeneous and repeatable. Ideally, these tests would be performed on several identical (and uniform) undisturbed sand samples. However, it is quite difficult to find in-situ sand samples that are uniform and identical. Further, the conventional sampling techniques may significantly alter the mechanical properties of sands (Seed et al. 1982), and the alternative in-situ ground freezing is very expensive. This has necessitated the use of reconstituted, rather than undisturbed sand specimens. If the properties of the reconstituted sands are to be assigned to the in-situ sands, then the laboratory specimen reconstitution technique must mimic the in-situ deposition process. This is because the sand behaviour is known to be profoundly affected by the fabric that ensues upon reconstitution (Mulilis et al. 1977, Ladd 1974, 1977).

Water pluviation was chosen as the preferred specimen reconstitution technique, after a comparison of the behaviour of sands reconstituted by different techniques, with the behaviour of the in-situ frozen undisturbed sands. Boiled, de-aired saturated sand slurry is deposited in the membrane lined cavity filled with de-aired water during pluviation. The sand does not come in contact with air during the deposition process, and this results in well saturated specimens. Sand was deposited to a few millimetres above the target specimen height, and then the excess was siphoned off to form a level surface in order to reduce the bedding errors. Details of the technique of water pluviation to reconstitute saturated sand specimens for use in triaxial tests is given in Chern (1981). The technique adapted in simple shear tests is similar to that described by Sivathayalan (1994), and that in hollow cylinder tests is detailed in Sayao (1989). Samples were prepared in the loosest state, and higher initial densities, if needed, were obtained by applying low frequency vibrations to the base of the apparatus, while the specimen was confined under a small seating load.

3.4.2 Specimen setup and consolidation

All triaxial specimens were confined by an isotropic effective stress of about 20 kPa (zero cell pressure and 20 kPa vacuum) when mounted in the apparatus. Saturation of the specimens was ensured by requiring a Skempton's value of at least 0.99 in all tests on reconstituted specimens

and most of the undisturbed specimens. A few undisturbed specimens were tested at the in-situ saturation levels (Vaid et al. 1996). After ensuring full saturation the vertical effective stress was increased to reach the required stress ratio of $K_c = 2$. The specimens were consolidated along the constant $K_c = 2$ path from this initial state.

The simple shear specimen was reconstituted in place and after siphoning off the excess sand, the specimen was confined by a vertical effective stress of about 20 kPa. Specimens were consolidated to the desired effective vertical stress, by increasing the pressure in the vertical loading piston. Consolidation in simple shear is essentially K_0 on account of the lateral restraint. The availability of compression time history in real time was used to determine the end of consolidation. The vertical movement clamp was tightened at the completion of consolidation, prior to undrained shear, in order to enforce constant volume conditions.

The HCT specimen was confined by about a 25 kPa vacuum at the end of reconstitution. Internal and external chambers are then filled with de-aired water. Sufficient volume of water was allowed to flow through the inner chamber in order to minimize the volume of air trapped in the inner chamber. A cell pressure of about 30 kPa was applied to the undrained specimen prior to checking saturation. Skempton's B value was checked by applying isotropic stress increments and a B value of at least 0.99 was insisted in all undrained tests. A back pressure of 200 kPa, and total mean normal stress of 400 kPa were used in all hollow cylinder torsional shear tests.

Following saturation, the specimen was consolidated along a predetermined stress path to the desired stress state. Consolidation stress parameters were defined in terms of mean normal stress σ_{mc} , inclination of major principal stress to vertical α_{c} , intermediate stress parameter b_{c} and

effective principal stress ratio $K_c = \sigma'_1/\sigma'_3$. The control program applies the required stresses at desired increments of mean normal stress. Upon reaching the target consolidation stress state, the specimen is allowed sufficient time (about one hour for Fraser River sand) for all drainage to be complete. The real time graphical display of the consolidation curve ($\varepsilon_v vs$ time) facilitated in deciding adequate time for consolidation.

3.4.3 Monotonic or cyclic undrained Shear

All triaxial specimens were monotonically sheared undrained. A constant speed motor drive was used to apply displacement controlled shearing at a rate of about 12% axial strain per hour both in compression and extension tests. Liquefaction was induced by static unloading, and specimens were loaded in post liquefaction using the motor drive at the same axial strain rate. Monotonic loading in simple shear was applied by the computer controlled motor drive at a loading rate of about 20% shear strain per hour. The capability to impose both positive and negative shear strains enabled shearing with unload-reload loops. The data acquisition and control program automatically applies a predetermined loading sequence to the specimen. Data was acquired at about 100 sets per second and saved as necessary, depending on the state of the test. All monotonic tests were performed in the strain controlled mode. In contrast, all cyclic simple shear tests were carried out using stress controlled loading. A sinusoidal, constant amplitude cyclic shear stress at a frequency of 0.1Hz was applied to the specimen until the sand liquefied. Upon liquefaction by stress controlled cyclic loading, the motor drive was connected

to the pneumatic piston to enable strain controlled loading. This enabled initiation of post liquefaction monotonic loading at the residual conditions following liquefaction with minimal disturbance. All transducers were monitored continually, and data were saved to file at regular intervals.

The shearing phase of the hollow cylinder tests is fully automated using feedback control. The desired stress/strain path is input to the control program. Displacement controlled loading was imposed using DVPCs controlled by stepper motors. The specimen can be sheared using constant rates of strain ranging from about 2% per hour to 30% per hour. A strain rate of about 6% per hour (axial or torsional) was used in the tests. Displacement controlled loading was applied in axial direction alone, torsional direction alone or simultaneously in axial and torsional direction, depending on the required stress path. Data were monitored continually and saved to a file at regular time intervals, or when significant changes occurred in any of the stresses or strains.

3.5 Experimental artifacts and measurement errors

The objective of laboratory shear tests is to measure element properties. Test results are interpreted assuming uniform states of stress and strain within the entire specimen. However, all testing apparatus contribute to some non-uniformity in stresses/strains, and it is necessary to minimize these non-uniformities. It is also vital to ensure that apparatus specimen interactions do not adversely influence the measured response.

End restraint effects are a common concern in shear tests, and they are minimized by using small porous stones, and polished anodized-aluminium pedestals. The porous stones used in the triaxial test constitute about 25% of the area of the specimen, and in the hollow cylinder torsional shear test they constitute only about 8% of the specimen area. Other possible sources of systematic errors are identified, and steps taken to eliminate/minimize them are discussed below.

3.5.1 Influence of loading system (in strain softening sands)

The stress-strain response of soils can be measured in the laboratory using either a load or displacement controlled loading mechanism. The measured response could be profoundly influenced by the loading system, depending on the degree of apparatus-specimen interaction. The influence of the loading system will be critical for strain softening specimens. Figure 3.10 illustrates comparative undrained strain softening response of essentially identical specimens of Ottawa sand ($D_{50} = 0.4 \text{ mm}$). The response under constant rate of strain loading with no inertia effects and dead weight inertial loading system recorded by an oscillographic recorder may be noted to be essentially identical. However, the non-inertial pneumatic stress controlled system with data recorded by a low frequency responding strip chart recorder yields a different measured behaviour. The measured response by the non-inertial loading system may be further modified by the rate at which the air regulator can bleed back the pressure, when reduction is required in axial load following the peak stress state. Only an inertial system, like the dead weight loading used by Castro (1969) can capture the true strain softening response of sand under stress



Figure 3.10 : The effect of loading system on the measured response in strain softening soils (after Chern 1985).

controlled loading. The stress decrease during the strain softening after the peak then occurs automatically, on account of the increasing inertia force of the accelerating dead mass, triggered by the deforming specimen. If the true material response is of the steady state type (deformation at constant stress and volume after strain softening), the mass will accelerate until a terminal constant acceleration is reached. This acceleration yields an inertia force that equals the reduction in vertical force required to reduce the peak stress to the steady state conditions. Contrary to the notion that steady state deformation in an inertial loading system occurs at constant velocity, advocated by the proponents of the steady state concepts, it in fact occurs at constant acceleration (Chern 1985, Vaid and Sivathayalan 1998).

A direct measure of the true load experienced by the specimen in the inertial loading system can only be made by an internal load transducer, located at the bottom pedestal. This indeed was done by Castro (1969). However, if the true sand response is of the QSS type, the load recorded before and until the minimum deviator stress dictated by the QSS value will again be reliable, but not after QSS, due to the impact action on account of the increasing resistance of the material with further strain. No such inertial problems arise when deformation is imposed under constant rate of strain or velocity. But, even the constant rate of strain devices may fail to continually measure the true resistance of highly collapsible sands (with meta stable structure), especially at low effective stresses. This may happen if the loading cap fails to follow the deformation of the sand as it collapses, and loses contact with the specimen. The recently reported zero shear strength in laboratory triaxial tests on sands (Ishihara 1993, Lade and Yamamuro 1997, Yamamuro et al. 1998) is apparently an incorrect interpretation of the real soil behaviour because of this apparatus-specimen interaction (Sivathayalan et al. 2000). In fact the zero strength values were indeed reported only for sands with potentially collapsible meta stable structure (moist tamped, air pluviation with fines) under strain controlled loading. No such behaviour has been recorded under the dead weight inertial loading systems for sand specimens reconstituted by the same techniques, but to much looser void ratios, even at negative relative densities (Castro 1969).

As noted earlier, all monotonic shearing tests in this study were performed using displacement controlled loading mechanisms. Displacement controlled loading ensures that the deviatoric

stress strain response of the water pluviated sand is properly measured both before and after the peak deviator stress.

3.5.2 Membrane penetration effects

The penetration of membrane into (or out of) the interstices of the peripheral voids causes a systematic error in volumetric measurements in drained tests and pore pressure measurements in the undrained tests, if the test involves changing radial effective stresses. Membrane penetration effects under changing effective confining stress were first recognized by Newland and Alley (1957, 1959) in drained triaxial tests; and since then have been of considerable interest to numerous researchers. Attempts have been made by several researchers to correct for this error, both in drained and undrained tests.

Conventional drained tests are often carried out under a constant effective confining stress and thus are not affected by membrane penetration volume changes during the shearing phase. Even if the effective confining stress is changing in a drained test (e.g drained test at constant mean normal stress), membrane penetration only affects the measured volume changes, and the correction is straight forward. The measured volume changes can be suitably corrected after the end of the test ("post test correction") with the volume changes induced by membrane penetration alone. However, membrane penetration directly influences the effective stress path in an undrained test. Membrane induced volume changes in fact amounts to a partially drained state and thus the measured stress strain response does not represent the undrained behaviour. This necessitates corrections to be made in real time during the test, if truly undrained conditions are to be maintained.

Even though it is recognized that the influence of membrane penetration decreases as the sand gradation gets finer, there is no consensus among researchers as to the particle size below which its effects become relatively insignificant. Therefore a study was undertaken to assess the influence of membrane penetration in undrained tests on Fraser River sand, which is the coarsest sand used in the subsequent test program. A series of tests was performed, first to accurately compute the magnitude of specimen specific membrane penetration, and then to assess its influence in conventional undrained tests. These tests results are presented in Chapter 4.

3.5.3 Calculation of void ratio

An accurate determination of the void ratio of the sand specimen at various stages of the test is essential in any laboratory test. A knowledge of the specific gravity of the solids, the amount of dry mass of the soil used to form the reconstituted specimen and the total volume of the specimen are needed to calculate the void ratio. Traditionally the volume of the specimen is calculated from the measured height and diameter. Specimen height can be determined quite accurately, but the accuracy of the diameter measurements is low. Vaid and Sivathayalan (1996b) pointed out that the measurement errors in dimensions can lead to considerable error in the calculated void ratio, and suggested alternative reliable methods for calculating the void ratio of both reconstituted and undisturbed sand specimens. All void ratios reported in this thesis are
calculated using volume and mass measurements, as suggested in Vaid and Sivathayalan (1996b). This method uses the volume of the membrane lined cavity over the height of the specimen to calculate the diameter. The cavity volume is accurately determined using the mass of the water required to fill the cavity. The precision of the void ratio measurement using this method is about 0.002. The initial void ratio is recorded under a confining stress of about 2 kPa in all tests. The void ratio changes are then continually monitored throughout the test, and this enables a confident measurement of the void ratio at any stage of the test.

3.6 Test program

The behaviour of reconstituted sands was assessed under triaxial, simple shear and under generalised multi axial loading conditions in the hollow cylinder test. Undisturbed specimens were tested in triaxial and simple shear only. The behaviour of specimens reconstituted to essentially identical initial states by different reconstitution techniques was compared among themselves, and to that of the undisturbed sand.

The undrained response of undisturbed in-situ frozen sands from four different sites was studied, and compared to the behaviour of equivalent specimens reconstituted in the laboratory. Post liquefaction behaviour of undisturbed sands was also assessed following liquefaction caused either by cyclic or static unloading.

The influence of inherent anisotropy and of the initial static shear on the undrained behaviour of reconstituted sands was evaluated using hollow cylinder torsional shear tests. A series of tests

on the loosest deposited sand, consolidated to different initial stress states, was performed at fixed principal stress directions. In addition, the influence of stress rotation on the behaviour and its dependency on the initial state were assessed. Details of the type of experiments performed in each category are outlined, and the results are discussed in the following chapters.

Chapter 4

Membrane Penetration Effects on Undrained Behaviour

4.1 Introduction

Most laboratory shear tests on granular soils are carried out on specimens enclosed in flexible membranes (e.g. triaxial tests). Under loading paths involving changes in effective confining stress, the penetration of the membrane into (or withdrawal out of) the interstices of the granular soil specimen causes a systematic error in the measured volume changes (in drained tests) or excess pore pressures (in undrained tests).

The effect of membrane penetration in a conventional undrained test is to induce a partially drained state as a result of the membrane penetrating into (or withdrawing out of) the peripheral voids of the soil specimen. An increase in pore pressure would cause the membrane to move outwards from the interstices as illustrated in Figure 4.1, and as a consequence the excess pore pressure generated will be smaller than that in a truly undrained test. Membrane penetration in a conventional drained test introduces an error in the measured volumetric strain, and does not influence the effective stress state in the sand. In contrast, the effective stress state is directly influenced by the membrane penetration in an undrained test. This may have a profound influence on the measured steady or quasi steady state undrained strength of the sand, depending on the





magnitude of membrane penetration. This undrained strength is used as the key parameter in steady state analysis for assessing the potential of a flow failure in a saturated sand mass. If the membrane compliance has a considerable influence on the undrained response, then steps must be taken to minimize or eliminate these effects in order to obtain its true undrained behaviour. This is of great importance in the confident assessment of both static and cyclic liquefaction potential of saturated granular materials.

In a drained test with changing effective confining stress, the measured volumetric deformation consists of both the soil skeleton volume change and the volume change due to membrane penetration. Thus, the measured volume change must be corrected for membrane volume change in order to obtain the actual volumetric strain of the soil skeleton. This is essential in the formulation or validation of constitutive models of granular materials. Clearly, the reliability of volumetric strain measurements in drained tests (or excess pore pressure and thus the shear strength, S_u in undrained tests) depends on how accurately the membrane penetration magnitude is assessed in drained tests or how its effect on pore pressure is accounted for in undrained tests.

4.2 Magnitude of membrane penetration

The systematic error caused by membrane penetration effects on the measured drained behaviour of sands is directly dependent on the magnitude of membrane penetration for a given specimen size. The magnitude of membrane penetration is often expressed in terms of the volume change per unit surface area of the membrane in contact with the specimen per 10-fold change in effective confining stress, and is called unit membrane penetration ε_m .

The volume change induced by the penetration of membrane into the interstices depends on the change in the effective confining stress and the surface area of the membrane, in addition to the magnitude of membrane penetration. The volume change induced by membrane penetration, ΔV_m will then be

$$\Delta V_{m} = \epsilon_{m} \times A_{m} \times \log\left(\frac{\sigma'_{\text{current}}}{\sigma'_{\text{initial}}}\right)$$
(4.1)

Where A_m is the surface area of the membrane and σ' is the effective confining stress. A known ε_m will enable corrections for the measured volumetric strain. Clearly, the effectiveness of any attempts to correct for membrane penetration induced errors is directly dependent on the accurate determination of its magnitude.

4.2.1 Assessment of membrane penetration

Newland and Alley (1959) were the first researchers to recognize the influence of membrane penetration, and they suggested a method of correction for membrane penetration induced volume changes by assuming material isotropy ($\varepsilon_x = \varepsilon_y = \varepsilon_z$) under hydrostatic load increments. Later, Roscoe et al. (1963) also used the same assumption. Based on this assumption the soil skeleton volumetric strain, ε_y equals three times the axial strain, ε_z ; and thus the volumetric strain recorded

in excess of $3\varepsilon_z$ is solely due to membrane penetration. This assumption has since then been shown invalid on account of inherent anisotropy in sands (Arthur and Menzies 1972, Oda, 1972, Lade and Wasif 1988). Vaid and Negussey (1984) recognized that the deformation of sands is more isotropic in hydrostatic unloading than in loading. They suggested a similar method assuming $\varepsilon_x = \varepsilon_y = \varepsilon_z$, but during hydrostatic unloading.

Since the magnitude of membrane penetration will depend on the surface area of the membrane covering the specimen, and the skeleton volume change on the total volume of the specimen, most of the subsequent attempts to measure membrane penetration were aimed at using different area to volume ratio specimens. The usage of large test specimens will reduce the effects of membrane penetration on account of the decreased area to volume ratio (Wong et al. 1975, Martin et al. 1978), but it will not eliminate them altogether. Roscoe et al. (1963) placed cylindrical dummy brass rods of the same height as the specimen, coaxially within the saturated sand. The diameters of dummy rods were changed from one sixth to eleven twelfths of the specimen diameter in order to vary the area to volume ratio, and infer the membrane induced volume changes as illustrated in Figure 4.2. Raju and Sadasivan (1974) noted that stress non-uniformities arise within the sand specimen due to the rigidity of the brass rod inclusions, and proposed annular flexible top platens to be used with dummy rods. Wu and Chang (1982) have, however, shown that the inclusion of a dummy rod within the specimen leads to the development of shear stresses and thus a nonhomogeneous stress state within the specimen, even under hydrostatic loading. They noted that the sand skeleton volumetric strains would then be not identical (a necessary requirement) among specimens with different diameter rod inclusions.

In an attempt to reduce the area of the membrane causing penetration, thus effectively changing the area to volume ratio, some researchers have used polythene strips, polyurethane coating (Raju and Venkatramana, 1980), plastic or brass plate liners (Choi and Ishibashi, 1992; Lade and Hernandez, 1977) alongside with the membranes to reduce membrane penetration. But, the usage of such stiffeners around the specimen is bound to lead to further stress non-



Figure 4.2: Evaluation of membrane penetration using dummy brass rods (After Roscoe et al. 1963)

uniformities. Choi and Ishibashi concluded that the nonuniform stresses caused by the introduction of plastic liners were the cause of the fallacious trend in their results. Alternatively, Kiekbusch and Schuppener (1977) used rubberized membranes and Lo et al. (1989) liquid rubber-coated membranes to enclose test specimens in an attempt to partially fill in the peripheral voids.

Frydman et al. (1973) used conventional triaxial and hollow cylinder specimens to assess the membrane penetration. They achieved different area to volume ratio by using hollow cylinder specimens of fixed external but varying internal diameters. Vaid and Negussey (1984) instead used triaxial specimens of different diameter to get different area to volume ratios. Both of these methods neither make any assumptions regarding the constitutive behaviour of sand nor suffer from serious stress non-uniformities within the specimen, except for minor end effects, that are inherently present in any soil testing device.

An accurate assessment of the magnitude of membrane penetration is of critical importance in any attempt to correct for its effects. The most credible among the above discussed methods are the Vaid and Negussey (1984) and Frydman et al (1973). However, they either require multiple specimens of different configurations (Vaid and Negussey 1984, Frydman et al. 1973) or assume isotropic response of soils during unloading (Vaid and Negussey 1984). The multiple specimen methods have a practical disadvantage since different configurations are not readily available. The alternative assumption of isotropic rebound may not be strictly valid for all soils. Further, these methods do not allow for specimen/membrane specific penetration correction. The most desirable method would be the one that is (i) non destructive, (ii) requires a single specimen, (iii) does not make any assumptions of the constitutive behaviour of soil, and (iv) allows for specimen specific corrections. The new method proposed later in this chapter satisfies these criteria, and uses the data obtained during the consolidation (either hydrostatic or K_0) phase in a hollow cylinder torsional test to determine the magnitude of membrane penetration.

4.2.2 Factors influencing the magnitude of membrane penetration

Several researchers have identified the factors that influence the magnitude of membrane penetration (Kiekbusch and Schuppener 1977, Ramana and Raju 1982, Seed et al. 1987, Nicholson et al 1993a). The soil grain size and its gradation have been found to be the most dominant factors that control the magnitude of membrane penetration. Mean particle size D_{50} has been used by most researchers to represent the average grain size, and the magnitude of membrane penetration has often been related to D_{50} . The influence of the relative density of the soil specimen has been investigated by several researchers (Ramana and Raju 1982, Seed et al 1989, Nicholson et al. 1993a), and all studies concur that membrane penetration magnitude decreases with denser packing of sand particles. However, all researchers consistently indicate that the influence of membrane penetration on grain angularity or fabric has been found to be negligible (Banerjee et al. 1979, Nicholson et al 1993a). Even though there is no agreement in the literature on the influence of membrane thickness on the membrane penetration magnitude, the majority of the researchers are of the opinion that the effect is only nominal for typical

membrane thicknesses (Kiekbusch and Schuppener 1977, Ramana and Raju 1982, Seed et al 1989, Nicholson et al. 1993a) used in laboratory tests.

4.2.3 Corrections in drained and undrained Tests

Membrane penetration induces a systematic error only when the effective confining stress changes during the test. Conventional drained tests are often carried out under a constant effective confining stress and thus are not affected by membrane penetration. Even under changing effective confining stresses during a drained test, membrane penetration only affects the measured volume changes, and the correction is straight forward. The measured volume changes can be suitably corrected after the end of the test ("post test correction") using the volume changes induced by membrane penetration.

Membrane penetration directly influences the effective stress path in an undrained test. Membrane induced volume changes induce a partially drained state and thus intrinsically affect the measured stress strain response. This necessitates corrections to be made in real time during the undrained test. Raju and Venkatramana (1980) and Kramer and Sivaneswaran (1989) compensated for the potential volume changes induced by membrane penetration by manually injecting water into (or withdrawing out of) the specimen. The injection systems have since been improved dramatically using computer controlled feedback methods (Nicholson et al. 1993b, Wijewickreme et al. 1994). Such a real time correction procedure can ensure that essentially undrained conditions prevail during the duration of the test. Numerical methods have also been proposed to correct the measured pore pressure to obtain pore pressure under truly undrained condition (Baldi and Nova 1984, Tokimatsu and Nakamura 1987, Ansal and Erken 1996). These methods require assumptions regarding constitutive relations of the sand in order to calculate the pore pressure changes from the membrane induced volume changes. These post correction methods need to be experimentally validated before they can be confidently used. No such experimental validation of these methods could be found in the literature. The applicability of these post correction methods to yield the truly undrained behaviour of sands is herein evaluated based directly on conventional and truly undrained tests on two sands; one coarse and one fine.

4.3 A New Method for determining the magnitude of membrane penetration

The proposed new method uses a single hollow cylindrical specimen which is loaded hydrostatically under drained conditions. The membrane penetration correction is obtained from the measured volumetric strain of the specimen and that of the inner cavity under hydrostatic load increment/decrement together with the measured axial strain. As discussed in Chapter 3, there are four strain components ε_{z} , ε_r , ε_{θ} and $\gamma_{z\theta}$ in a hollow cylinder test, that are functions of the measured axial and torsional deformations and the volume changes of the specimen and of the inner cavity (Vaid et al. 1990b). The special design of the UBC hollow cylinder torsional shear apparatus minimizes the radial end restraint effects by the use of highly polished radially ribbed anodized aluminum end platens with porous stones that cover only 10% of the platen area as illustrated in Figure 3.5. As a result, the developed radial shear stresses are insignificant under hydrostatic loading, and the normal stress at a given point in the specimen remain the same in all directions, thus closely satisfying the conditions of an element test.

The force equilibrium of an element in the hollow cylinder wall (Figure 4.3) in the radial direction yields,

$$\sigma_r - \sigma_{\theta} + r \times \frac{d\sigma}{dr} = 0$$
 (4.2)

where σ_r and σ_{θ} are the normal radial and tangential stresses respectively. If u is the radial displacement of an element at radius *r* then consideration of the axisymmetric deformation of the cylinder yields the radial and tangential strains as



(a) : Stresses acting on an element



(b) : Displacement in horizontal plane

Figure 4.3 : Stresses and strains in a hollow cylindrical element

$$\epsilon_{\theta} = \frac{u}{r}$$

$$\epsilon_{r} = \frac{du}{dr}$$
(4.3)

As Saada (1988) pointed out, when identical pressures act inside and outside the hollow cylinder, it is quite proper to infer that there is no variation in the value of normal radial stress σ_r across the wall. Thus, under hydrostatic or K_o consolidation, radial and tangential stresses are equal throughout the specimen and the whole specimen can be considered an element. As a consequence the tangential strain will also be identical at every radius across the thickness of the wall in a homogeneous specimen. The membrane penetration correction is obtained by enforcing the condition of this equality of tangential strains at the internal and external radii.

$$\epsilon_{\theta \text{ at } r=R_{i}} = \epsilon_{\theta \text{ at } r=R_{e}}$$

$$\therefore \qquad \frac{\Delta R_{i}}{R_{i}} = \frac{\Delta R_{e}}{R_{e}} \qquad (4.4)$$

The inner cavity volume V_{IC} of the cylindrical specimen of height h and radius R_i is given by $V_{ic} = \pi R_i^2$ h, and

$$\delta V_{IC} = \frac{\partial V_{IC}}{\partial h} \cdot \delta h + \frac{\partial V_{IC}}{\partial R_i} \cdot \delta R_i$$

$$\delta V_{IC} = \pi \cdot R_i^2 \cdot \delta h + 2\pi \cdot R_i \cdot \delta R_i$$
(4.5)

Dividing by V_{IC} and rearranging gives,

$$\frac{2.\delta R_i}{R_i} = \frac{\delta V_{IC}}{V_{IC}} - \frac{\delta h}{h}$$
(4.6)

Similarly, consideration of the total volume of the specimen V and of the inner cavity together vields,

$$\frac{2.\delta R_e}{R_e} = \frac{\delta V_{IC} + \delta V}{V_{IC} + V} - \frac{\delta h}{h}$$
(4.7)

The strain criterion given in equation 4.4 together with equations 4.6 and 4.7 now leads to

$$\frac{\delta V_{IC}}{V_{IC}} = \frac{\delta V_{IC} + \delta V}{V_{IC} + V}$$

$$\frac{V_{IC} + V}{V_{IC}} \cdot \delta V_{IC} = \delta V_{IC} + \delta V$$

$$\frac{\delta V_{IC}}{\delta V} = (\chi^2 - 1)$$
(4.8)

where χ is ratio between the external and internal radii of the specimen. Equation 4.8 relates the actual volume change of the specimen to that of the inner cavity under hydrostatic loading. It indicates that the ratio of the true volume change of the specimen to the true volume change of the inner cavity is uniquely related to the internal and external radii. The measured volume changes, however, are subject to systematic errors caused by membrane penetration of both the inner and outer membranes. The specimen volume change is affected by both the inner and outer membranes, and the inner chamber volume change is affected only by the inner membrane, as illustrated in Figure 4.4. If δV_{IR} and δV_{R} respectively are the recorded volume changes of the inner cavity and of the specimen, and δV_{IM} and δV_{OM} the volume changes induced by inner and outer membrane penetrations, then

$$\delta V_{IC} = \delta V_{IR} + \delta V_{IM} \delta V = \delta V_R - \delta V_{IM} - \delta V_{OM}$$
(4.9)

The membrane penetration induced volume change per unit area of the membrane $\overline{\epsilon}_m$, for a given effective stress increment is given by

$$\overline{\epsilon}_{m} = \epsilon_{m} \times \log\left(\frac{\sigma_{initial} + \Delta\sigma_{increment}}{\sigma_{initial}}\right)$$
(4.10)



(a) : Membrane Penetration into the peripheral voids





and the volume changes induced by the penetration of the inner and outer membranes are given by,

$$\delta V_{IM} = \overline{\epsilon}_m \cdot A_{IM}$$

$$\delta V_{OM} = \overline{\epsilon}_m \cdot A_{OM}$$
(4.11)

where A_{IM} and A_{OM} are the surface areas of the specimen covered by the inner and outer membranes respectively. Combining equations 4.8, 4.9 and 4.11 yields the following expression for each effective stress increment.

$$\overline{\epsilon}_{m} = \frac{\Delta V_{R} - \Delta V_{IR} \cdot (\chi^{2} - 1)}{\chi \cdot A_{mem}}$$
(4.12)

Equation 4.12 is used to calculate the membrane penetration correction from the recorded volume changes of the inner cavity and of the specimen together with the measured axial deformation for each axisymmetric stress increment. The unit membrane penetration correction ε_m can then be obtained from a series of such axisymmetric effective stress increments.

4.3.1 Characteristics of the new method

The new method proposed for the evaluation of the magnitude of membrane penetration has several advantages over the existing methods. It does not require any assumptions regarding the constitutive behaviour of sands. Further, the sand is often consolidated axisymmetrically (most common consolidation stress state being $\alpha_{\sigma} = 0$; b = 0) prior to drained or undrained shearing. The volume change data obtained during this consolidation phase can then be utilized to obtain specimen specific unit membrane penetration. This enables the effects of specimen density, membrane thickness and fabric, if any, on membrane penetration to be accounted for. The specimen specific and non destructive nature of this method are not applicable if the consolidation stress state is non-axisymmetric, but such states are not very common in research studies.

Hollow cylinder tests are often subject to criticism on account of the nonuniform stresses that develop across the wall of the specimen. But these occur only when different internal and external pressures are used and/or when the specimen is subjected to torsional shear. Nonuniform stresses are also caused by the end restraint effects that are inherently present in any soil testing device. As already pointed out in chapter 3, these were minimized by a careful design of the apparatus (Vaid et al. 1990b). In the new method suggested, the stress state throughout the specimen is, therefore, closest to hydrostatic (equal internal and external pressures; no shear stress), making it an element test, that virtually guarantees uniform stresses and strains throughout the specimen.

It is interesting to note that the condition of constant tangential strain across the wall implies constant radial strain, and $\varepsilon_{\theta} = \varepsilon_{r}$ across the thickness of the wall, regardless of any anisotropy in the granular material. Such a kinematic condition is imposed as a result of the hydrostatic loading and the axisymmetric geometry of the specimen. In any case, for alluvial deposits formed under gravity (or specimens deposited vertically) the horizontal plane is normally the plane of deposition, and the existence of isotropy has been demonstrated in the horizontal plane in true triaxial tests (Yamada and Ishihara, 1979; Lade and Duncan, 1973). The vertical strain ε_{z} is usually smaller than the radial or tangential strains measured in the horizontal plane on account of this inherent anisotropy even when the loading is hydrostatic (Ladd et al. 1977, Oda, 1972). Existence of such a cross anisotropic structure has also been reported by Stokoe et al. (1991) based on horizontal direction independence of measured P-wave velocities.

4.3.2 Experimental investigations (using the new method)

Membrane penetration induced volume changes were determined for Fraser River sand, that has a $D_{50} = 0.27$ mm, using the hollow cylinder apparatus. These tests were also carried out on a coarser Silica Sand, (herein called Sand A) with $D_{50} = 0.90$ mm and on uniform glass spheres of various diameters (D_{50} ranging from about 0.10 to 2.00 mm). This enabled quantification of membrane penetration as a function of D_{50} . The gradations of the materials used are illustrated in Figure 4.5. Specimens were formed in the UBC HCT device described in the previous chapter using 0.3 mm thick membranes on either side of the hollow cylindrical specimen. Studies by several researchers (e.g., Kiekbusch and Schuppener, 1977, Martin et al. 1978) demonstrate that a change in the thickness of the membranes does not significantly influence the membrane penetration induced volume change.

Specimens were formed by water pluviation, in order to simulate the alluvial deposition process. As noted earlier in chapter 3, this deposition technique leads to uniform specimens with a horizontal planes of deposition. All tests were carried out on the loosest deposited specimens. As noted earlier, the effect of density on membrane penetration has been found to be relatively



Figure 4.5 : Particle gradation of the materials used in assessing the magnitude of membrane penetration

small in comparison to that of the mean grain size (Frydman et al. 1973, Ramana and Raju 1982, Nicholson et al. 1993a). Specimens were hydrostatically consolidated in stages up to an effective stress of 500 kPa. They were then unloaded hydrostatically in several stages down to an effective stress of 15 kPa. The height and volume changes of the specimen and of the inner cavity were recorded under each applied hydrostatic effective stress increment.

4.3.3 Test Results

Results of drained tests on Fraser River sand, Sand A and glass beads are presented in this section to evaluate the magnitude of membrane penetration. Unit membrane penetration was calculated from the measured volume changes of the sample and that of the inner cavity using the equations derived earlier. Figure 4.6 shows the relationship between unit membrane penetration and effective confining stress for the two sands and glass beads of different sizes. It may be noted that there is very little scatter in the data and the linear semi-logarithmic relationship between unit membrane penetration and confining stress holds very well. In order to assess whether the membrane penetration is reversible, specimens were subjected to several load-unload-reload cycles.

Figure 4.7 shows the relationship of unit membrane penetration with confining stress during several such cycles for glass bead B ($D_{50} = 0.72$ mm). The data obtained during different cycles match excellently. This indicates the very high level of test repeatability, and that the membrane penetration effects are identical for increasing and decreasing effective stresses. Similar consistent data confirming excellent repeatability of unit membrane penetration for increasing and decreasing effective confining stresses were obtained for several other glass bead sizes. This essentially constant value of unit membrane penetration among several unload-reload cycles is a reflection of the fact that membrane penetration is not influenced by the fabric changes, if any, that occur due to the hydrostatic unload-reload cycles.



Figure 4.6 : Variation of membrane penetration induced volume change with effective confining stress.

The unit membrane penetration of Fraser River Sand and glass beads B using the single specimen method suggested by Vaid and Negussey (1984) were also determined for comparison with the new method. The results in Figure 4.8 show that though the average slopes of the semi linear log relationships are not that different, there is much larger scatter (and possibly a curvature) in the data implied by the single specimen method of Vaid and Negussey. Lin and Selig (1987) noted a similar curvature, which was more significant in the low confining stress



Figure 4.7 : Magnitude of membrane penetration during several unload-reload loops range. But the test data interpreted by the proposed new method shows no curvature, even down to confining pressures as low as 15 kPa. The non linearity in data interpreted from the hydrostatic unloading in triaxial tests apparently stems from the fact that the sand may not necessarily rebound isotropically as assumed. Thus the curvature at low stress levels in other interpretative methods should not be considered as a fundamental characteristics of membrane penetration, but rather a manifestation of the invalidity of the assumption of isotropy during unloading.



Figure 4.8: Comparison of the magnitude of membrane penetration calculated using the new method and Vaid and Negussey (1984)

The dependence of normalized unit membrane penetration S (slope of ε_m vs log p) on particle size has often been expressed in a semi log chart by several researchers (Frydman et al. 1973; Ramana and Raju 1982, Nicholson et al. 1993a). As such, Figure 4.9(a) illustrates this dependence on the logarithm of D₅₀ and also on D₅₀ in an alternative linear plot. Normalized unit membrane penetration data from previous studies reported in the literature are also shown in Figure 4.9(a). It can be seen that all available data essentially falls within a narrow band for mean



Figure 4.9(a) : Dependence of normalised unit membrane penetration on the logarithm of D_{50}

particle sizes ranging from about 0.1 mm to 1.0 mm even though different techniques and assumptions have been used to obtain these data. This implies that the previously suggested methods, in spite of some theoretical concerns, may provide reasonably good estimates of membrane penetration for fine to medium sands when compared to the proposed method, which neither makes any assumption regarding material behaviour nor is subject to questionable stress non-uniformities within the specimen during hydrostatic loading. A linear unit membrane

penetration vs D_{50} appears to be a better approximation of the data than the semi-log plot. Plots on linear scale as in Figure 4.9(b) should, therefore, be preferred over the semi-log relationships on account of their simplicity.

4.4 Membrane penetration and undrained behaviour

The effective confining stress in an undrained test continuously changes on account of excess pore water pressure development. As a result the membrane may withdraw out of the interstices (during increasing pore water pressure) or penetrate into them (during decreasing pore water pressure) even in a conventional test with constant total radial stress. This is equivalent to partial drainage, and would invariably influence the developed excess pore water pressure and the effective stress state in the specimen.

4.4.1 Truly undrained behaviour of sands

Compensating the potential volume changes induced by membrane penetration either by injecting water into (or withdrawing out of) the specimen will ensure a truly undrained state. An undrained test is termed truly undrained, if the volume changes caused by the membrane penetration in a conventional undrained test are compensated for. Raju and Venkatramana (1980) and Kramer and Sivaneswaran (1989) manually injected water into the specimen during undrained tests. Nicholson et al. (1993b) recognized the limitations of this manual injection system and



Figure 4.9(b) : Variation of normalised unit membrane penetration with mean grain size, D_{50}

developed a computer controlled system to automate the process. Since then, the injection systems have improved dramatically with the use of computer controlled feedback systems. The pore water injection system with a proper computer control can ensure truly undrained conditions throughout the test by continually feeding water in (or taking it out of) the specimen in order to prevent partial drainage that would otherwise occur on account of the penetration of membrane into (or its withdrawal out of) the peripheral voids.

4.4.2 Experimental investigations

Conventional and truly undrained tests were performed on Fraser River sand and Sand A to assess the influence of membrane penetration on their undrained behaviour, and its dependency on average particle size. These tests were also used to evaluate the current analytical "post correction" methods.

A computer controlled feedback injection system (Wijewickreme et al. 1994) was used to compensate for membrane induced volume changes in order to yield the truly undrained response for its direct comparison with the conventional undrained behaviour. Figure 3.6 illustrates the schematic arrangement of the pore water drainage system in truly undrained tests. Pore water pressure is measured at the bottom of the specimen, and the membrane induced volume changes are compensated by injecting water through the top drainage line using the computer controlled DVPC. Both conventional and truly undrained tests were performed at an axial strain rate of about 6% per hour. This ensured that the velocity head during injection was negligibly small, and the injection process does not inadvertently influence the measured pore pressure.

Truly undrained triaxial compression tests were carried out by using the unit membrane penetration values obtained by the suggested new method. All tests were performed on saturated specimens with a Skempton B value better than 0.99. The volume of the pore water was continuously adjusted so as to maintain truly undrained state (within a volumetric strain of 2×10^{-6}) throughout the test. The excellent continuity in pore pressure - strain relationship in Figure 4.10 showing the amount of water injected (or removed) during a typical test and the measured

excess pore pressure is an indication of the high reliability of the injection process. A conventional undrained triaxial compression test was then carried out on an essentially identical specimen for comparison with the observed truly undrained behaviour. Material parameters required for the analytical prediction procedures of truly undrained behaviour were also determined as needed.



Figure 4.10 : Excess pore pressure and the volume of water injected or withdrawn to maintain a truly undrained state in a typical test

4.4.3 Post test correction to pore pressure in conventional undrained shear

The analytical school of thought has proposed corrections to the measured pore pressure in a conventional test in order to infer the pore pressure which would have been developed under truly undrained conditions. Such methods do appear very appealing, as they may allow resurrecting vast amounts of available undrained test data in the literature. However, direct experimental validation of such analytical methods is essential for their confident use. These correction procedures essentially involve translating the volumetric strain caused by the membrane penetration in the conventional drained test into an equivalent pore pressure in an undrained test by assuming a certain volumetric constitutive behaviour of the granular skeleton. The validity of such models, however, has never been experimentally verified. The analytical method proposed by Martin et al. (1978) to correct for system compliance in undrained simple shear tests was later adopted by Tokimatsu and Nakamura (1987) to correct pore pressures for the membrane penetration effects under undrained triaxial loading. They showed that the excess pore pressure in a truly undrained test can be related to the excess pore pressure measured in a conventional test by

$$\frac{\Delta U_{\text{truly undrained}}}{\Delta U_{\text{conventional}}} = 1 + C_R$$
(4.13)

in which ε_{vm} and ε_v are the volumetric strains due respectively to membrane penetration and of the soil skeleton rebound and $C_R = d\varepsilon_{vm}/d\varepsilon_v$.

Baldi and Nova (1984) also suggested a similar correction procedure based on a model regarding compressibility of the sand skeleton. The method suggested by Ansal and Erken (1996) is essentially identical to that suggested by Baldi and Nova. They both obtain the pore pressure correction by multiplying the bulk modulus of the sand skeleton by the volumetric strain induced by membrane penetration. Yet, Figure 4.11 (reproduced from Figure 14 in Ansal and Erken 1996) shows a considerable difference in the pore pressures predicted in a truly undrained test by the two methods. The measured maximum positive excess pore pressure in the conventional test



Figure 4.11: Comparison of measured and predicted excess pore pressures (After Ansal and Erken 1996).

was about 135 kPa. The predicted maximum excess pore pressure in a truly undrained test by Baldi and Nova's correction method would be about 270 kPa and about 330 kPa by the Ansal and Erken method. The difference apparently arises on account of the different constitutive relationships adopted to relate the bulk modulus of the sand to its stress state. Baldi and Nova assume $C^*\sigma'_m$ as a function of η only, where C* is the bulk compressibility of sand and η is the ratio of deviator stress to effective mean normal stress, whereas Ansal and Erken assume C as a function of σ'_m only, and not influenced by the η level.

4.4.4 Test results

Figure 4.12 shows the response of a typical sample of Fraser River sand measured in truly undrained and conventional undrained compression. The maximum excess pore pressure developed in the truly undrained test was only about 10 kPa higher than that developed in the conventional test. The corresponding phase transformation undrained strengths ($1/2 \sigma_{d,PT}$) measured were 65 kPa and 72 kPa respectively. The friction angle mobilized at phase transformation was 33°, and 37° at maximum obliquity, in both the tests.

A comparison of the truly undrained and conventional undrained response of the loosest deposited coarser Sand A is shown in Figure 4.13. Much larger maximum excess pore pressure(152 kPa) developed in the truly undrained compared to that in the conventional undrained test (91 kPa). The specimen was marginally contractive in the conventional test, but responded in a strain softening manner when truly undrained. The phase transformation strength



Figure 4.12 : Comparison of truly undrained and conventional undrained behaviour of Fraser River sand.



Figure 4.13 : Comparison of truly undrained and conventional undrained behaviour of coarse Silica sand A.

in the truly undrained test was now less than half of that recorded in the conventional test, even though the friction angle mobilized at phase transformation was essentially constant at 27° between the two tests. The larger discrepancy between truly and conventional undrained response in this case is clearly due to the larger membrane penetration effects in the coarser Sand A than the finer Fraser River sand.

4.4.5 Evaluation of post-test correction methods

The pore pressures measured in the conventional test were also corrected by using the suggested post test correction methods discussed earlier. The test data obtained in hydrostatically consolidated conventional drained tests (constant confining stress) were used to compute the compliance factor C_R for the Tokimatsu and Nakamura's method, and the experimental program prescribed in Baldi and Nova was carried out to obtain the values of $C^*\sigma'_m$ at different values of η . The modulus C_t of Ansal and Erken method was obtained from hydrostatic compression test data.

Figure 4.14 compares the predicted truly undrained pore pressures calculated by the three proposed methods, together with those observed in an actual truly undrained test on Fraser River sand. The data from the conventional undrained test is also shown. As noted earlier the measured maximum excess pore pressures in the truly undrained test was only about 10 kPa higher for this sand ($D_{50} = 0.27$ mm; $D_r = 25\%$), whereas the post correction methods, in contrast, predict values several times larger, at 40, 60 and 82 kPa respectively for the Ansal and Erken's,



Figure 4.14 : Comparison of the excess pore pressure measured in a truly undrained test with those predicted by the post correction methods in Fraser River sand.

Baldi and Nova's and Tokimatsu and Nakamura's methods. This amounts to an over prediction by about 400% to 800%. It is important to note that the predicted excess pore pressures even exceed the initial confining stress in two of the methods, an impossible scenario, which would imply negative effective confining stresses. This phenomenon may also be noted in the results reported by Ansal and Erken. These researchers were apparently oblivious to the fact that the upper limit of excess pore pressure cannot exceed the initial confining stress. The computed
effective confining stress states which become negative is a clear indication that the corrections applied are extreme.

The predicted pore pressures similar to those for the Fraser River sand in Figure 4.14 are illustrated in Figure 4.15 for the coarser Sand A. Again, the maximum excess pore pressure predicted in this case by both the Tokimatsu and Nakamura and Ansal and Erken's methods are extremely high compared to those recorded in the truly undrained test, which again amount to impossible values, larger than the initial confining stress.

The test results presented clearly demonstrate that the available post correction methods will yield predicted pore pressure under truly undrained conditions to be too large for both the Fraser River sand ($D_{50} = 0.27$ mm) and Sand A ($D_{50} = 0.90$ mm). The data reported by Ansal and Erken for the coarser Podima sand (D_{50} of about 1.20 mm), also predict much higher truly undrained pore pressures than observed in actual truly undrained tests. The evaluated post correction methods predict truly undrained maximum excess pore pressures that are too large. Apparently, the constitutive soil models assumed in the correction procedures are not realistic for the tested sands. Figures 4.14 and 4.15 also show that the rate of pore pressure development with strain is much faster when the sand is truly undrained compared to when conventionally undrained. Such differences would lead to erroneous prediction of effective stress paths and stress strain behaviour of sand by the post-correction methods. In particular, the failure to recognize the transformation of the behaviour of Sand A from essentially strain hardening in conventional undrained shear to strain softening when truly undrained, would be err on the unconservative side in problems needing safeguard against flow failures.



Figure 4.15 : Comparison of the excess pore pressure measured in a truly undrained test with those predicted by the post correction methods in Silica sand A.

4.5 Summary

A new method for reliable estimates of membrane penetration induced volume changes using a single specimen has been developed. This non destructive method enables determination of specimen specific membrane penetration correction. In contrast with the existing single specimen methods, this method does not make any assumption regarding the constitutive behaviour or isotropy of the sand specimen, and no special measurements other than those obtained in a conventional hollow cylinder test are required. The stress and strain conditions within the HCT specimen are virtually uniform on account of the imposed hydrostatic stress state.

For a given sand, membrane penetration is found to be linearly related to the logarithm of effective stress for stresses varying from as low as 15 kPa to 500 kPa. The normalized unit membrane penetration can be better approximated as a linear function of D_{50} in preference to log D_{50} . It is also found that most of the existing methods give reasonably good estimates of normalized unit membrane penetration for particle sizes ranging from 0.10 to 1.0 mm, despite assumptions that are not strictly valid.

The maximum excess pore pressure in a truly undrained triaxial compression test was experimentally found to be larger than that under conventional undrained compression by about 7% for Fraser River sand ($D_{50} = 0.27$ mm), but a high 70% for Sand A ($D_{50} = 0.90$ mm). The currently available analytical post-correction methods predict much larger excess pore pressures, and if used in practice will lead to very conservative designs. Transformation of the strain hardening response of sand noted in conventional undrained tests into the strain softening type, if truly undrained state has persisted, is experimentally illustrated. This is a matter of great concern from a practical standpoint.

The comparison of the conventional and truly undrained tests reveal that the error induced by membrane penetration effects is quite small in Fraser River sand, that has a D_{50} of about 0.27mm. Unlike in the coarse Silica Sand A, the results of conventional undrained tests are very

representative of the truly undrained response in Fraser River sand. Therefore the membrane induced volume changes were not compensated in subsequent undrained tests on Fraser River sand reported in the following chapters. However, if any undrained tests are to be performed on Sand A, then it would be essential to compensate for membrane induced volume changes, in order to represent the true undrained behaviour.

Chapter 5

Fabric Dependent Behaviour of Sands

5.1 Introduction

The influence of fabric on the undrained behaviour of sands is highlighted in the results presented in this chapter. Fabric dependency is demonstrated using test results from static and cyclic undrained tests in triaxial and simple shear loading modes. A direct comparison of the behaviour of sand specimens prepared to the same density and effective stress state by different reconstitution techniques with that of the in-situ sands facilitates the assessment of the influence of fabric. The behaviour of in-situ sands is assessed using "undisturbed" specimens obtained by in-situ ground freezing. Undisturbed specimens were obtained from four different sites; two of them are natural sand deposits in the Fraser River Delta in British Columbia and the other two are hydraulically placed tailings sand deposits associated with the mining industry in Alberta. Reconstituted specimens were formed from bulk samples of sands obtained from these sites, and from the sand retrieved during in-situ ground freezing. The undrained triaxial and simple shear

behaviour of undisturbed specimens reveals the influence of the loading mode on the response of alluvial or hydraulically placed in-situ sands.

5.2 Undisturbed sand specimens

Ideally, undisturbed sampling should enable replication of all in-situ conditions in the laboratory. This includes stress state, void ratio, fabric and past stress history. However, a truly undisturbed specimen cannot be tested because of the additional stress history imparted to the soil element on retrieval, due to the stress release as part of the sampling process and subsequent reinstatement of in-situ stresses. Ladd and Lamb (1963) define a sample as perfect if it has not been disturbed by the boring, sampling and trimming except by in-situ stress release. Hvorslev (1949) considered a sample to be undisturbed if the disturbance is so small that the material is suitable for all laboratory tests and can be used to determine the strength, consolidation and permeability characteristics, and other properties of the soil in-situ. The emphasis is thus to minimize the disturbances during sampling and subsequent restoration of in-situ stress state.

Obtaining undisturbed samples of sands using conventional sampling methods often fails to provide satisfactory results. Conventional sampling using thin walled tubes causes disturbance during drilling, sampling, transportation, storage and setting up of the test. This irreversibly alters the stress history of the specimen sampled. Brooms (1980) and Seed et al. (1982) describe such techniques used in sampling undisturbed specimens, and discuss the causes of disturbance in detail. In-situ ground freezing, on the other hand, has the potential for yielding high quality undisturbed sand specimens if proper procedures are followed. These consist of minimizing disturbances during sampling and in-situ ground freezing.

5.2.1 Canadian Liquefaction Experiment (CANLEX)

Assessment of the behaviour of in-situ sand deposits using undisturbed specimens was undertaken as part of the Canadian liquefaction experiment (CANLEX) - a co-operative research endeavour. Several Canadian universities and industrial participants took part in this collaborative research project that involved extensive laboratory testing, in-situ testing and numerical modelling to characterise the behaviour of in-situ sands. The main objective of the CANLEX project was to assess the liquefaction susceptibility of loose sand deposits. Preliminary in-situ investigations were carried out at several sites, in order to identify sites with loose sand layers. The identified loose sand layers were then comprehensively characterised using a combination of laboratory and in-situ tests.

The behaviour in laboratory tests of undisturbed sands specimens obtained from four different sites in three phases is presented in this chapter. Phase I and III sites are located at an oil sand mine operated by Syncrude Canada Ltd., north of Fort McMurray in Alberta. Two natural sand deposits in the Fraser River Delta, just south of Vancouver were investigated in Phase II. Figure 5.1(a) shows the location of the Phase II sites in the lower mainland. Phase I and Phase III sands are similar except for the deposition environment; Both are rounded sands with about 10% fines and are predominantly quartz. The Phase II Fraser Delta sands are sub rounded with a mineral composition of 40% quartz, quartzite and chert, 11% feldspar and about 45% unstable rock



Figure 5.1(a): CANLEX phase II site locations in lower mainland, British Columbia.

Phase	Undisturbed sand	D ₅₀ (mm)	C _u	e _{max}	e _{min}
Ι	Mildred Lake sand	0.2	2.2	0.958	0.522
II	Kidd sand	0.35	2.5	1.061	0.703
	Massey sand	0.3	2.1	1.102	0.610
III	Syncrude J pit sand	0.2	2.2	0.901	0.579

Tal	ble	5.1	1:	Prope	rties	of	the	undistur	bed	sands	S
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fragments (Garrison et al. 1969). Table 5.1 lists maximum and minimum void ratio of the undisturbed sands according to ASTM D4253 (1991a) and D4254 (1991b), together with other key index properties.

5.2.2 In-situ ground freezing

In-situ ground freezing has been used for soil stabilization and ground freezing for a long time. Langer (1937) and Kollbrunner and Langer (1939) were the first to use ground freezing to facilitate sampling of sands and silts. Even though the potential of ground freezing for providing relatively undisturbed samples has been recognized for a long time, its usage did not gain widespread acceptance because of the high costs involved. The recognition that ground freezing is the only technique that can yield relatively undisturbed specimens of sands has led several researchers to refine the freezing techniques in order to improve the quality of the retrieved undisturbed specimens. The void ratio and fabric of the sand could be altered due to any volume expansion associated with pore water freezing into ice. Unlike in clayey soils, the attractive

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forces between soil particles and water molecules are extremely weak in sands. In addition, sands are highly permeable. Therefore, much less energy is required to expel pore water out of the void space than to move apart the sand grains under the confining stresses. Under appropriately slow rates of freezing, the excess volume of water is driven out ahead of the freezing front in order to accommodate the volume increase associated with water freezing into ice, without disturbing the sand particle assembly.

Yoshimi et al. (1977, 1978), Hatanaka et al (1985) provide valuable data on the practical aspects of freezing sand samples in ground. Yoshimi et al (1978) has shown that the process of freezing and subsequent thawing does not alter the static shear strength of the sand. Singh et al (1982) found that the freezing process does not induce significant volume changes, if the confining pressure is maintained and drainage not impeded. Based on cyclic loading tests on hydrostatically consolidated sand specimens, they have shown that the cyclic shear strength characteristics are not significantly altered by a "freeze-thaw" cycle.

The freezing system used at the CANLEX sites used a two inch nominal diameter steel pipe installed in the ground to the desired depth. Liquid nitrogen was fed into this freeze pipe using 1/2 inch copper tubes. Nitrogen gas was expelled through another coaxial tube after the phase change of nitrogen in the ground. Figure 5.1(b) shows a schematic of the freeze pipe systems used at the CANLEX sites. Frozen ground was cored using either a 100mm or 200mm inside diameter Cold Regions Research Engineering Laboratory (CRREL) core barrel. The frozen cores were transported to the University of Alberta (UoA) in dry ice and stored in a cold room. Triaxial and simple shear specimens were trimmed to the required size in a cold room at the University of Alberta before shipment to the University of British Columbia (UBC) in dry ice for



Figure 5.1(b) : Schematic of the freeze pipe (after Hofmann 1997).

testing. Some undisturbed specimens obtained from these sites were also tested at the University of Alberta and Laval University, but the majority of the testing was carried out at UBC. Ground freezing procedures employed at each of the CANLEX sites together with the details of coring and trimming of specimens are explained in detail elsewhere (Hofmann 1997). Table 5.2 summarizes the scope of the CANLEX laboratory testing program.

Phase	Test Type		Laboratory					
		UBC	UoA	Laval	Consulting Engineers			
Phase I	Triaxial	8	11	8	2	29		
	Simple Shear	8	_	-	-	8		
Phase II	Triaxial	21	-	12	-	33		
	Simple Shear	26	-	-	-	26		
Phase III	Triaxial	23	-	-	-	23		
	Simple Shear	13	-	-	-	13		
Total Tests (Phases I, II & III)		99	11	20	2	132		

Table 5.2: CANLEX laboratory Testing: Phase I, II & III Undisturbed sand specimens

5.2.3 Thawing and restoration of in-situ stress state

The frozen sand specimen has to be thawed and the in-situ stresses need to be reinstated prior to testing. It is crucial that the process of thawing and/or restoring the in-situ stresses does not cause excessive disturbance to the specimen. Ideally all in-situ conditions including stress state,

void ratio, fabric and stress history that prevailed prior to freezing would get restored in the laboratory in a truly undisturbed specimen. This, however, is virtually impossible. The mechanics of freezing soils is quite complex, as it deals with four phase systems *viz*. sand particles, ice, water and gaseous components. The complex physico-mechanical phenomena accompanying the freezing of soils is given by Tystovich (1952).

The most effective means of freezing sand with least disturbance is to invoke radial freezing outwards at a rate slow enough to expel the excess volume of water. The thermal conditions during thawing of the laboratory specimens would rarely facilitate such a unidirectional phase change. The specimen thaws radially inwards and axially from the pedestals during thawing. It is important to recognize that a sheer reversal of the boundary conditions during freezing will not necessarily yield the best procedure for thawing.

Several procedures have been proposed for thawing frozen sand specimens (BC Hydro 1993, Hofmann 1997, Konrad and Pouliet 1997). The method adopted herein was the method that resulted in minimal disturbance to the specimen. Because of the difficulty of directly tracking all the volume changes during thawing of the specimen, the change in its height, which can be measured with much greater confidence was considered as the most suitable index of their void ratio change.

5.2.3.1 Setting up of frozen specimens

The loading caps, porous stones and the membranes used in testing frozen specimens were kept in the refrigerator together with adequate amounts of de-aired water. The drainage reservoir was filled with cold de-aired water so that the specimen would not begin to thaw when connected to the drainage reservoir. The undisturbed specimen was removed from the dry iced storage compartment in the freezer and setup in the apparatus using the cold loading caps, porous stones and membrane. A vacuum of about -80kPa was applied through the cold water drainage reservoir, immediately after sealing the membranes to the loading caps to remove the air trapped between the specimen and the membrane and porous stones. The cell was assembled and filled with cold water, and a cell pressure of about 20kPa was applied with the drainage reservoir open to the atmosphere. The vacuum on the specimen was then released. The entire setup procedure was streamlined, and the entire process (from the removal of the specimen from the freezer to the application of cell pressure) was completed within about six minutes. Since all surfaces of the specimen contact are essentially at close to 0°C, no thawing occurred during the setup process. This was confirmed by continuously monitoring the height of the specimen, which remained essentially constant during the setup process. The specimen was then allowed to thaw under a controlled room temperature of 23°C for at least 18 hours with both top and bottom drainage lines open. The height change of the specimen from the beginning of setup to the end of thaw was recorded. Following thawing, the specimen was consolidated to the anticipated in-situ stress state. In-situ vertical stress was calculated from the unit weight of the sand and the water table depth, and the horizontal stress was calculated assuming a K_o value of 0.5. The recorded volume and height changes during consolidation were considered indicative of the disturbance of the undisturbed specimen.

5.2.3.2 Disturbance during thawing & reconsolidation

Ideally, the restoration of the in-situ state, by the thawing and the subsequent reconsolidation of the undisturbed frozen specimen should not cause any void ratio change. This, however, is impossible to achieve because of the unavoidable minor disturbances an in-situ frozen specimen is bound to experience during sampling and subsequent storage prior to testing. The in-situ stress state is normally anisotropic in natural fluvial and hydraulic fill deposits, with vertical effective stress generally higher than the horizontal. Ideally the stress state should remain unaltered in the soil element after freezing, except that the pore water would have been transformed into pore ice. However, tensile stresses would develop in ice when the specimen is removed from the ground, similar to the development of negative pore water pressure when sampling a saturated clay specimen. In contrast to pore water, ice can carry some shear stresses, at least temporarily in the early stages of sampling (Ladanyi 1981). However, ice has an extreme propensity to creep under shear and therefore cannot sustain large shear stresses for long periods of time (McClung 1998, Voitkovsky 1967). Thus, the stress state in the retrieved frozen specimen will approach a hydrostatic state as ice creeps. Since undisturbed specimens were in storage for several weeks prior to testing, the stress state prior to thawing would have been virtually isotropic due to the inevitable unloading of the in-situ shear stresses.

An accurate determination of the in-situ lateral stress is quite difficult. The coefficient of earth pressure at rest K_0 is estimated to be about 0.5 at all four sites using pressuremeter tests (Hughes 1996), and consequently undisturbed specimens were re-consolidated to a K_0 of 0.5. Any deviations from the actual in-situ value of the lateral stress would inevitably result in some

disturbance causing alterations of the void ratio. Singh et al (1982) have shown that freeze - thaw cycles do cause insignificant volume changes, but their evidence was confined to initially hydrostatically consolidated specimens only. It is essential to recognize that their evidence cannot be translated to soils frozen under an initially anisotropic stress state.

As pointed out earlier, the void ratio change during thawing and reconsolidation to the in-situ stresses would be the best index of the degree of disturbance in the undisturbed specimen. However, the measured volume change during thawing consists of several components and that corresponding to the volumetric strain of the specimen cannot be easily separated out. The measured volume change includes (1) the volume intake due to ice thawing into water, (2) volume change of the specimen and (3) the high volume compliance of the system because the specimen is set up in air. Thus a confident measure of the void ratio change during thaw is difficult to achieve unless some unverifiable assumptions are made to calculate the volume change of the specimen. The height change during thaw, on the other hand, can be confidently measured using a dial gauge positioned along the loading ram connected to the loading cap of the specimen. The height changes are therefore regarded as a more reliable substitute index of void ratio changes.

The measured net height changes, during thawing and reconsolidation of specimens, indicate relatively larger disturbance in Phase I Mildred Lake sand. The average axial strain during the restoration of in-situ stresses was about 1.5% in this phase. The height changes in both Phase II sands amounted to an equivalent average axial strain of about 0.14%. Unlike Phase I Syncrude sand, restoration of in-situ stress state in Phase III Syncrude sand also resulted in small height changes. These differences are believed to arise on account of the initial degree of saturation of

the in-situ sands. B- value measurements at the end of thaw under 20 kPa effective stress, but at the in-situ level of the pore pressure revealed that Phase I Mildred Lake sand was unsaturated. The degree of in-situ saturation of Phase I sand was much smaller than the Phase III sand. The unsaturated Phase I specimens were saturated by applying differential vacuum. Both the pore space and the cell pressure chamber were subjected to negative (relative to atmospheric) pressure. The effective stress on the specimen was maintained within the range of 20 to 25 kPa during the application of differential vacuum. It is believed that the large height and thus void ratio changes in Phase I specimens occur in part due to freezing in the unsaturated state and in part due to the saturation process.

The B-value measurements of Kidd and Massey sands established that both Phase II sands were fully saturated. B-value of better than 0.99 were recorded in these sands. The Phase III Syncrude sand gave B-values in the range of 0.92 to 1 with an average of about 0.98. Thus both Phase II and Phase III sands were tested without subjecting them to any saturation process, in contrast to Phase I sands.

The total height change during thaw and reconsolidation of the undisturbed specimen as a function of the degree of saturation is shown in Figure 5.2. Data are shown for both undisturbed specimens, and specimens frozen and thawed under controlled conditions in the laboratory. At a given degree of saturation, the height change in the undisturbed specimens may be noted to be higher than those that were subjected to a freeze-thaw cycle in the laboratory. This suggests that the in-situ sampling process is imparting some disturbance to the sands. It can also be noted that Phase I sand, which is essentially identical to the Phase III sand with regard to the mineral composition and gradation, underwent much larger height change, and therefore disturbance,



Figure 5.2 : Height change during thaw and reconsolidation of undisturbed and reconstituted sands

apparently on account of its low in-situ degree of saturation. Fully saturated specimens underwent minimal height changes. This data supports the contention that partial saturation might be a cause of the disturbance, and suggests that freezing is more suitable for sampling fully saturated sands. When partially saturated soils are frozen, gas/air pockets are likely to exist within the frozen specimen, which might cause collapse during thawing. Since partial saturation appears to influence the degree of disturbance in sampling sands by in-situ ground freezing, the height changes of the unsaturated Phase I sand specimens are not considered indicative of the quality of the freezing technique in assessing the liquefaction susceptibility of sands.

Figure 5.3 illustrates the height change during thaw and subsequent reconsolidation of fully saturated Massey sand triaxial specimens. Since the effective stress during thaw was smaller than the in-situ stresses, the specimens were expected to rebound slightly, and all specimens did rebound. This could be viewed as an indication that the undisturbed specimens are of high quality. The subsequent application of the in-situ stresses will cause compression, and in the ideal case, the magnitude of the initial rebound would be approximately equal to the compression during reconsolidation, because hysteresis during a small unload/reload cycle is small. The very small overall height change is an indication that relatively undisturbed samples of saturated sands can be obtained using in-situ ground freezing, and by allowing them to thaw under a small effective stress prior to reinstating in-situ stresses.



Figure 5.3 : Height change during thaw and reconsolidation of saturated Massey sands.

5.2.3.3 Comparison of different thawing techniques

The effective stress state during thaw was maintained hydrostatic, at about 20 kPa in all tests reported herein. This methodology was developed during an extensive testing program of frozen sand specimens for seismic assessment of the stability of sands in the foundation of Duncan Dam in British Columbia (BC Hydro, 1993). Other techniques proposed in the literature (Konrad and Pouliet 1997, Hofmann 1997) suggest thawing the specimen under in-situ stresses and pore

pressure. It is contended that thawing specimens under in-situ stress levels will cause lesser disturbance than thawing under a small hydrostatic effective stress and subsequently reconsolidating to the in-situ stress state (Hofmann 1997). The merits and shortcoming of these two methods are evaluated by examining the measured height change data.

Comparative specimens of Phase I Syncrude sand were thawed both under a small hydrostatic effective confining stress (Method I) and under estimated in-situ stresses (Method II). Specimens thawed by Method I were subsequently reconsolidated to the estimated in-situ stress state. A summary of the total height changes, from the initial frozen state to that after the restoration of the in-situ stresses is given for both methods in Table 5.3. Both undisturbed in-situ frozen Mildred Lake sand specimens, and specimens reconstituted in the laboratory and then unidirectionally frozen were used in these studies. Only those specimens identified as good quality (Hofmann 1997) are included in the data.

As noted earlier, the actual void ratio change of a specimen during thawing and the restoration of in-situ effective stresses cannot be measured without resorting to some assumptions. The height change measurements, on the other hand, can be reliably measured, and used as an alternative index of the void ratio changes. In this light, the data in Table 5.3 indicates that the specimens thawed under a small hydrostatic effective stress and then reconsolidated to in-situ stresses undergo lesser disturbance than those thawed under high in-situ stresses. Specimens reconstituted in the laboratory were well saturated compared to the undisturbed in-situ frozen sand specimens. The in-situ frozen specimens undergo much larger height change in both methods, partly due to the degree of saturation and partly due to the disturbances caused by sampling in-situ.

	Met	thod I	Method II		
	undisturbed reconstituted		undisturbed	reconstituted	
No of Specimens	8	61	52	22	
Axial Deformation (mm)					
Minimum	1.81	0.06	0.70	0.50	
Maximum	3.49	0.29	8.65	0.86	
Average	2.06	0.20	2.85	0.68	

I dolo bibi i toight onding o duining tobiol attoin of the black of a just and be	Table 5.3: Height ch	nange during resto	ration of in-situ	state of S	yncrude	e sands
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¹ part of the data from Hofmann (1997), ² data from Hofmann (1997)

Frozen specimens were allowed to thaw under room temperature in Method I, and in Method II, warm glycol solution was circulated through the end platens in an attempt to simulate unidirectional thaw, as the inverse of the unidirectional freezing process in-situ. An increase in the rate of axial thawing would be achieved by circulating warm liquids through the end platens. However, thawing in the radial direction cannot be completely prevented because of the temperature differences between the cell fluid and the soil specimen. Thus, unidirectional thawing is only an idealized scenario, and in reality the specimens would thaw radially inwards as well as axially from the end platens. As a result, a frozen sand core, that continually decreases in size, would exist in the middle of the specimen surrounded by the thawed sand. Such a rigid inclusion would induce shear stresses in the specimen, even under hydrostatic boundary stresses. Consequently a non-uniform stress distribution will occur within the specimen. The severity of this non uniformity will be greatly enhanced, if thawing occurs under large non hydrostatic stresses. In contrast, the magnitude of these non uniform stresses would be small if the specimen

is thawed under small hydrostatic stresses as in Method I. The influence of these small nonuniformities on the measured mechanical behaviour would diminish after the specimen is subsequently reconsolidated to the higher in-situ stresses. Since specimens are not reconsolidated to higher stresses in Method II, the non-uniform stress history will persist, and could influence the measured response to an unknown extent.

Numerical simulations of the thawing process using FLAC (1992) by Byrne (1998) indicates very high non uniform stress distributions if the specimens are thawed under anisotropic in-situ stresses, rather than under a small hydrostatic stresses. Analytical simulations also show that shear strains developed during thawing by Method II to be an order of magnitude higher than those developed in Method I (Hofmann 1997). Even though simplifying assumptions were made in the analyses regarding the complex constitutive relations of frozen soils, these results support the experimental observations reported in Table 5.3.

It has been argued, based on the data in Table 5.3, that Method II causes the least disturbance while thawing frozen specimens (Hofmann 1997). The disturbance quantified by the void ratio change, computed by assuming an arbitrary and questionable radial to axial strain ratio, was used as the index instead of the measured axial strain. Radial strain was assumed to be twice the axial strain for Method I, but zero for Method II. It was believed that the presumed unidirectional thawing of specimens in Method II would not induce any radial strains. As noted earlier, unidirectional thawing is only an idealized scenario, and even in the theorized case of unidirectional thawing, the circumferential stress boundary of the specimen, with flexible membrane cannot also be deemed to be a strain boundary at the same time.

Figure 5.4 shows the measured axial and radial strains during reconsolidation (along the K_o line) of all undisturbed specimens thawed by Method I. The radial strain during consolidation is consistently smaller than the axial strain during K_o consolidation. The ratio of radial strain to axial strain, $\varepsilon_r/\varepsilon_a$ is plotted in Figure 5.5. This ratio is very small for most of the specimens, and the



Figure 5.4 : Variation of radial and axial strains during anisotropic consolidation.

few higher values are a reflection of the very low radial strains (as low as 0.02%) recorded in those tests, and does not imply large volumetric strains nor a change in void ratio. Several specimens in Figure 5.5 exhibit negative $\varepsilon_r/\varepsilon_a$ ratios ranging from 0.0 to about -0.3. It is interesting to note that an $\varepsilon_r/\varepsilon_a$ ratio of -0.50 would correspond to zero volumetric strain. In



Figure 5.5 : Ratio of $\varepsilon_r/\varepsilon_z$ of undisturbed sands during anisotropic consolidation along $K_c = 2$ line.

contrast to consolidation along K_0 path, the $\varepsilon_r/\varepsilon_a$ ratio in hydrostatically consolidated sands is very large (often greater than 2) on account of the inherent anisotropy.

Consolidation along the K_0 line in itself should result in zero radial strain. However, the in-situ K_0 is not exactly known and all undisturbed specimens were consolidated assuming a K_0 of 0.5.

Small radial strain would therefore occur during reconsolidation, partly on account of the difference between this laboratory K_o value and the actual unknown in-situ K_o , and partly due to disturbances during sampling. It can be noted in Figure 5.5 that almost all Syncrude J-Pit sand specimens exhibit negative $\varepsilon_r/\varepsilon_a$ values. This might be an indication that the in-situ K_o of the Syncrude J-Pit sands might be higher than 0.50. Nevertheless, the measured radial strains are very small in all these tests. The data in Figure 5.4 demonstrates that assuming the radial strain to be twice that of the measured axial strain during K_o consolidation is not rational, and such an assumption leads to progressively erroneous results with increasing axial strain (Figure 5.5).

For a given axial strain, the radically different assumptions as to the $\varepsilon_i/\varepsilon_a$ ratio result in volumetric strain five times higher for specimens thawed by Method I than by Method II (Hofmann 1997). Thus even when the sand specimens undergo equal axial displacement under both methods, these assumptions lead to five time larger calculated void ratio changes in Method I. The calculated void ratio change using this scenario is therefore a manifestation of the assumptions, and is not indicative of the actual void ratio change of the sand. The measured height change data, which is an indirect indicator of the void ratio change, clearly suggest that Method I causes lesser change in the specimen void ratio at the conclusion of restoring the in-situ effective stresses, compared to Method II.

5.3 Undrained behaviour of undisturbed sand

The undrained behaviour of in-situ frozen undisturbed specimens from four different sites is presented in this section. Extensive in-situ tests were carried out at these sites to characterize the

soil deposits. Some results of these tests are summarized in Table 5.4, together with the location of the water table, and the estimated age of the deposits (Fear 1996). All sands retrieved by insitu ground freezing are uniform, and have D_{50} ranging from 0.20 to 0.35 mm. The fines content (passing ASTM #200 sieve) in Kidd and Massey site sands is less than 5%. Both Syncrude sands have a fines content of about 12%. The grain shape of these sands is a direct reflection of their source origin and subsequent transport and deposition at the current locations. The undrained response in triaxial compression, extension and simple shear is discussed below.

5.3.1 Monotonic Triaxial compression and extension tests

Figure 5.6 shows undrained triaxial compression and extension response of undisturbed Mildred Lake sand specimens, consolidated to the estimated in-situ stress state. The stiffest and the softest extremes of the measured response is shown in the figure. Denser specimens exhibited more dilative response both in compression and extension loading. None of the specimens exhibited true liquefaction type of response, either in compression or extension loading mode. Further, no significant strain softening was noted in the compression mode. In extension mode, however, looser specimens strain softened and exhibited limited liquefaction type of response. This indicates a very strong direction dependent behaviour of undisturbed sands. Undrained behaviour of undisturbed Mildred Lake sand reported by other researchers is also shown in Figure 5.6 for comparison (Vaid et al. 1996). These results also support the finding that undisturbed Mildred Lake sand does not significantly strain soften in the triaxial compression mode of loading even in its loosest in-situ state.

					1
		Mildred Lake	Massey	Kidd	Syncrude J-Pit
Appr age	oximate (years)	30	200	4000	0.1
Dep	oth (m)	27 to 37	8 to 13	12 to 17	3 to 7
Wate	er Table (m)	21	1.5	1.5	0.5
	Ko	0.5	0.5	0.5	0.5
Unit abov belo	t weight ve GWT w GWT	18.5 19.5	18.5 19.5	18.5 19.5	18.5 19.5
in-situ	test indice	s (average & standa	ard deviation)		
SP'	T N _{1, 60}	18.2(3)	10.3(3.8)	17.2(4.8)	3.4(2)
CPT	q _{c1} (MPa)	7.5(1.7)	5.3(1.0)	6.5(1.8)	2.4(1.5)
	F(%)	0.73(0.15)	0.40(0.09)	0.37(0.05)	0.87(0.33)
Vs	(ms ⁻¹)	156(20)	168(6)	177(5)	127(3)

Table 5.4: CANLEX Undisturbed specimens: in-situ characteristics (after Fear 1996)

Similar triaxial data on the behaviour of Kidd and Massey sands is shown in Figure 5.7 and 5.8. The Kidd sand is very dilative in triaxial compression. Little positive excess pore pressures developed, and the very strong strain hardening response resulted in very high strength associated with the generation of very high negative excess pore pressure that eventually led to cavitation of the pore fluid. The essentially constant shear strength exhibited by the medium dense Kidd sand ($e_c = 0.886$) with strain, at about 900 kPa is a result of deformation at constant effective confining stress due to pore fluid cavitation. Cavitation of pore water has in effect rendered the undrained test a drained one. The sand would have exhibited an even higher



Figure 5.6: Undrained triaxial compression and extension response of undisturbed Syncrude sand.



Figure 5.7: Undrained triaxial compression and extension response of undisturbed Kidd sand.



Figure 5.8 : Undrained triaxial compression and extension response of undisturbed Massey sand.

strength, had the initial back pressure been higher than its in-situ value of 100 kPa. In such scenarios involving very dilative soils, the maximum operating strength would be the drained strength. In contrast to triaxial compression, Kidd sand exhibits limited liquefaction type of response in triaxial extension. The minimum undrained strength measured in triaxial extension is only a fraction of that measured in triaxial compression.

Massey sand also exhibits similar direction dependent behaviour in triaxial compression and extension. All but one specimens are strain hardening in triaxial compression. The void ratio of the specimen that exhibited limited strain softening deformation is 0.926. This void ratio is much denser than those of several other specimens that are strain hardening. This inconsistency in the data suggests that this individual specimen might have suffered larger disturbance during sampling. Again, the strength of the sand in triaxial extension is much smaller on account of the strain softening behaviour in this loading mode.

Figure 5.9 shows the response of Syncrude J-pit sand specimens. These specimens were tested at in-situ as well as at higher stress levels. The higher stress levels were used to simulate the stresses after the building of the test embankment. One specimen at the in-situ stress level and one at higher stresses exhibit marginally strain softening response in triaxial compression. Increasing confining stress appears to have increased the potential for strain softening. Similar observation as to the effect of increasing confining stress at a given K_c was made by Vaid and Thomas (1995) and Vaid et al (2000b). In contrast to being dilative in triaxial compression, the sand is very strain softening in triaxial extension. Consequently its strength in extension is quite small compared to that in compression.



Figure 5.9: Undrained triaxial compression and extension response of Syncrude J-pit sand

Specimens that were exhibiting dilative behaviour after reaching phase transformation would presumably realise steady state after all dilation is complete. However, the specimens would have to be subjected to very large strains to realise this state. The end restraint effects that are inherently present in all soil testing devices often restrict the range of strains that can be imposed in a laboratory. During these triaxial tests, the specimens remained essentially uniform up to an axial strain of 12-15% in compression and to about 10-12% in extension. Excessive bulging in compression and necking in extension was clearly noted beyond these strains. Several researchers have imposed axial strains in excess of 30% in conventional triaxial apparatus in order to find the ultimate or steady state strength of sands. Extreme caution is required in interpreting such data as the specimen at that large strain is far from being an element. The measured stresses and strains rather than being reflective of one state, are in reality now the average of excessively non uniform states of stresses and strains within the specimen.

The results presented above, comparing the triaxial compression and triaxial extension behaviour of four undisturbed sands clearly indicate that the behaviour of in-situ sands is direction dependent. Sands tested are rarely strain softening, and never exhibit significant loss in strength beyond its peak in compression even in their loosest state. Yet, they are almost always strain softening in triaxial extension. This direction dependent behaviour is a manifestation of the inherent undrained anisotropy in sands. Bjerrum (1972) recognized the existence of this anisotropy in clays a long time ago. There is no reason to expect any difference in the response of sands. The existence of such anisotropy in reconstituted sand specimens has been demonstrated by several researchers over the years (Miura and Toki 1982, Vaid et al 1990a, Vaid and Thomas 1995, Riemer and Seed 1997 among others). The results presented above indicate that this direction dependence in undrained behaviour is predominant in undisturbed sands as well.

5.3.2 Monotonic and cyclic simple shear response

Figure 5.10 shows the undrained simple shear behaviour of undisturbed Mildred Lake Syncrude sand specimens over a range of void ratio. The loosest specimen exhibits strain softening response, but with little loss of shear strength beyond the peak. The response gradually transforms from contractive into dilative as the relative density increases. The simple shear response appears to be stiffer than the triaxial extension, but is softer than the triaxial compression. Vaid and Sivathayalan (1996a) have reported similar findings in specimens of this sand reconstituted by water pluviation. This implies a systematic softening in the response as the direction of major principal stress moves from 0° to the vertical in triaxial compression to 45° in simple shear (Roscoe 1970) and finally to 90° in triaxial extension. The friction angle at steady, quasi steady or phase transformation state, however, is essentially constant regardless of the void ratio or confining stress level.

The undrained simple shear behaviour of the other undisturbed sands is also essentially similar, in that they marginally strain soften in the loosest state, and increasingly strain harden as the void ratio decreases. However, an increase in density did not always produce a more dilative response. This inconsistency is apparently due to the possible non uniformity of the in-situ sand deposit, associated with changes in both the composition and gradation with depth, and due to variations within an individual specimen. It is essential to recognize that the void ratio (or relative density)



Figure 5.10 : Constant volume simple shear behaviour of Phase I Syncrude sand.
assigned to an undisturbed specimen is only an average value, and there can be variations within the specimen. Figure 5.11 shows effective stress path and the stress-strain loops of an undisturbed Syncrude sand specimen in undrained cyclic simple shear loading. The specimen suffers liquefaction in twelve cycles due to strain development on account of strain softening. This flow deformation is characteristic of the true liquefaction type of response identified by Castro (1969). Unlimited unidirectional flow, characteristic of steady state of deformation would have ensued at constant stresses, if not for the externally imposed unloading pulse that terminated the strain development at about 10% shear strain. At this point the shear stress amplitude dropped below the steady state strength of the sand. The excess pore pressure at the steady state of deformation was a constant at about 85% of the initial confining stress. A state of zero effective stress in sands occurs only when the shear stress is zero. Thus, a state of zero effective stress (or 100% excess pore pressure) is not realised when liquefaction occurs as a result of flow deformation, and the sand deforms at a constant non zero shear stress. However, a state of 100% excess pore pressure is realised when the unloading pulse in the cyclic wave form reduces the shear stress to zero.

Flow deformation resulting from cyclic shearing of Massey sand at a void ratio $e_c = 0.968$ is shown in Figure 5.12(a). The sand realises steady state in the 7th cycle with a steady state strength of about 10 kPa. The behaviour of Massey sand, at a slightly denser void ratio of $e_c = 0.958$ is shown in Figure 5.12(b). The sand now exhibits dilative response and progressively large strains develop as the effective stress path traverses through transient states of essentially zero stresses. This type of deformation is termed cyclic mobility (Ishihara et al. 1975, Vaid and Chern 1985). Figures 5.13 and 5.14 show the measured cyclic resistance in the form of number



Figure 5.11: Undrained cyclic shear leading to flow failure in Syncrude sand



Figure 5.12(a):

Undrained cyclic shear leading to flow deformation in loose Massey sand



Figure 5.12(b):

Undrained cyclic shear causing deformation on account of cyclic mobility in Massey sand.



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Figure 5.14 : Cyclic simple shear resistance of Syncrude sand.

of cycles vs cyclic stress ratio, for the undisturbed sands. These cyclic tests were carried out at the in-situ void ratio and stress levels. As such not enough data points are available at a given void ratio to develop the cyclic resistance curve of the sand at a given void ratio. Nevertheless, the trend lines are drawn linking specimens having similar void ratios. The rather flat cyclic resistance curves observed are typical of many reconstituted sands.

5.3.3 Post liquefaction behaviour in triaxial and simple shear

The post liquefaction behaviour of undisturbed sands was assessed both in triaxial compression and simple shear following liquefaction by cyclic loading or by static unloading. Figure 5.15



Figure 5.15 : Post liquefaction behaviour of undisturbed Kidd sand following liquefaction induced by static unloading

illustrates the post liquefaction response of loose undisturbed Kidd sand in triaxial compression. The sand, which was consolidated to $\sigma'_{vc} = 138$ kPa, realised quasi steady state conditions at an axial strain of about 1.5%, in triaxial extension (Figure 5.15 inset). The unloading of the deviatoric stress after an axial strain of about 10% liquefied the sand by inducing 100% excess pore pressure. The sand was now loaded in compression at a constant rate of strain. Figure 5.15 indicates that post liquefaction stress strain curve is comprised of three distinct regions of deformation; initial deformation at essentially zero strength and modulus, followed by deformation with increasing modulus, and finally deformation at essentially constant modulus. The effective stress state is confined along the angle of maximum obliquity right from the commencement of the post liquefaction loading. Similar behaviour has been reported for sands reconstituted by water pluviation (Vaid and Thomas 1995, Vaid and Sivathayalan 1996a).

Figure 5.16 shows the post liquefaction behaviour of two undisturbed Massey sand specimens in simple shear. Both specimens were consolidated to essentially identical effective confining stress, and then were cyclically loaded with identical cyclic shear stress amplitude. The denser sand liquefied in 30 cycles with a maximum shear strain of about 5.5%, and the looser sand in 7 cycles with a maximum shear strain of about 6.5%. The relative density can be noted to significantly influence the post liquefaction response, of the otherwise similar sands. The post liquefaction stress strain curves in simple shear may be noted to be similar to those under triaxial conditions, with three distinct regions of deformation.

The residual effective stress state at the onset of post liquefaction loading directly influences the first phase of deformation. The magnitude of the strain developed at essentially zero modulus is directly dependent on the residual effective stress. The stiffness of the sand increases steadily



Figure 5.16: Post liquefaction response of loose and dense Massey sand following liquefaction induced by cyclic loading

with strain during the second phase of deformation. This phenomenon of increasing stiffness with strain is the reverse of the assumed behaviour of soils, where straining is commonly associated with a loss of stiffness. This unusual response arise from the fact that on post liquefaction loading the sand dilates all the way from the beginning, causing the effective confining stress to increase with strain. In contrast, in pre liquefaction monotonic loading almost all sands initially contract, and thus cause a decrease in effective confining stresses, which in turn leads to stiffness degradation. The deformation during all three phases of the post liquefaction loading occurs at a mobilised friction angle that equals the angle of maximum obliquity noted under static loading.

The characteristics of the post liquefaction stress strain curve are apparently independent of the nature of the loading that induced liquefaction, i.e either static or cyclic, and the mode of loading, triaxial or simple shear. Figure 5.16 indicates that shear modulus and the rate of strength recovery in post liquefaction loading is dependent on the density of the sands. However, the response in the effective stress space is unique, since the angle of maximum obliquity is independent of stress level, density and loading mode (Vaid et al. 1990a, Vaid and Thomas 1995).

As noted in the pre liquefaction static and cyclic loading, the post liquefaction stress strain curve (and the associated shear modulus and strength) is also heavily dependent on the loading mode. The post liquefaction resistance of undisturbed Syncrude sand measured in triaxial compression and simple shear is compared in Figure 5.17. Both specimens were initially consolidated to essentially identical void ratio and confining stress level, and were liquefied by static load-unload cycle with a maximum shear strain of about 10%. The sand has a much higher modulus in triaxial compression than in simple shear. This loading direction dependent behaviour is a reflection of the anisotropic nature of the undisturbed sand. It appears that neither the large



Figure 5.17: Post liquefaction response of undisturbed Syncrude sand in triaxial compression and simple shear.

strains (up to about 10%) the sand had experienced in pre liquefaction loading nor the development of 100% excess pore pressure has altered the anisotropic nature of the fabric of the undisturbed sand.

The post liquefaction behaviour of undisturbed Syncrude sand under multiple unload-reload cycles is shown in Figure 5.18. Liquefaction was initially triggered by the development of 100% excess pore pressure by a static load-unload cycle in triaxial extension. On post liquefaction loading in compression the sand required about 8% shear strain to mobilize a shear strength of a mere 5 kPa, before entering the second phase of deformation where the modulus increased with strain. However, after the second unload cycle in compression which also terminated in 100% excess pore pressure, the sand mobilized the same 5 kPa strength in only about 1% strain in its subsequent reloading in compression. This implies that the amount of post liquefaction deformation required for the sand to exhibit commencement of any appreciable modulus increase with strain is dependent on the sense of post liquefaction loading with reference to the loading that induced liquefaction. A change in the direction of strain results in reduction in modulus compared to straining in the same direction. The post liquefaction response during the third phase of deformation in each cycle, however, may be noted to occur at an essentially constant modulus regardless of the number of load cycles.

5.4 Influence of specimen reconstitution technique

Several specimen reconstitution techniques, tamping and pluviation being the most common, are in use in current practice. The objective in all of these is to replicate a uniform sand specimen



Figure 5.18: Post liquefaction response of Syncrude sand under multiple unload-reload cycles

at the desired void ratio and effective stresses to simulate the sand mass in-situ. Moist tamping has been used to reconstitute very loose specimens (even looser than the ASTM maximum void ratio). This becomes possible due to capillary effects between grains. The loose, collapsible structure that ensues upon moist tamping often leads to a contractive strain softening response, that may unjustifiably condemn a sand as potentially liquefiable. As noted in chapter 2, pluviation in water has been shown to resemble the alluvial deposition process, because the fabric that ensues upon water pluviation has been found to be similar to that of the naturally deposited alluvial and hydraulic fill sands (Oda et al. 1978).

The influence of the method of specimen reconstitution was assessed from triaxial and simple shear tests. Syncrude sand from the Mildred Lake site (site used to sample undisturbed Phase I specimens) was consolidated to identical void ratio and effective stress following reconstitution by water pluviation, air pluviation and moist tamping. Specimens of Fraser River sand (similar to those obtained during Phase II undisturbed sampling) were also reconstituted using different techniques and were tested in triaxial compression, extension and simple shear loading (Vaid et al. 1999). In previous studies, the dominant interest on the effects of specimen reconstitution on undrained response has centred primarily around the cyclic resistance of sands (Mulilis et al. 1977, Ladd 1974, 1977). A direct comparison of the behaviour of undisturbed sand with its reconstituted counterpart using the same solids is pivotal in field characterisation using laboratory tests on reconstituted specimens.

A critical requirement of any specimen reconstitution method is the ability to produce uniform test specimens. Uniformity of specimens is essential since all laboratory tests are intended to measure element properties. Uniformity of reconstituted specimens is often tacitly assumed, and only a few researchers (Castro 1969, Emery et al. 1973, Mulilis et al. 1977, Vaid and Negussey 1988) have sought its direct confirmation. These limited studies demonstrate that the specimens reconstituted by moist tamping (MT) tend to be less uniform than those prepared by water pluviation (WP). Water pluviated specimens are much closer to being uniform with height (Vaid and Negussey 1988). Figure 5.19 illustrates the void ratio profile of test specimens reconstituted by water pluviation and moist tamping. The specimen was solidified using the gelatin technique (Emery et al. 1973), and cut up in a number of horizontal slices, after peeling the rubber membrane away. The void ratio of each slice was then computed after drying the solids in each slice. Serious non uniformities of void ratio may be noted in the moist tamped specimen over the specimen height. The local relative density varies by as much as $\pm 10\%$ from the average of the entire specimen, when the method of reconstitution was moist tamping. Similar serious non uniformities have been reported by Castro (1969) in moist tamped specimens. The local deviation of relative density from the average may be noted to be much smaller (within about 3%) for water pluviated sands. Unlike a moist tamped specimen, the water pluviated specimen therefore closely approaches an element test, in its requirement of homogeneity.

5.4.1 Compressibility and Accessible states

One-dimensional compressibility of Syncrude sand specimens during consolidation in the simple shear apparatus is shown in Figure 5.20. The specimens were reconstituted by air pluviation, water pluviation and moist tamping. Moist tamping yielded the loosest void ratio after reconstitution with an initial relative density of about -20% (i.e. looser than the ASTM e_{max}). The



Figure 5.19 : Direct assessment of uniformity of reconstituted specimens (a) water pluviated Ottawa sand, (b) moist tamped Fraser River sand, and (c) moist tamped sand C (after Castro 1969).

loosest initial relative density that could be attained was about 2% when the specimens were air pluviated, and about 10% when water pluviated. During subsequent set up and after application of a vertical confinement of about 20 kPa, relative density increased by about 15% for WP and about 25% for MT specimens. The compressibility of the moist tamped sand may be noted to be very high compared to that of the air and water pluviated sand. This is believed to be due to the potentially collapsible fabric that ensues upon moist tamping. Evidence of such collapse resulting in large decrease in void ratio may be found even while saturating such specimens, in the results reported by several researchers (Sladen et al. 1985, Chang et al. 1981, Marcuson and Gilbert 1972).

Figure 5.20 vividly indicates that the range of accessible states in the void ratio - confining stress domain is highly dependent on the method of specimen reconstitution. Similar data for Fraser River sand also indicate that the domain of accessible states is dependent on the method of reconstitution, in addition to the confining stress level (Sivathayalan 1994). Meaningful application of laboratory studies to in-situ sands require that the states attained in the laboratory be compatible with those in the field.

In-situ void ratio and effective stress state of two hydraulic fill tailings sands and a water laid deltaic sand at the CANLEX research sites are shown in Figure 5.21. In-situ void ratios were directly computed from the ice content of the frozen sand specimens. Accessible void ratio - effective stress states of these sands, when water pluviated in the loosest state, (assuming estimated $K_0 = 0.5$), and states attained by moist tamping (Wride and Robertson 1997) are also illustrated in the figure for comparison. For all three sands, virtually all in-situ states are accessible by reconstituting the sand by water pluviation.



Figure 5.20: Range of accessible states by different reconstitution techniques

The few data points that fall above the compressibility curve of the water pluviated specimens can be regarded as a normal scatter expected in experimentation. Looser void ratios might also have been caused by any upward gradient in the deposition environment. These undisturbed specimens were retrieved from sites identified as loose deposits after consideration of several alternative sites (Wride & Robertson, 1997). Further, the test embankment in Phase III of the CANLEX project was built by deposition of sand in water under gravity. As such these in-situ void ratio effective stress relationships illustrated in Figure 5.21 can be assumed to be representative of the loosest possible in-situ states. There is no evidence in the literature to



Figure 5.21 : In-situ states of undisturbed sands in comparison to states attainable by laboratory reconstitution methods.

suggest that the in-situ void ratio of a fluvial or hydraulic fill sand at a given vertical confining stress can be looser than that obtainable by water pluviation in the loosest state.

The data in Figure 5.21 could thus be viewed as evidence in support of the contention that water deposited in-situ sands are unlikely to exist in states looser than those achievable after loosest deposition by water pluviation. Similar evidence was found for another sand by Vaid and Pillai (1992) in a comprehensive study to assess the liquefaction susceptibility of Duncan Dam in BC.

Several researchers have reported sand liquefaction data at very loose density states -even looser than the ASTM (1991b) maximum void ratio. This has been made possible by adopting moist tamping as the method of reconstitution. The presence of capillary tension during moist tamping promotes the formation of very loose specimens. However, it seems inconceivable that such negative relative density states are accessible to the in-situ sands in an alluvial environment. Thus, the relevance of the looser states (that are inadmissible in an alluvial environment) simulated by moist tamping in characterizing the liquefaction response of in-situ sands seems questionable. Attempts to assign the flow characteristics of such artificially loose sand specimens to in-situ sands would be overly conservative (even if the influence of the collapsible moist tamped fabric on the undrained behaviour of sands is neglected).

5.4.2 Static Undrained Behaviour

Figure 5.22 shows direct comparative static undrained simple shear response of three specimens of Syncrude sand, reconstituted by different techniques to an essentially identical



Figure 5.22 :

Influence of specimen reconstitution technique on the undrained simple shear response of Syncrude sand

effective confining stress of 200 kPa, and void ratio of about 0.770. Water and air pluviated specimens were reconstituted at targeted initial deposition void ratios so that the void ratio after consolidation to the desired vertical confining stress of 200 kPa was essentially identical to that of the moist tamped specimen. The measured undrained response may be noted to be drastically different even though the initial void ratio and effective stress state were all essentially identical. The moist tamped sand is very strain softening and exhibit true liquefaction type of response associated with a very small steady state strength. The air pluviated sand, though contractive, shows only a marginal drop in shear strength beyond peak and exhibits limited liquefaction type of response. In contrast, the water pluviated sand does not even exhibit any strain softening. Instead, it behaves dilatively. Undrained simple shear tests on air and water pluviated specimens of Fraser River sand also exhibit similar differences in response (Vaid et al. 1999).

The undrained triaxial compression behaviour of Fraser River sand specimens reconstituted by moist tamping and water pluviation to essentially identical initial states are illustrated in Figure 5.23. Similar data in triaxial extension is shown in Figure 5.24. Like in undrained simple shear, the moist tamped sand strain softens and suffers true liquefaction type of response both in triaxial compression and extension. The water pluviated sand on the other hand does not strain soften in compression. It does strain soften in triaxial extension in the limited liquefaction manner. But the minimum undrained strength of the water pluviated sand is about an order of magnitude higher than that of the moist tamped specimen.

The strain softening response of the moist tamped sand is primarily due to the collapsible *"honeycomb like"* (Casagrande, 1975) structure that ensues upon reconstitution by moist tamping. In addition, considerable variations from the average specimen void ratio occur in a

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Figure 5.23 : Effect of specimen reconstitution technique on undrained triaxial compression response





moist tamped specimen (Figure 5.19), and the existence of such looser regions may also contribute to the softer response. However, it is important to recognize that non-uniformities within the specimen have be to small in order to satisfy the requirements of an element test.

Figures 5.23 and 5.24 clearly indicate that the characteristic response of water pluviated sands is highly influenced by the mode of loading. As noted in chapter 2, such direction dependent response of water deposited sands has been reported by several other researchers as well. In contrast, moist tamped sands exhibit true liquefaction type of response in both triaxial compression and extension implying that loading direction has little bearing on the response. Similar essential independence of the response of moist tamped sand on the loading mode may also be seen in the result reported by other researchers (Ishihara 1993, Castro et al 1982, Wride and Robertson 1997, Eshtehard 1996). This has often been cited as evidence in support of the results presented herein demonstrate that undrained response of water deposited sands is highly influenced by the loading mode, but that of the moist tamped sands is not. Therefore any postulates of the existence of a unique steady state line in sands reconstituted by other techniques nor to in-situ sands.

Dilative behaviour in an undrained test is characterized by the development of negative pore pressures. This increases the effective confining stress and thus the shear strength. It is significant to note that the moist tamped sands did not develop any negative pore pressure in any loading mode. Other test data on moist tamped sands also show that moist tamped specimens do not develop negative pore pressures, unless when the specimens are reconstituted to a very

dense state (Wride and Robertson 1997). Water pluviated sands, on the other hand, develop negative pore pressures even at the loosest reconstituted state. This difference is very critical, in that the liquefaction susceptibility and stability of sand deposits is directly linked to the fact whether a sand develops positive or negative pore pressures on undrained loading.

5.4.2.1 Mobilised friction angle at SS/QSS/PT

Figure 5.25 illustrates the effective stress states at steady or quasi steady state of contractive, and at phase transformation of dilative sand specimens of Syncrude sand reconstituted by different techniques. The effective stress states of all specimens lie on an essential straight line passing through the origin regardless of the method of reconstitution or the mode of loading implying a unique mobilized friction angle. Several other researchers have also reported that the mobilised friction angle at steady state, quasi steady state or phase transformation, $\phi_{SS/PT}$ is independent of the confining stress level, relative density or the loading mode (Ishihara et al. 1975, Vaid and Chern 1985, Vaid et al 1990a, Uthayakumar and Vaid 1998). The results shown in Figure 5.25 further demonstrate that $\phi_{SS/PT}$ is independent of the fabric as well. This suggests that $\phi_{SS/PT}$ of a given sand is unique and is dependent only on the mineralogy (Negussey et al. 1988).

5.5 Comparative undrained behaviour of undisturbed and reconstituted sands

Test results presented in the preceding sections demonstrate that the undrained behaviour of undisturbed sands is direction dependent. None of the undisturbed in-situ sands from the four tests sites are highly strain softening in triaxial compression even in their loosest state. However, they do exhibit strain softening behaviour in simple shear and triaxial extension. Such a behaviour is a confirmation of the existence of inherent undrained anisotropy in in-situ sand deposits.



Figure 5.25 : Effective stress state at steady, quasi steady (if contractive) and phase transformation (if dilative) states of undisturbed and reconstituted sands.

The triaxial compression and extension behaviour of reconstituted sands reported in the preceding sections, however, illustrates that not all types of reconstituted specimens exhibit direction dependent behaviour. Unlike the water pluviated sands that strain softened in extension, but strain hardened in compression at identical initial states, the moist tamped sand strain softened

in both loading modes culminating in steady state of deformation. This implies that the effect of inherent anisotropy is more pronounced in water pluviated compared to moist tamped sands, and points to a qualitative equivalence in the behaviour of water pluviated and natural alluvial (or hydraulic fill) sands in regard to their inherently anisotropic nature.

5.5.1 Static undrained behaviour

Test results illustrating a direct quantitative equivalence in the response of undisturbed sand and its water pluviated counterpart are presented in this section. As noted in Chapter 3, the reconstituted specimens were formed using the entire solids retrieved at the end of the undrained test on the undisturbed specimen. The void ratio and effective stresses of the reconstituted specimens prior to undrained shear were targeted to be identical to those of the undisturbed specimens.

Figure 5.26(a) compares the static undrained triaxial compression response of undisturbed Massey sand and its counterpart reconstituted by water pluviation to an essentially identical initial density and stress state. Similar response in triaxial extension is illustrated in Figure 5.26(b).

Both undisturbed and water pluviated specimens are dilative in triaxial compression, but exhibit strain softening of the limited liquefaction type in extension. Similar comparison of the behaviour of Kidd sands in triaxial compression and extension loading modes is illustrated in Figure 5.27. The Kidd sand specimens were densified by tapping the base of the triaxial apparatus following reconstitution in the loosest state by water pluviation so that the void ratio



Figure 5.26(a): Comparative undrained triaxial compression behaviour of undisturbed and water pluviated Massey sand



Figure 5.26(b):

Comparative undrained triaxial extension behaviour of undisturbed and water pluviated Massey sand



Figure 5.27(a):

Comparative undrained triaxial compression behaviour of undisturbed and water pluviated Kidd sand



Figure 5.27(b) :

Comparative undrained triaxial extension behaviour of undisturbed and water pluviated Kidd sand.

after consolidation to the in-situ stresses were essentially similar. The characteristics of the response can again be seen to be reasonably similar in both loading modes. Such direction dependent undrained response indicates that both alluvial in-situ and water pluviated sands are inherently anisotropic.

Static undrained simple shear response of undisturbed and their water pluviated counterparts from the Massey and Kidd sites is shown in Figure 5.28. The behaviour of water pluviated and undisturbed sands may be noted to be much closer in simple shear compared to that in triaxial. This could apparently arises from the fact that the sands in-situ are likely to have a somewhat non uniform void ratio with depth. The computed in-situ void ratio of the of the undisturbed specimen is in fact a representation of the average void ratio over the entire height of the specimen. Water pluviated specimens, on the other hand, are very uniform as illustrated earlier in Figure 5.19. This could render the response of the undisturbed triaxial specimen differ from those of their uniform water pluviated counterparts. The simple shear specimens are relatively thin (~20mm) and therefore are more likely to uniform over this small depth.

A number of undisturbed triaxial specimens were cut into several slices at the end of the undrained test, and the void ratio of each of the slices were measured and compared to the average void ratio. A variation in void ratio of as much as ± 0.03 from the average specimen void ratio were noted over the specimen height. The void ratio distribution in two undisturbed Syncrude sand specimens is shown in Figure 5.29. These results tend to substantiate the postulate behind the larger difference in behaviour noted in triaxial than in simple shear.

Figure 5.30(a) illustrates the effective stress conditions of all undisturbed in-situ frozen specimens at steady, quasi steady or phase transformation states, and their water pluviated



Figure 5.28: Comparative undrained simple shear behaviour of undisturbed and water pluviated sands.



Figure 5.29: Void ratio variation along the height of undisturbed Syncrude sand specimens

counterparts of Kidd and Massey sands. The minimum undrained strength for each pair of specimens may be noted to be essentially similar for both states. Even though individual specimens realise steady/quasi steady state at different effective stresses at due to the differences in void ratio and confining stress levels prior to undrained shear, the friction angle $\phi_{QSS/SS}$ mobilized is essentially constant This suggests that the behaviour of the sand in the water pluviated state is consistent with that of the same sand in the undisturbed state.

Test results similar to those in Figure 5.30(a) for Syncrude sand specimens are shown in Figure 5.30(b). Unlike in Kidd and Massey sands, reconstituted specimens of Syncrude sand were not tested at identical void ratio and confining stresses as the undisturbed specimens. Rather, tests



Figure 5.30(a) : Effective stress states at steady, quasi steady or phase transformation states in undisturbed and equivalent water pluviated sands.

were performed over a range of void ratio and confining stress levels. The locus of the effective stress states is again a straight line passing through the origin implying that the friction angle $\phi_{OSS/SS}$ mobilised at this state is also constant for Syncrude sand.

The minimum undrained strength at steady or quasi steady state normalised by the initial vertical effective stress in shown in Figure 5.31 for undisturbed and reconstituted Syncrude sand. The normalised shear strength of undisturbed and water pluviated specimens are very similar. In


Figure 5.30(b) : Effective stress states steady, quasi steady or phase transformation state of Syncrude sand over a range of effective confining stress and void ratio.

contrast, the moist tamped sand yielded very low normalised strength. Despite this low shear strength of moist tamped sands, the mobilized friction angle $\phi_{QSS/SS}$ is essentially independent of the method of specimen reconstitution.

Dyvik and Hoeg (1998) have recently reported that the undrained behaviour of undisturbed sands is quite different to that of the moist tamped sands. They tested silty sand and silty specimens obtained from a fluvial deposit in Sweden, and found that moist tamped sand exhibited



Figure 5.31 : Normalised effective stress states at steady/quasi-steady states.

much lower strength than the undisturbed sands. They concluded that ".. specimens prepared to in-situ density by moist tamping cannot be used to predict in-situ behaviour". Their results are a reflection of the different fabrics that ensue upon deposition in a fluvial environment and reconstitution by moist tamping, and it provides supporting evidence to the findings of this study. On the other hand, Ishihara et al. (1998) report a comparative study on the behaviour of undisturbed and reconstituted specimens of Masado soils, and conclude that "there is no strong evidence to decide which of the sample preparation methods can best duplicate the in-situ

process of deposition". The contrasting finding of Ishihara et al (1998) are likely due to flaws in their methodology and experimentation as pointed out by Sivathayalan et al. (2000).

5.5.2 Cyclic Resistance

The equivalence in the cyclic resistance of undisturbed and water pluviated sands were assessed by performing cyclic simple shear tests. As in monotonic tests, the reconstituted specimens were formed using the entire solids retrieved at the end of the test on the undisturbed specimen. The void ratio and effective stresses at the end of consolidation were targeted to be essentially identical to those of the undisturbed specimen. The specimens were then subjected to the same cyclic shear stress amplitude, as the undisturbed specimens. Figure 5.32 compares the measured cyclic response of an undisturbed and equivalent water pluviated Massey sand. The undisturbed sand developed strains in excess of 3.75% in the last half of the eighth cycle, and the water pluviated counterpart developed large strain in the first half of the ninth cycle. The characteristics of the effective stress path during cyclic loading can also noted to be similar. The primary interest in cyclic behaviour of sands is the number of stress cycles of a given amplitude required to induce liquefaction, and as the cyclic resistance of sands is expressed in the form of cyclic stress ratio vs number of load cycles, as illustrated in Figure 5.33(a). The data in this figure summarise the cyclic resistance of both undisturbed and equivalent water pluviated sands. The relative density of the specimens shown in Figure 5.33(a) ranges from about 22 to 30%. The cyclic resistance of the undisturbed sands may be seen to be reasonably similar to that of the equivalent water pluviated sands. This suggests that the cyclic resistance of in-situ sand deposits



Figure 5.32: Comparison of cyclic simple shear behaviour of undisturbed and equivalent water pluviated sand



Figure 5.33(a) : Cyclic resistance of undisturbed and equivalent water pluviated sand can be conveniently assessed by testing specimens reconstituted by water pluviation in the laboratory. This is a highly attractive economical alternative to testing undisturbed in-situ sands. Several previous studies have shown that the cyclic resistance of sands is highly dependent on the method of specimen reconstitution that controls the ensuing fabric in the laboratory (Mulilis et al 1977, Ladd 1977). The results in Figure 5.33(a), therefore imply that the laboratory characterization of the cyclic resistance of in-situ sands could be reliably made only if specimens are reconstituted by water pluviation. Water pluviation closely mimics the deposition process, and the ensuing fabric of fluvial and hydraulic fill sands.

The cyclic resistance of undisturbed Massey sand and its water pluviated counterparts is now compared in Figure 5.33(b) to the cyclic resistance of water pluviated Fraser River sand reported in the literature from earlier studies (Sivathayalan 1994, Vaid and Sivathayalan 1996a). Even though the Massey sand used in this study was obtained from a depth of about 10 to 12m, and Fraser River sand was simply dredged from the river, they are both comprised of the same geologic source material. A close equivalence is therefore noted in the measured cyclic resistance of the two sands. These results clearly suggest that water pluviated sands can be used to assess the liquefaction susceptibility of in-situ sands even under dynamic loading conditions. They would provide an inexpensive alternative to obtaining undisturbed sand sample for testing.

5.5.3 Post liquefaction behaviour

Figure 5.34 compares the range of post liquefaction response of both undisturbed and water pluviated Massey sand. Unfortunately, no direct comparison of the post liquefaction response can be attempted due to the large number of factors that influence the post liquefaction behaviour. Nevertheless, both undisturbed and water pluviated Massey sand may be noted to display the same three distinct phases of deformation; (i) the initial deformation at essentially zero stiffness, (ii) deformation with modulus increasing with strain and (iii) deformation at essentially constant



Figure 5.33(b): Comparison of measured cyclic simple shear resistance of undisturbed sands with a previous study on water pluviated specimens.

modulus. Similar characteristics of the liquefied water pluviated sands have been reported by Vaid and Thomas (1994) and Yasuda et al. (1995) in triaxial loading, and by Vaid and Sivathayalan (1997) in simple shear.

Figure 5.35 shows the magnitude of the shear strain required in post liquefaction simple shear loading to mobilize a small arbitrary shear strength of 2.5kPa as a function of the maximum shear strain developed during liquefaction. Results for both undisturbed and water pluviated sands are



Figure 5.34 : Range of post liquefaction stress strain curves of undisturbed and water pluviated Massey sand.

shown. This figure reflects the influence of the strain history prior to the initiation of the post liquefaction loading on the subsequent post liquefaction deformation. The data indicates that the strain range during which the sand deforms with essentially zero stiffness is dependent on the maximum shear strain developed during liquefaction. The large scatter in the data signifies that other parameters such as the relative density and confining stress level, may also influence this phase of deformation. The data shows that in particular the domain of strain over which the



Figure 5.35 : Influence of the magnitude of maximum shear strain during liquefaction on post liquefaction stress strain resistance.

stiffness remains essentially zero is highly dependent on the maximum past strain experienced during the loading that resulted in liquefaction.

Post liquefaction behaviour of water pluviated sands over multiple load cycles was also found to be similar to that of the undisturbed sand (Figure 5.18). Water pluviated sands also exhibit essentially constant shear modulus during the third phase of deformation among different post liquefaction load cycles (Vaid and Thomas 1995). The dependence of the shear modulus during third phase of post liquefaction deformation of undisturbed sands on the relative density and on the loading mode may be noted in Figures 5.16 and 5.17. Similar dependency of shear modulus on density and loading mode of water pluviated sands, illustrated in Figure 5.36, has been reported by Vaid and Sivathayalan (1997). Thus, there is a strong indication that the characteristics of the post liquefaction response of undisturbed sands is also similar to that of their water pluviated counterparts.



Figure 5.36: Influence of relative density on post liquefaction stress strain response (after Vaid and Sivathayalan, 1997).

The post liquefaction behaviour does not appear to be dependent on the deformation mechanism, viz true liquefaction or cyclic mobility, that was the cause of strain development. Even if large deformations develop as a result of true liquefaction, the unloading pulse of the dynamic load causes an essentially zero effective stress state in the sand. All subsequent deformation from a state of zero effective stress is associated with strain hardening where the sand dilates all the way from its initial zero effective stress state. This strain hardening response does not show any tendency to diminish, so as to reach a residual state, even when the sand is sheared in excess of about 20% shear strain.

Figure 5.37 shows the post liquefaction behaviour of undisturbed Syncrude sand specimen that exhibits true liquefaction type of response during cyclic loading. The sand reaches steady state of deformation in the twelfth loading cycle, and deforms at a constant residual strength of about 15 kPa. The unloading pulse in the applied cyclic load terminates the steady state deformation at about 8% strain. In post liquefaction loading the sand strain hardens and deforms at an essentially constant modulus after a strain of about 15%, without any approach to a residual state. The steady state strength of the sand in virgin loading does not appear to be a limiting strength on post liquefaction loading. Similar behaviour of undisturbed Kidd sand is shown in Figure 5.38. These results imply that a sand may yield higher strength after liquefaction, compared to the minimum undrained strength of the virgin sand. The stiffness, of course is much smaller in post liquefaction. It appears that a sand that originally exhibited steady state type of response that led to liquefaction will not realise a steady state in post liquefaction loading, even at strains in excess of 20%. Vaid and Thomas (1995) and Sivathayalan (1994) have also reported that the minimum undrained strength of the virgin contractive water pluviated sand does not



Figure 5.37: Post liquefaction stress- strain response in comparison to that of virgin loading in Syncrude sand.

represent a cap on the post liquefaction strength. Both water pluviated and undisturbed sands exhibit similar characteristics in this regard as well (Figures 5.37 and 5.38). This dramatic change apparently occurs because such a sand in the virgin state could undergo contractive deformation, whereas its deformation in post liquefaction loading is always dilative. On virgin loading, a contractive sand deforms at a constant friction angle $\phi_{QSS/SS}$ during steady state, but in post liquefaction it deforms at a mobilized friction angle ϕ_r that is a few degrees higher than $\phi_{QSS/SS}$ for all sands.



Figure 5.38: Post liquefaction stress- strain response in comparison to that of virgin loading in Kidd sand.

The prediction of earthquake induced displacements in sands requires the knowledge of its post liquefaction stress-strain response. The initial attempts to model the seismically induced displacements treated the failed sand mass as a rigid plastic system (Newmark 1965). Newmark's method was later refined and extended by several researchers (Ambraseys and Sarma 1967, Sarma 1975, Makdisi and Seed 1978, Byrne et al. 1992). Improved analytical techniques using effective stress analysis suggested by Prevost (1981) and Finn et al. (1986) modelled the behaviour within loading cycles assuming hyperbolic stress strain relationship that imply modulus

degradation. But in all cases it was assumed that the steady state, and thus the residual strength, remains unaltered at its pre liquefaction value. Furthermore, the current analytical methods for determining liquefaction induced displacements assume the stress strain relationship of the liquefied soils as shown schematically in Figure 5.39. The earlier methods assumed a linear relationship until the residual state. The recent models (Beaty and Byrne 2000), however, do accommodate stiffness increase with strain. Both approaches, however, limit the maximum post liquefaction strength at a residual value, that is often assumed to be slightly smaller than the strength of the virgin contractive sand. The test data on the undisturbed sands do not exhibit such a residual state in post liquefaction loading even up to about 40% shear strain. As a result, the current analytical methods would be conservative, and thus may over predict the earthquake induced displacements. Sands that exhibit true liquefaction type of response never dilate on virgin loading. Their effective stress path is bound within the contractive region below the steady state line as schematically shown in Figure 5.39(b). However, upon unloading after a controlled amount of steady state of deformation and on subsequent reloading in post liquefaction the deformation occurs due to dilation. The locus of the stress path in post liquefaction loading stays confined along the angle of maximum obliquity associated with strain hardening. The higher strength in post liquefaction is apparently caused by the imposition of deformation in the dilative region as a result of initial zero effective stress on the sand that would have otherwise deformed within the contractive region under virgin loading. In virgin loading, strain hardening sands (both

dilative and limited liquefaction type) could ultimately realize steady state at very high stresses, "*after all dilation is complete*" (Poulos 1981). At this stage, the confining stress would increase to a very large value making the sand respond contractively. Such large negative pore



Figure 5.39: Schematic illustration of the characteristic post liquefaction response in undrained shear

pressure cannot, however, develop in the field because it will be limited to the initial pore pressure + 1 atm. Cavitation of pore water would occur at this pressure and the sand would then deform drained, and not undrained. However, it is essential to recognize that very large strains would be required to mobilise the strength in post liquefaction loading.

5.6 Summary

The undrained behaviour of undisturbed in-situ sands from four different sites, and specimens reconstituted by different techniques in the laboratory are presented in this chapter. The effect of fabric was assessed from triaxial compression, extension and simple shear tests. A comparison of the response of these undisturbed and reconstituted specimens clearly indicate that

- Undrained response of sands is highly dependent on the fabric, and thus on the method of specimen reconstitution. A sand at a given void ratio and stress state may Serious non-uniformities exist in moist tamped specimens compared to those pluviated in water.
- Designs based on the very low shear strength of moist tamped sands could be very conservative as they would likely condemn a sand as being liquefiable, whereas in reality the sand may not be strain softening in the specified loading mode.
- In-situ ground freezing yields high quality "undisturbed" samples in saturated sand. Thawing the frozen specimen at a low effective stress state and subsequently reconsolidating to the in-situ stress state causes minimal disturbance. Ground freezing is an expensive endeavour and may not be economically justifiable or even affordable in all projects.

- The behaviour of in-situ sands is direction dependent, on account of the inherent anisotropy.
- The behaviour of water pluviated sands is quantitatively and qualitatively very similar to that of water deposited in-situ sands in triaxial compression, extension and simple shear loading modes, both under static and cyclic loading conditions, and in post liquefaction loading.
- the friction angle mobilised at steady state, quasi steady state, or phase transformation appears to be a unique property of the sand. It is independent of the void ratio, confining stress level, loading mode or fabric.
- Provided $\sigma_{3residual} = 0$, the post liquefaction stress strain curve is characterised by an increasing shear modulus with strain. Pre liquefaction steady state/quasi steady state strength of the sand is not a limiting strength in post liquefaction loading. The essential characteristics of the post liquefaction response of the undisturbed sands are similar to those of the water pluviated sands.

These results clearly demonstrate that the behaviour of in-situ sand masses can be conveniently assessed from specimens reconstituted by water pluviation in the laboratory. This is an attractive alternative to testing undisturbed sands, as sampling undisturbed sand specimens is quite tedious and expensive.

The ultimate objective of any fundamental studies on the behaviour of sands is to characterize the behaviour of in-situ sands, that are often fluvial or hydraulically placed. As these results clearly indicate that the behaviour of moist tamped sands has little resemblance to the behaviour of in-situ sands, extreme caution is required in assigning the characteristics of the moist tamped sand to the sand in-situ. These results clearly indicate that the characteristics of water pluviated sands in static, cyclic and post liquefaction loading can be directly assigned to in-situ sand deposits that are fluvial or hydraulically placed. Thus water pluviation should be the reconstitution method of choice in fundamental laboratory studies, if the results of these studies are to have any relevance to in-situ sand deposits.

Chapter 6

Generalised stress state and stress path dependent behaviour

6.1 Introduction

Undrained behaviour of reconstituted Fraser River sand is presented in this chapter. Only loose density states that are reconstituted by water pluviation are considered. Since the fabric and density are held constant, the effective stress state was the sole initial state variable. The generalised initial effective stress state is completely specified by the magnitudes and the directions of the three principal stresses. For convenience, it is represented by the derived stress parameters, effective mean normal stress σ'_{me} , inclination of major principal stress to vertical $\alpha_{\sigma e}$, intermediate stress parameter b_e , and effective stress ratio $K_e = \sigma'_{1e}/\sigma'_{3e}$. These, of course, are uniquely related to σ'_{1e} , σ'_{2e} , σ'_{3e} and $\alpha_{\sigma e}$ or the applied surface tractions σ'_{ze} , σ'_{ne} , σ'_{0e} and τ_{z0} .

Water pluviated fabric was selected because it mimics the behaviour of natural fluvial and hydraulic fill sands (Chapter 5). The findings of this study will therefore be of direct relevance to the behaviour of such sands in-situ. The loosest density state was chosen because of its greatest susceptibility to liquefaction. The hollow cylinder torsional shear (HCT) device was used for performing the tests. It enables control of the magnitudes of all three principal stresses, and the directions of two in the vertical plane of deposition. The undrained stress paths imposed

consisted of no change in the direction of principal stresses as well as changing principal stress directions.

6.1.1 Initial States

A variety of generalised initial stress states, characterised by σ'_{mc} , $\alpha_{\sigma c}$, b_c , and K_c were simulated. They were selected with the specific objective of isolating the influence of each state variable on undrained response from that of the others. In order to keep the scope of the investigations within tractable limits, the effect of σ'_{mc} level was, however, not explored. This was kept constant at 200 kPa in all tests.

The influence of the direction of major principal stress σ'_{1c} that was not coincident with the principal axes of anisotropy of the sand (vertical or horizontal) was also explored (i.e. $\alpha_{\sigma c} \neq 0$). All previous investigators focussed only on initial states characterised by $\alpha_{\sigma c} = 0$. The former constitutes non-axisymmetric, and the latter axisymmetric initial stress states. The intermediate principal stress parameter b_c selected was 0.4 or 0.5. This is typical of field problems, the majority of which closely approximate plane strain. Some axisymmetric initial states with $b_c = 0$ were also investigated. The influence of static shear bias as the initial state variable was assessed by choosing a range of K_c values from 1 to 2.5. The initial stress conditions and the subsequent undrained loading paths selected are those typically encountered in soil elements insitu; below sloping ground, embankments and other gravity structures.

At the end of reconstitution the HCT specimens were confined by a hydrostatic effective stress state of about 30 kPa. The desired initial stress state was arrived at by first loading drained at K_c = 1 to $\sigma'_m = 200$ kPa, followed by further drained shear at constant σ'_m , $\alpha_{\sigma c}$, and b_c , until the target K_c level (Figure 6.1a). In a few tests, where the target $\alpha_{\sigma c}$ and b_c were zero, drained loading to the initial state consisted of first loading to the target K_c under constant $\sigma'_3 = 30$ kPa, followed by further loading along the K_c line to the desired σ'_{mc} (Figure 6.1b).

6.2 Inherent Anisotropy

The behaviour of Fraser River sand under axisymmetric effective stress increments is presented in this section (Figure 6.1b). The major principal stress acts vertically and the intermediate and minor principal stresses are equal in these tests. The relative magnitudes of the measured axial, radial and tangential strains, which are the principal strains during axisymmetric loading, are used to illustrate the existence of anisotropy in water deposited sands.

The strain increments in Fraser River sand under hydrostatic stress increments from an initial hydrostatic effective stress state of about 30 kPa to 500 kPa are illustrated in Figure 6.2. The radial and tangential strains are properly corrected for membrane penetration effects, as discussed in Chapter 4. The essentially identical values the strain components ε_r and ε_{θ} under hydrostatic loading implies that the sand is isotropic in the horizontal plane. But the vertical axial strain ε_z , perpendicular to the bedding planes is much smaller than the radial and tangential strains along the bedding planes. This implies greater stiffness in the vertical than horizontal direction, and a



Figure 6.1 : Consolidation to generalised initial effective stress states

cross anisotropic fabric that ensues on deposition in water. Identical strains would have resulted in all three directions (i.e. $\varepsilon_r = \varepsilon_0 = \varepsilon_z$), if the sand was isotropic. The essential linearity of the strain paths shown in Figure 6.2 further indicates that the initial degree of anisotropy is not altered over the range of stresses probed. Similar findings have been reported in the literature by several other researchers from triaxial and torsional shear tests (Rowe 1971, El-Sohby and Andrawes 1972, Negussey 1984, Wijewickreme 1990).

The existence of inherent anisotropy has been attributed to both higher frequency of contact normals in the vertical deposition direction, and the preferred orientation of the longitudinal axis of the sand grains in the horizontal direction (Oda 1972). Fraser River sand grains are not spherical, but sub-angular. Therefore, when deposited in water the sand would assume an anisotropic fabric on account of both the larger frequency of contact normals, as well as the preferred horizontal orientation of the longitudinal axis of the grains. The existence of inherent anisotropy in the essentially rounded Ottawa sand, in contrast, would occur predominantly due to the higher frequency of contact normals in the vertical direction. This gives rise to the higher compressibility in the horizontal direction than the vertical (Sayao 1989). This contention is supported by the data from hydrostatic compression of mono-sized spherical glass beads from about 25kPa to 450 kPa (Figure 6.3). It may be noted that the radial strains are still larger than the axial, but the ratio $\varepsilon_r/\varepsilon_a$ is essentially independent of the size of spheres. It is of interest to note that the radial strains are about 100% larger than axial in Fraser River sand, but only about 50% larger in spherical glass beads. This suggests that the degree of inherent anisotropy may be much higher in angular sands compared to rounded sands. The former owes its fabric both to

higher frequency of contact normals in the vertical directions and the preferred horizontal orientation of the longitudinal axis of the grains, whereas, little preferred orientation would be expected in the latter.



Figure 6.2: Axial, radial and tangential strain increments under hydrostatic stress increment.



Figure 6.3 : Strain increments in uniform spherical particles under hydrostatic stress increments.

6.2.1 Drained deformation along constant K_c path (and K_o)

Figure 6.4 illustrates the strains induced in Fraser River sand under drained axisymmetric effective stress increments at constant effective stress ratio K_c . The sand was loaded along



Figure 6.4 : Axial, radial and tangential strains under axisymmetric proportional loading constant effective stress ratio path, from $\sigma'_m \approx 30$ kPa to 200 kPa. This type of loading constitutes what is commonly known as the proportional loading path. The essentially linear strain path at each K_c level implies that the nature of inherent anisotropy is fixed by K_c and is not significantly altered during loading at a constant effective stress ratio. The very small strain in the horizontal plane(Figure 6.4) when K_c = 2 implies that the coefficient of earth pressure at rest K_o

I

 \mathcal{C}^{*}

for loose Fraser River sand is slightly less than 0.5. Even though all test specimens were consolidated to a constant effective mean normal stress $\sigma'_{mc} = 200$ kPa, much larger axial strains develop when $K_c = 2.5$ compared to the hydrostatically consolidated sand ($K_c = 1$). This occurs because of increase in maximum vertical effective stress with increase in K_c for identical σ'_{mc} .

6.3 Undrained behaviour with generalised initial stress state

Undrained behaviour of sand consolidated to a generalised initial effective stress state was investigated under two types of loading paths.

- (i) principal stress directions fixed in directions identical to those prior to shear. This involves shear under constant α_{σ} (= $\alpha_{\sigma c}$), σ_m (= σ_{mc}) and b (= b_c).
- (ii) principal stress directions allowed to rotate in a controlled manner. This involves shear under constant $\sigma_m (= \sigma_{mc})$ and b (= b_c).

The response under a given static shear K_e , but at different initial $\alpha_{\sigma e}$, as well as under given $\alpha_{\sigma e}$, but different levels of K_e was explored under each of the two categories of tests. The desired undrained stress path was followed using feedback control in real time. The maximum excursions from the targeted parameters σ_{me} , α_{σ} and b_e in a typical test in type (i), shown in Figure 6.5, may be noted to be relatively insignificant. This indicates that the prescribed undrained stress paths are faithfully followed by the control system.



Figure 6.5 : Targeted and actual control parameters followed in a typical test.

6.3.1 Response under fixed principal stress directions

The series of tests carried out in this category were aimed at investigation of the influence of the initial static shear and the direction of major principal stress, and are listed in Table 6.1. The initial effective stress state, and the subsequent undrained loading path in each test have been identified.

The variations in the surface tractions $\sigma'_{z_c} \sigma'_{r_c} \sigma'_{\theta_0}$ and $\tau_{z\theta}$ and the corresponding strains induced in specimens consolidated to an initial $K_c = 2.0$ and sheared at several constant values of $\alpha_{\sigma} = \alpha_{\sigma c}$ are illustrated in Figure 6.6. The computed principal stresses and induced principal strains, and the mobilized effective stress ratio are shown in Figure 6.7 for the same series of tests. The vertical strain ε_{z} , referred to the fixed direction, is the monotonically changing independent variable against which all stresses and strains are plotted. ε_z gradually changes from the major principal strain at $\alpha_{\sigma} = 0^{\circ}$ to minor at $\alpha_{\sigma} = 90^{\circ}$. The magnitude of the intermediate principal strain is much smaller than either the major or minor, confirming that the chosen stress path with b = 0.4 imposes conditions close to plane strain during undrained shear. Other researchers have also demonstrated similar value of b during undrained plane strain deformation (Uthayakumar and Vaid 1998, Yoshimine et al. 1998).

The strain paths referred to fixed co-ordinate directions are essentially linear (Figure 6.6b) at small strains, and hence until a threshold mobilised stress ratio $R = \sigma'_1/\sigma'_3$ at each α_{σ} . This implies that the degree of initial anisotropy that is determined by the magnitude of $\alpha_{\sigma c}$ and K_c stays preserved for small increments in R in excess of the initial value of K_c . The paths, however,

Test	Initial State					Remarks
	σ_{mc}	e _c	b _c	$\alpha_{\sigma c}$	K _c	
A01	200	0.897	0.4	0	1.00	Shearing at constant values of σ_m , b and α_σ σ_d
A02	200	0.892	0.4	30	1.00	
A03	200	0.890	0.4	60	1.00	
A04	200	0.883	0.4	60	1.00	K_{c} $\int_{0}^{0} \int_{0}^{0} \int_{0}^{0} \int_{\alpha_{cc}}^{\infty} d_{\alpha_{cc}}$
A05	200	0.880	0.4	0	1.25	
A06	200	0.893	0.4	30	1.25	
A07	200	0.895	0.4	60	1.25	
A08	200	0.889	0.4	90	1.25	
A09	200	0.889	0.4	0	1.50	
A10	200	0.889	0.4	30	1.50	
A11	200	0.891	0.4	60	1.50	
A12	200	0.894	0.4	90	1.50	
A13	200	0.891	0.4	0	1.75	
A14	200	0.883	0.4	90	1.75	
A15	200	0.888	0.4	0	2.00	
A16	200	0.884	0.4	30	2.00	
A17	200	0.886	0.4	60	2.00	
A18	200	0.880	0.4	90	2.00	

Table 6.1: Initial states of specimens tested under fixed principal stress directions



Figure 6.6(a) : Variation of surface tractions with vertical strain during undrained shear at fixed principal stress directions.



Figure 6.6(b): Variation of radial, tangential and shear strain on the horizontal plane with vertical strain during undrained shear with fixed principal stress directions.



Figure 6.7(a) : Variation of principal stresses and effective stress ratio during undrained shear with fixed α_{σ} .



Figure 6.7(b) :

Variation of principal strains with vertical strain during undrained shear at fixed principal stress directions.

become progressively non linear with increasing stress ratio at all α_{σ} levels, which demonstrates a gradual evolution of strain induced anisotropy. At each α_{σ} , the change in anisotropy can be attributed to the increase in effective stress ratio alone, since α_{σ} stays constant in these tests. The degree of induced anisotropy characterised by the curvature of the strain path is small, but does increase with increasing effective stress ratio, and strain.

The relationships of maximum shear stress $(\sigma_1 - \sigma_3)/2$ with the maximum shear strain $\gamma_{max} = (\epsilon_1 - \epsilon_3)$ and the effective stress path for the data in Figure 6.6 are shown in Figure 6.8. The sand, which is hardly strain softening when $\alpha_{\sigma} = 0^{\circ}$, becomes increasingly contractive as α_{σ} increases, and finally responds in an essential steady state manner at $\alpha_{\sigma} = 90^{\circ}$. Such a systematic softening of sand occurs on account of the inherent anisotropy on deposition and its evolution, if any, until loading to the initial $K_c = 2$ (Arthur and Menzies, 1972). As noted in the results presented in the previous section, the higher compressibility along the bedding planes contributes to progressively softer undrained response as the direction of major principal stress moves from the vertical and approaches the direction of the bedding plane. This is consistent with the findings of larger volumetric strains at higher α_{σ} in drained shear (Symes et al. 1985, Sayao 1989, Wijewickreme 1990). A systematic softening in undrained response of Fraser River sand with initial hydrostatic state ($K_c = 1$) has been reported by Uthayakumar and Vaid (1998) as α_{σ} increases from 0° to 90°

The maximum shear stress at steady or quasi steady state, (the minimum undrained strength, $S_{ss/Qss}$) which is about 80 kPa for loading at $\alpha_{\sigma} = 0^{\circ}$, reduces to about 38 kPa as α_{σ} increases to 90°. Thus, at a given void ratio, effective confining stress and static shear stress level, the minimum undrained strength of the sand is highly dependent on the direction of principal stresses





Figure 6.8 : Undrained behaviour of Fraser River sand at different fixed principal stress directions at $K_c = 2$.
relative to the bedding plane. Data similar to that in Figure 6.8, but at a lower initial static shear level corresponding to Kc = 1.5, (Figure 6.9) again shows increased propensity toward strain softening at higher α_{σ} .

Comparative behaviour of sand at identical initial $\alpha_{\sigma} = 0^{\circ}$, but different K_c levels illustrate the effect of static shear level only on the undrained response (Figure 6.10). No strain softening to any significant degree is now noted when the sand is sheared at $\alpha_{\sigma} = 0^{\circ}$, regardless of the initial level of static shear. Larger K_c does promote marginally more strain softening, but the differences are relatively minor at this $\alpha_{\sigma} = 0^{\circ}$. The sand starts strain hardening at shear strain in excess of about 2%, and there is no tendency to approach a steady state up to a shear strain of about 10%. Similar influence of static shear, but in axisymmetric triaxial compression ($\alpha_{\sigma} = 0^{\circ}$ and b = 0) in contrast to the essentially plane strain conditions in these tests, has been reported by Vaid et al. (2000b).

The influence of the level of initial static bias K_c , but now at an $\alpha_{\sigma} = 60^{\circ}$ is assessed in Figure 6.11. The sand becomes very contractive when sheared at this higher $\alpha_{\sigma} = 60^{\circ}$, and the resulting strain softening leads to an essential steady state type of response at each K_c level. This is in significant contrast to the dilative behaviour that was noted at $\alpha_{\sigma} = 0^{\circ}$ (Figure 6.10). The minimum undrained strength of sand at identical initial void ratio and effective mean normal stress may be seen to be dependent on the initial static shear stress level. Its magnitude at $K_c=1.25$ is about half that of that at $K_c=2.0$ at $\alpha_{\sigma} = 60^{\circ}$ even though the differences were minor at $\alpha_{\sigma} = 0^{\circ}$. Thus the influence of initial static shear stress level on the minimum undrained strength is more prominent when the direction of α_{σ} is larger at 60° (Figure 6.11) than at 0° (Figure 6.10).







Figure 6.10 : Undrained behaviour of Fraser River sand over a range of initial static shear at $\alpha_{\sigma c} = 0^{\circ}$



Figure 6.11 :

Influence of initial static shear on the undrained response of Fraser River sand at $\alpha_{\sigma c}$ = 60 $^\circ$

The influence of static shear on the liquefaction potential is often assessed from triaxial compression tests, and is considered relatively minor for loose sands, based on these test results at $\alpha_{\sigma} = 0^{\circ}$. However, test results in Figure 6.11 imply that static shear level may have a profound influence depending on the initial direction of α_{σ} , and that triaxial compression loading will fail to fully describe the dependence of minimum undrained strength for initial generalised stress conditions. Further, a sand that is stable at $\alpha_{\sigma} = 0^{\circ}$ at given initial static shear level may even possess the potential of a flow slide at values of $\alpha_{\sigma} > 0$ (Figure 6.8). The most commonly adopted triaxial compression results in practice will, therefore, err on the unsafe side. This direction dependence of the minimum undrained strength is consistent with the nature of inherent anisotropy in sands.

The minimum undrained strength at $\alpha_{\sigma} = 90^{\circ}$ and $K_c = 2$ is much smaller than the initial static shear stress of 67 kPa (Figure 6.8). A relatively minor undrained perturbation in deviator stress of about 5 kPa triggers strain softening and causes the sand to flow with a minimum undrained strength of about 35 kPa. The potential for such instability is of great concern in engineering design. Conventional undrained triaxial compression tests will not reveal the potential for such a catastrophic failure. This emphasizes the need to duplicate the generalised initial effective stress states that exist in the field for estimating meaningful minimum undrained strength values of insitu sands

6.3.1.1 Triggering of strain softening

The peak of the stress-strain curve signifies the initiation of strain softening, and the mobilised effective stress ratio at that instant is termed the critical stress ratio, CSR (Vaid and Chern 1985). The effective stress states at the onset of strain softening in Figure 6.12(a) for a range of α_{σ} and K_{e} values show that the friction angle Φ_{CSR} mobilised at this state is essentially constant at each α_{σ} level, regardless of K_{e} . The minor scatter in the data at $\alpha_{\sigma} = 90^{\circ}$, is apparently due to the influence of initial static shear level, but is relatively insignificant. It can be noted that ϕ_{CSR} is strongly influenced by the direction of principal stresses. It varies from a low of about 19° for $\alpha_{\sigma} = 90^{\circ}$ to about 30° for $\alpha_{\sigma} = 0^{\circ}$ (Figure 6.12b). Triaxial compression, extension and simple shear studies on Fraser River sand also show similar dependency of ϕ_{CSR} on the loading mode (Vaid and Thomas 1995, Vaid and Sivathayalan 1996a).

A systematic decrease in peak shear strength $S_{u,peak}$ may be noted with increase in α_{σ} at each K_c level (Figure 6.13). The greatest decrease in the peak undrained shear strength appears to be associated with low levels of static shear stress, and when α_{σ} is between 0° and 30°. Similar systematic decrease in $S_{u,peak}$ with increasing α_{σ} has been reported for hydrostatically consolidated sands by Uthayakumar (1996) and Shibuya and Hight (1987). Figure 6.13 also shows that at a given α_{σ} , $S_{u,peak}$ increases somewhat at higher K_c , the larger increases being associated with shear at higher α_{σ} values.



Figure 6.12 : Variation of (a): effective stress states at the instant of strain softening, and (b): friction angle mobilised at peak with α_{σ}



Figure 6.13 : Influence of initial static shear, and direction of principal stresses on peak undrained strength.

Triaxial compression test results presented by Chern (1985), Vaid and Thomas (1995), and Vaid et. al. (2000b) also confirm that $S_{u,peak}$ of strain softening sands at essentially identical initial void ratio and confining stress is dependent on the initial static shear level.

The degree of strain softening is often characterized by the brittleness index I_B (Bishop 1971), which is considered to be a good indicator of the flow potential of a contractive sand. Bishop defined brittleness index as

$$I_B = \frac{\left(S_{peak} - S_{min}\right)}{S_{peak}}$$
(6.1)

in which S_{peak} and S_{min} are the peak and minimum undrained strengths. The brittleness index at a given K_c increases dramatically as α_{σ} increases from 0° towards 90° (Figure 6.14). The behaviour of sand in triaxial compression, extension and simple shear loading also supports the contention that flow potential of sands increases with α_{σ} (Vaid and Thomas 1995, Vaid and Sivathayalan 1996a). I_B, however, does not appear to be significantly influenced by the initial static shear stress level, at a given α_{σ} . I_B increases marginally with increasing static shear level at $\alpha_{\sigma} = 0^{\circ}$, but decreases somewhat at $\alpha_{\sigma} = 90^{\circ}$ suggesting that I_B may not be a good index to assess the influence of initial static shear on impending strain softening. The initial stress state with high K_c and higher α_{σ} may strain soften to a much lower steady state strength than the initial static shear stress (Figure 6.11). Such a response where the undrained steady state strength is lower than the current in-situ stresses will have catastrophic consequences once the flow is triggered. The original definition of brittleness index by Bishop was intended to characterize the behaviour of hydrostatically consolidated soils. As noted earlier, this definition is unable to fully describe the flow potential for anisotropically consolidated sands. A modified brittleness index $I_{\bar{R}}$ defined by

$$I_{\overline{B}} = \frac{\left(S_{\text{peak}} - S_{\text{min}}\right)}{\left(S_{\text{peak}} - S_{\text{static}}\right)}$$
(6.2)

is used in this thesis to conveniently characterize the flow potential of sands consolidated to initially anisotropic stress state. This definition uses the shear stress increment ($S_{peak} - S_{static}$) required to initiate strain softening deformation, and for hydrostatically consolidated soils the expression of $I_{\overline{B}}$ reduces to I_{B} . A value of $I_{\overline{B}} = 0$ would represent no strain softening and a value of 1 would imply minimum undrained strength equal to the initial static shear stress. Values of $I_{\overline{B}}$



Figure 6.14 : Variation of brittleness index I_B with $\alpha_{\sigma c}$ and K_c .

larger than 1 correspond to minimum undrained strength lower than the static shear stress, and would imply a catastrophic collapse if equilibrium is disturbed by a small undrained perturbation. The modified $I_{\overline{B}}$ indicates that the flow potential of a sand depends both on the initial static shear level and the direction of major principal stresses during shear (Figure 6.15).

Higher initial static shear levels represent a potential for catastrophic consequences at larger values of α_{σ} , but its influence at $\alpha_{\sigma} = 0^{\circ}$ is only marginal. It is important to emphasize that this modified definition of brittleness index is applicable only when principal stress directions remain fixed throughout the shearing phase.



Figure 6.15 : Potential instability as assessed by the modified brittleness index

6.3.1.2 Steady or quasi steady state

Figure 6.16 shows the effective stress conditions at steady state or quasi steady state corresponding to the minimum undrained strength over the full range of α_{σ} and K_c values. Quasi steady state essentially coincides with the instant of maximum excess pore pressure which occurs when the undrained response transforms from contractive (positive excess pore pressure generation) to dilative (negative excess pore pressure generation) during shear. Thus, this state



Figure 6.16 : Effective stress state at steady, quasi steady or phase transformation states.

is synonymous with the phase transformation state for sands that exhibit strain softening behaviour. The data points in the figure lie along a straight line passing through the origin regardless of the initial α_{σ} or K_c, and correspond to a friction angle $\Phi_{ss/Qss}$ of about 34°. Therefore, the friction angle mobilised at the instant of minimum undrained strength is independent of both the direction of principal stresses during shear and the level of initial static shear.

Even though there exists a general agreement in the literature regarding the uniqueness of $\phi_{SS/QSS}$ for a sand there has been some reluctance in accepting that both steady and quasi steady states can be treated within the same framework (Been 1998). Vaid and Chern (1985) were the first to experimentally demonstrate that in triaxial compression both steady and quasi steady states can indeed be treated within the same framework in the effective stress space. Test data presented in Figure 6.16 includes data for initial states that culminated in steady state following flow failure, as well as those that exhibited quasi steady state type of response. These results thus do support the contention that both steady state and quasi steady state (i.e., phase transformation) can be treated within the same framework in the effective stress space, even under generalised stress conditions.

It has also been demonstrated in a number of earlier studies that the friction angle mobilised at steady or quasi-steady state is independent of the void ratio, confining stress levels, and mode of loading, triaxial or simple shear (e.g. Bishop 1971, Ishihara et al. 1975, Vaid and Chern 1985, Kuerbis et al. 1988, Vaid et al. 1990a, Vaid and Sivathayalan 1996a, Riemer and Seed 1997). Negussey et al (1988) have further shown that $\phi_{ss/oss}$ equals the constant volume friction angle, ϕ_{CV} measured in a drained test. The results in Figure 6.16 thus extend the earlier findings in that $\phi_{SS/QSS}$ is also independent of the level of static shear, direction of α_{σ} during shear and the loading mode, and suggests that steady and quasi-steady states are unique in the effective stress space. However, as noted elsewhere in this chapter, uniqueness of steady/quasi steady state in the effective stress space neither guarantees nor requires uniqueness in the void ratio-effective stress space.

6.3.1.3 Angle of maximum obliquity

Sand that strain softens in a quasi steady state manner or dilative reaches the line of maximum obliquity with continued straining beyond the phase transformation state, and thereafter deforms along this line of maximum obliquity. The mobilised friction angle at maximum obliquity, ϕ_f , is about 3 to 4° larger than that at phase transformation. Its value for Fraser River sand is about 37°. This angle, like $\phi_{SS/QSS}$ has been demonstrated to be unique for a given sand regardless of its initial effective stress state, stress path in triaxial or simple shear or loading history (Vaid and Chern 1985, Chung 1985, Vaid and Thomas 1994, Uthayakumar 1996).

6.3.2 Minimum undrained strength of sands

The shear stress at steady state (for sands exhibiting true liquefaction type of response) or at quasi steady state (for sands exhibiting limited liquefaction type of response) corresponds to the

minimum undrained strength $S_{ss/Qss}$ of a strain softening sand. This is also called the residual strength. Figure 6.17 illustrates the dependence of the minimum undrained strength on α_{σ} and K_c . A systematic decrease in the strength may be seen with increasing α_{σ} at a given initial K_c . The strength in loading paths with $\alpha_{\sigma} = 90^{\circ}$ is only about a third of that under loading paths with $\alpha_{\sigma} = 0^{\circ}$ at $K_c = 1$, and about half at $K_c = 2$.



Figure 6.17: Dependence of minimum undrained strength on static shear and direction of major principal stress.

Principal stress directions would normally vary along potential failure surfaces that are commonly encountered in field problems. The essentially plane strain in-situ conditions are often simulated using axisymmetric triaxial compression and extension. As illustrated in Figure 6.18, α_{σ} in soil elements along the potential failure surface in a slope is close to 0° at the crest, but gradually increases to about 90° at the toe. Based on the test results in Figure 6.17, this implies a gradual reduction in the undrained strength along the failure surface, for a given initial void ratio. However, a constant undrained strength measured in triaxial compression tests is normally assigned in practice to all elements along the failure surface regardless of the direction of α_{σ} .





Since the minimum undrained strength is a critical parameter in the deformation and stability analyses of earth structures, the use of triaxial compression strength ($\alpha_{\sigma} = 0^{\circ}$) as the operating strength would err on the unsafe side. It is encouraging to note that the dependency of undrained strength on α_{σ} is being accepted by the profession and has been incorporated in recent models for predicting earthquake induced displacements and in stability analysis (Beaty and Byrne 2000, Puebla 1999). As noted by Vaid et al. (1990b) the failure to recognize the dependency of undrained strength on α_{σ} might have been the cause of the difficulties Castro et al (1985) and Sladen et al. (1985) experienced in predicting liquefaction induced failure or displacements in San Fernando dam and Nerlerk berm slides respectively. Castro et al. (1985) had to apply a reduction factor of 20 to the $\alpha_{\sigma} = 0^{\circ}$ strength in order to match the back calculated average operating strength along the failure surface.

Even though an increase in K_c promotes more severe strain softening response as quantified by $I_{\bar{B}}$, (Figure 6.15) it does not seem to increase the minimum undrained strength substantially at a given α_{σ} . The dependency of $S_{SS/QSS}$ on K_c is much smaller than its dependency on α_{σ} . A small dependence of $S_{SS/QSS}$ on K_c can also be noted in the triaxial compression test data reported by Vaid et al (2000b). Furthermore, the rate of increase in $S_{SS/QSS}$ with increasing K_c appears to be not dependent on the value of α_{σ} (Figure 6.17). Even though the minimum undrained strength increases with K_c , it may be smaller than the static shear stress particularly at higher levels of K_c . Thus, regardless of the higher $S_{SS/QSS}$ mobilised, the sand at a higher initial static shear might undergo the most damaging strain softening response leading to flow failure at a given α_{σ} . It is important to emphasize that results shown in Figure 6.17 represent initial states with essentially identical void ratios and mean normal effective stresses. Yet they yield significantly different steady state (or quasi steady state) strengths depending on α_{σ} and K_c. The specimens that realised a steady state following strain softening are identified in the Figure by solid symbols. The data clearly shows that the minimum undrained strength (whether steady state strength or quasi steady state strength) is not uniquely related to the void ratio alone, but rather is dependent, in addition on α_{σ} , K_c and confining stress level, σ'_{3c} .

Triaxial extension (Vaid and Thomas, 1995) and simple shear (Vaid and Sivathayalan, 1998) test data on contractive sands reproduced in Figures 6.19(a) and 6.19(b) also confirm the dependency of steady/quasi steady state strength on confining stress level and the loading mode, in addition to void ratio. However, the minimum undrained strength when normalised by the initial confining stress appears essentially independent of the confining stress level. The loading mode plays an important role in the magnitude of the realised steady state strength, and the results shown in Figure 6.17 showing a systematic decrease in $S_{SS/QSS}$ with α_{σ} is consistent with these simple shear and triaxial test results.

The data shown Figure 6.17, is replotted in Figure 6.19(c) with minimum shear strength normalised with respect to the major principal stress. This is attempted in order to assess whether the normalised minimum undrained strength under generalised loading conditions is independent of initial stress state as observed in triaxial and simple shear tests. Unlike the minimum undrained strength, which is considerably influenced by the initial static shear (K_c level), the normalised shear strength does not appear to be dependent on K_c regardless of the value of α_{σ} . Apparently,





Dependence of undrained strength on confining stress level, and void ratio in triaxial extension



Figure 6.19(b):

Dependence of undrained strength on confining stress level, and void ratio in simple shear



Figure 6.19(c): Variation of normalised minimum undrained strength with α_{σ} at different initial static shear levels

normalising the minimum undrained strength with major principal stress results in response essentially independent of static shear at a given $\alpha_{\sigma c}$. This is in contrast to common practice where shear strength is often normalised with respect to the effective confining (or mean normal) stress This provides a rationale for using σ'_v in simple shear and σ'_1 (= σ'_3 under hydrostatic consolidation) triaxial as the normalising parameters.

The combined treatment of quasi-steady state and steady states has often been criticized by the proponents of the steady state concepts based on the argument that shearing at quasi-steady state takes place at constant stresses and void ratio only temporarily. Steady state concepts contend that all sands will reach this state ("a state at which the soil continues to deform under constant stresses at a constant void ratio") that is uniquely related to their void ratio only.

Pluviated sands rarely strain soften under loading modes involving major principal stress direction essentially perpendicular to the bedding planes. In such dilative sands (Figure 6.8; $\alpha_{\sigma} = 0^{\circ}$) steady state can only be realised after all dilation is complete, and it occurs at very large strains as illustrated schematically in Figure 6.20. Dilative specimens, if undrained, must accommodate very large negative excess pore pressure in order to reach the steady state. If the in-situ pore pressure (or the laboratory back pressure) is not large enough, cavitation of pore water would render the undrained test a virtually drained one. The undrained strength in such a case may be even higher than the drained strength, and is of little interest in designing for such loading. In fact, the steady state of a dilative sand is often inaccessible in the laboratory on account of the very large strains required to mobilise it.

According to the steady state concepts a sand at a given initial state, which may be strain softening in triaxial extension, but strain hardening in triaxial compression should realise steady state at the same effective stress conditions. In Figure 6.8, for example, at $\alpha_{\sigma} = 90^{\circ}$, there is no dilation (i.e. no negative pore pressure developed) and the soil will reach steady state at effective stresses lower than its value prior to undrained shear. In contrast, when $\alpha_{\sigma} = 0^{\circ}$, the sand will develop high negative excess pore pressures, and hence will reach steady state at very high effective stresses. Thus, if the soil at a given initial state responds in flow failure type strain softening in one loading mode (Figure 6.8, $\alpha_{\sigma c} = 60^{\circ}$ or 90°) but exhibits limited liquefaction or dilative response in another mode (Figure 6.8, $\alpha_{\sigma c} = 0^{\circ}$ or 30°) then it is highly improbable that



Effective mean normal stress



Schematic illustration of minimum undrained strength and steady state strength.

it will reach steady state at identical effective stresses, as presumed in the steady state theory. Thus, the steady state concepts that may be valid in a specific loading mode may not hold true among different modes. Further, it is essential to recognize that a steady state that is mobilised at very large strains, after all dilation is complete, is of little practical interest from the perspective of performance of an earth structure. Only the steady state strength (or quasi steady state strength) following strain softening that corresponds to the minimum undrained strength will be of primary concern in engineering design.

6.4 **Response under principal stress rotation**

The series of tests carried out for investigating the influence of initial static shear K_c and $\alpha_{\sigma c}$ on undrained behaviour that also involves principal stress rotation are listed in Table 6.2. The initial state and subsequent undrained loading path in each test has been identified. Three distinctly different types of undrained stress paths were considered.

- (a) σ_d and α_σ allowed to increase simultaneously from their initial values of σ_{dc} and $\alpha_{\sigma c}$. For convenience the ratio $\Delta \alpha_{\sigma} / \Delta \sigma_{dn}$, which is called herein as the degree of stress rotation, was held constant.
- (b) σ_d held constant at its initial value σ_{dc} , while α_{σ} is increased over its initial value $\alpha_{\sigma c}$. This represents deformation by a mere rotation of principal stress directions, while their magnitudes stay constant.

Test ID	Initial State					Undrained Shear	
	σ_{mc}	e _c	b _c	$\alpha_{\sigma c}$	K _c	$\Delta lpha_{\sigma} / \Delta \sigma_{dn}$	Remarks
B01	200	0.907	0.0	0	1.0	1.75	At constant values of σ_m , b_c and increasing α_σ
B02	200	0.904	0.0	0	1.5	1.75	
B03	200	0.901	0.0	0	2.0	1.75	
B04	200	0.892	0.0	0	2.5	1.75	
B05	200	0.898	0.0	0	2.0	0.70	
B06	200	0.898	0.0	0	2.0	3.50	
B07	200	0.898	0.5	30	2.0	1.75	
B08	200	0.894	0.5	60	2.0	1.75	
B09	200	0.893	0.0	0	2.0	œ	At constant values of σ_m , b_c , σ_d and increasing α_σ
B10	200	0.887	0.5	0	2.0	∞	
B11	200	0.89	0.4	0	1.5	~	
B12	200	0.882	0.4	30	2.0	∞	
B13	200	0.892	0.4	45	2.0	~	
B14	200	0.896	0.5	45	2.0	~	
B15	200	0.889	0.4	45	2.0	- ∞	At constant values of σ_m , b_c , σ_d and decreasing α_σ
B16	200	0.886	0.4	45	2.0	- ∞	
B17	200	0.879	0.4	30	1.5	- ∞	
B18	200	0.896	0.5	90	2.0	- ∞	
B19	200	0.897	0.5	30	2.0	-1.75	At constant values of σ_m , b_c and decreasing α_{σ}
B20	200	0.891	0.5	60	2.0	-1.75	

Table 6.2: Initial states and loading paths in stress rotation tests

(c) Comparative response at identical initial stress states, and increasing σ_d , both with increasing as well as decreasing α_{σ} from its initial value $\alpha_{\sigma c}$. This was intended to find the influence of the sense of rotation of σ'_1 under increasing σ_d .

A finite value of $\Delta \alpha_{\sigma} / \Delta \sigma_{dn}$ represents simultaneous change in deviator stress and direction of principal stresses which often is the case in-situ. Smaller values of $\Delta \alpha_{\sigma} / \Delta \sigma_{dn}$ corresponds to lower levels of stress rotation, and $\Delta \alpha_{\sigma} / \Delta \sigma_{dn} = 0$ implies shearing under fixed principal stress directions. The value of $\Delta \alpha_{\sigma} / \Delta \sigma_{dn}$ in pure rotation tests with no change in deviator stress is infinity. It is recognized that the changes in deviator stress and principal stress direction in-situ will result in continually changing values of $\Delta \alpha_{\sigma} / \Delta \sigma_{dn}$. However, tests were carried out at fixed values of $\Delta \alpha_{\sigma} / \Delta \sigma_{dn}$ in order to isolate the influence of the degree of stress rotation.

External control of the stresses cannot be exercised in these tests if the sand cannot carry the applied stresses and strain softens. α_{σ} was held approximately constant at its value at the trigger of strain softening, whereas σ_d automatically assumed values consistent with the strain softening characteristics of the sand under this ambient α_{σ} . Prescribed stress paths were followed if the sand recovered from strain softening upon realising the quasi steady state.

6.4.1 Simultaneous increase in σ_d and α_σ

An axisymmetric initial state of stress ($\alpha_{\sigma c} = 0^{\circ}$) with equal intermediate and minor principal stresses ($b_c = 0$) is considered, and undrained shear is imposed under displacement controlled loading, both in axial and torsional directions. Figure 6.21 shows typical excursions from the



Figure 6.21 : Variation of control parameters σ_m , b, σ_d and α_σ during principal stress rotation

target parameters σ_m (= σ_{mc}), b (= b_c), σ_d and α_σ with ε_z under the control rate $\Delta \alpha_\sigma / \Delta \sigma_{dn} = 0.7$. As pointed out earlier, the prescribed stress path cannot be strictly followed during the strain softening phase of deformation (from A to B). This resulted in minor fluctuations in the targeted σ_m and b values. Since α_σ was held essentially constant at its value at the trigger of strain softening until its recovery, the ratio $\Delta \alpha_\sigma / \Delta \sigma_{dn}$ increased slightly during this phase of shearing.

The variations in the surface tractions $\sigma_{z_c} \sigma_{r_c} \sigma_{\theta_c}$ and $\tau_{z\theta}$ and the corresponding strains induced versus ε_z are shown in Figure 6.22. All specimens had essentially identical initial states, but were subjected to different degrees of stress rotation $\Delta \alpha_o / \Delta \sigma_{dn}$. The computed principal stresses, and principal strains for this series of tests are shown in Figure 6.23. The strain paths, though essentially linear at low strain levels, become progressively non linear as the deformation progresses (Figure 6.22b). The evolution of strain induced anisotropy during such shearing is due both to the increase in effective stress ratio, and the change in principal stress directions. Much larger non linearity noted in these strain paths compared to those in Figure 6.7(b) suggests that the strains accompanied by stress rotation induce anisotropy at a more accelerated rate than an increase in effective stress ratio alone. Moreover, the non linearity of strain paths increases systematically as the sand undergoes larger principal stress rotation (increasing $\Delta \alpha_o / \Delta \sigma_{dn}$) - a further illustration of the prominence of principal stress rotation over increase in stress ratio on the evolution of strain induced anisotropy.

The maximum shear stress - shear strain response and the effective stress paths for a series of tests on specimens at different initial static shear levels, but subjected to identical degree of stress rotation $\Delta \alpha_{\sigma} / \Delta \sigma_{dn} = 1.75$ are illustrated in Figure 6.24. Also shown in the figure are the direction



Figure 6.22(a) :

Variation of surface traction during principal stress rotation



Figure 6.22(b) :

Variation of radial, tangential and horizontal shear strain with vertical strain ε_z during principal stress rotation.



Figure 6.23(a) :

Variation of effective principal stresses and stress ratio with ε_z during principal stress rotation.



Figure 6.23(b):

Variation of principal strains during stress rotation against fixed direction strain ε_z .



Figure 6.24 : Influence of initial static shear stress level on subsequent undrained shear with simultaneous increase in σ_d and α_{σ} .

of major principal stress α_{σ} at the trigger of strain softening and at minimum undrained strength. It is clear that the higher the level of static shear level, K_e , the smaller is the increase in $\Delta\sigma_d$ until peak and the trigger of strain softening. For the selected degree of stress rotation, an increase in static shear promotes higher strain softening response, as quantified by the modified brittleness index. The associated increase in principal stress rotation $\Delta\alpha_{\sigma}$ at the trigger of strain softening, however, is least at the highest level of static shear. $\Delta\alpha_{\sigma}$ is a high 45° for the hydrostatic initial stress state, but is only about 5° for $K_e = 2.5$.

Figure 6.25 now illustrates the influence of the degree of principal stress rotation $\Delta \alpha_{\sigma}/\Delta \sigma_{dn}$ on the maximum shear stress-strain relationship and stress path in undrained shear at identical initial state. The sand was subjected to different degrees of stress rotation, quantified by $\Delta \alpha_{\sigma}/\Delta \sigma_{dn} =$ 0.70, 1.75, 3.50. As noted earlier, these prescribed paths can not be strictly enforced during the strain softening phase of the deformation, in case strain softening is triggered. An increase in the degree of stress rotation mobilises the smallest minimum undrained strength apparently because of the largest rotation incurred in the principal stress directions.

Both undrained rotation of principal stress directions and increase in σ_d induce excess pore pressure. This decreases the effective confining stress and hence increases the effective stress ratio. The inability of the sand then to carry the applied σ_d at a threshold effective stress ratio and α_{σ} triggers strain softening deformation. This is apparent from the results presented in the previous section, where the trigger of strain softening was shown to be dependent on α_{σ} at a certain σ_d . The flow deformation may be triggered due to an increase in effective stress ratio or increase in α_{σ} or a combination of the two. The sand strain softens until the minimum strength





Influence of degree of stress rotation $\Delta \alpha_{\sigma} / \Delta \sigma_{dn}$ on subsequent undrained shear with simultaneous increase in σ_{d} and α_{σ} .

at this threshold α_{σ} is reached. If the minimum strength at this new α_{σ} is smaller than the initial static shear, trigger of strain softening will lead to catastrophic flow failure. The characteristics of the stress strain curves during principal stress rotation with respect to the initial strain softening and subsequent strain hardening are similar to the limited liquefaction type of response observed in typical triaxial tests. Therefore, for response of the quasi steady state type, further straining beyond phase transformation is associated with strain hardening. The prescribed constant $\Delta \alpha_s / \Delta \sigma_{dn}$ path can once again be enforced during this phase of deformation.

6.4.2 Increase in α_{σ} only at constant σ_{d}

Undrained behaviour of sand at three different initial $\alpha_{\sigma c}$, but otherwise identical initial state $(\sigma'_m = 200 \text{ kPa}, b_c = 0.5, K_c = 2.0)$ under principal stress rotation prescribed by $\Delta \alpha_{\sigma} / \Delta \sigma_{dn} = \infty$ is shown in Figure 6.26. The changes in α_{σ} and the mobilisation of stress ratio R, in addition to the stress-strain relationships and the effective stress paths are also illustrated in the figure. The stress strain response may be noted to be highly dependent on the initial magnitude of $\alpha_{\sigma c}$. As explained earlier, constant deviator stress σ_d cannot be maintained if strain softening is triggered on account of the increases in stress ratio and α_{σ} .

The sand at initial $\alpha_{\sigma c} = 0^{\circ}$ strain softens only marginally, and therefore the deviator stress stays essentially constant as α_{σ} increases from 0° to about 35°. However, at initial $\alpha_{\sigma c}$ of 45° (or 90°) strain softening leads to true liquefaction type of response, and results in essentially steady state conditions at a shear strain of about 6%. This strain softening gets triggered


Figure 6.26: Deformation due to undrained rotation of principal stresses alone with no increase in σ_d at two initial K_c levels.

spontaneously when initial $\alpha_{\sigma c} = 90^{\circ}$, and the minimum undrained strength mobilised is much lower than the static shear stress. The sand strain softens because of its inability to carry the initial static shear stress of 67 kPa at the ambient confining stress, effective stress ratio, and α_{σ} at the initiation of strain softening. Strain softening at initial $\alpha_{\sigma c} = 45^{\circ}$, however, is triggered when α_{σ} increases to about 65° under constant σ_{d} , at a small shear strain of about 0.2%. The increase in α_{σ} due to principal stress rotation alone progressively aligns the direction of major principal stress toward the weaker horizontal plane. This decreases the corresponding minimum undrained strength (Figure 6.17), and triggers strain softening, once a threshold effective stress ratio is reached due to increasing pore pressure. The realised strength may even be less than the initial static shear stress of 67 kPa.

In contrast, at initial $\alpha_{\sigma c} = 90^{\circ}$, principal stress rotation decreases α_{σ} right from the start, and progressively aligns the major principal stress towards the stronger vertical plane. This, in fact, would result in an increase in the minimum undrained strength as α_{σ} deviates away from the weakest horizontal direction. The increase in effective stress ratio, nevertheless, triggers strain softening, and results in flow deformation, despite the decrease in α_{σ} .

The magnitude of excess pore pressure generated as a result of stress rotation may be noted to be the largest when $\alpha_{\sigma c} = 90^{\circ}$, and smallest when $\alpha_{\sigma c} = 0^{\circ}$. At initial $\alpha_{\sigma c} = 0^{\circ}$ rotation of the major principal stress direction from the vertical by about 35° induces about 100 kPa excess pore pressure. But approximately the same amount of rotation at initial $\alpha_{\sigma c} = 45^{\circ}$ induces much larger (150 kPa) excess pore pressure. When $\alpha_{\sigma c} = 90^{\circ}$, a mere 14° rotation of α_{σ} induces a high 160 kPa excess pore pressure even though σ'_1 is in fact moving away from the weakest horizontal direction. The inclination of major principal stress α_{σ} during steady state deformation for both initial $\alpha_{\sigma c}$ = 45° and 90° was essentially identical at about 79°. This was coincidental, but nevertheless reveals another important characteristic of the mobilised minimum undrained strength. Even though the sand reached steady state at $\alpha_{\sigma} = 79^{\circ}$ in both cases, the minimum undrained strengths mobilised are somewhat different. This occurs, despite a unique mobilised friction angle at steady state, due to higher excess pore pressure developed for initial $\alpha_{\sigma c} = 90^{\circ}$. Even though the difference is relatively minor (about 10% difference in the minimum undrained strength), this suggests stress path/stress history dependency of the minimum undrained strength.

The influence of the level of initial static shear on the undrained response under increasing α_{σ} from the initial $\alpha_{\sigma c} = 45^{\circ}$ is also included in Figure 6.26. At lower static shear of $K_c = 1.5$, very little strain (< 0.1%) develops even after α_{σ} is increased to 90°. The excess pore pressure developed was about 40 kPa. This increases the effective stress ratio to about 1.67, but not high enough to trigger any strain softening and therefore, the deviator stress can be held constant. In contrast, a higher initial K_c of 2.0, under otherwise identical initial state has resulted in catastrophic flow deformation. Therefore, the influence of rotation of principal stresses for a given $\alpha_{\sigma c}$ is likely to be much less severe at lower K_c levels.

The effective stress states at the trigger of strain softening in stress rotation tests are illustrated in Figure 6.27(a) for a range of initial K_c and $\alpha_{\sigma c}$ values. Also superimposed in the figure is the range of effective stress states noted in the constant α_{σ} tests. The stress ratio mobilised at the trigger of strain softening is found to gradually decrease with increasing α_{σ} . The associated mobilised friction angle is shown in Figure 6.27(b) as a function of the α_{σ} at the instant of the



Figure 6.27 : Trigger of strain softening: (a) Effective stress states, (b): variation of mobilised friction angled with α_{σ} .



Figure 6.28: Effective stress states at steady/quasi steady states during principal stress rotation.

peak deviator stress, together with the relationship noted in the constant α_{σ} tests. Φ_{CSR} mobilized at fixed direction of principal stresses forms an essential upper bound to the data. Stress rotation appears to induce strain softening at a slightly lower stress ratio compared to deformation at fixed direction of principal stresses at all levels of α_{σ} . The effective stress conditions corresponding to steady/quasi steady states in tests with principal stress rotation are shown in Figure 6.28, together with the measured unique SS/QSS line in constant α_{σ} tests. The mobilised friction angle at steady state or quasi steady state can now be noted to be essentially identical to the friction angles measured in constant α_{σ} tests. This indicates that the friction angle $\phi_{PT/SS}$ corresponding to minimum undrained strength is a material property, and unlike the friction angle mobilized at peak deviator strength, it is not influenced by the rotation of principal stresses.

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Figure 6.29 shows the variation of normalised minimum undrained strength, $S_{SS/QSS}/\sigma'_1$ with the direction α_{σ} of the major principal stress at the steady/quasi-steady state. Also superimposed in the figure is the relationship noted during undrained shear at fixed α_{σ} . The small scatter in the data at a given α_{σ} suggests that stress history/stress path may have a minor influence on the minimum undrained strength, as noted earlier in Figure 6.26. The influence of stress rotation on the normalised shear strength, for practical purposes, can be considered insignificant. This implies that the direction of the major principal stress α_{σ} at steady/quasi-steady state, and not that at the initial state ($\alpha_{\sigma c}$) is of concern in assessing the undrained strength.



Figure 6.29: Dependence of normalised minimum undrained strength on the direction of major principal stress α_{σ} .

6.4.3 Initial state and the direction of stress rotation

The direction of principal stress rotation during shear should have no influence on the resulting deformations if the principal stresses are initially aligned along $\alpha_{\sigma c} = 0^{\circ}$ or 90°. This is because these directions constitute principal axes of anisotropy. However, when the initial $\alpha_{\rm sc}$ is not along either of these axes, the imposed principal stress rotation would align the major principal stress either towards the weaker horizontal direction or along the stronger vertical direction. Many field problems involve shearing that includes principal stress rotation, and hence α_{σ} could either increase or decrease depending on the initial state. Analysis of the CANLEX test embankment using FLAC (1992) shows that the direction of major principal stress σ'_1 within the dam may rotate by as much as 60° from its initial direction. At the end of construction, σ'_1 directions were inclined at about 25° on either side of the vertical axes. When the reservoir was filled, α_{α} on the upstream slope of the embankment decreased and that on the downstream increased as a result of pressure build up (Puebla 1998). In addition, the superposition of earthquake induced shear stresses on these initial non-axisymmetric stress state may either increase or decrease the inclination of σ'_1 depending on the direction of the shear stress increment (Figure 6.30). The following data is intended to examine the influence of the direction of similar stress rotation on undrained behaviour, and its dependency on the initial stress state.

The development of excess pore pressure due to rotating principal stresses under different initial and undrained loading conditions is shown in Figure 6.31 against the magnitude of rotation $\Delta \alpha_{\sigma}$. Sand at different initial states was subjected to the same degree of stress rotation. The



Figure 6.30 : The direction of stress rotation in-situ: Increasing and decreasing α_{σ} from initial $\alpha_{\sigma c}$.



Figure 6.31 : Range of excess pore pressure development for increasing and decreasing α_{σ} at different initial static shear K_{c} .

initial states considered were $\sigma'_{mc} = 200 \text{ kPa}$, $b_c = 0.5$, $\alpha_{\sigma c} = 45^\circ$, and $K_c = 2.0$ and 1.5. Undrained principal stress rotation was imposed using $\Delta \alpha_{\sigma} / \Delta \sigma_{dn} = \infty$ at initial $\alpha_{\sigma c} = 45^\circ$. At $K_c = 2.0$, an increase in α_{σ} causes much higher excess pore pressure than a decrease in α_{σ} . Increasing α_{σ} from 45° to about 65° induces about 150 kPa of excess pore pressure, in contrast to only about 50 kPa when α_{σ} is decreased from 45° to zero. For a given initial $\alpha_{\sigma c}$, an increase in α_{σ} aligns σ'_1 closer to weaker horizontal direction, whereas a decrease towards zero aligns it closer to the stronger vertical direction. Hence larger excess pore pressures develop when α_{σ} increases towards 90° compared to when it decreases to zero.

Excess pore pressure development is smaller for lower initial static shear $K_c = 1.5$. Increasing α_{σ} from 45° to about 90° induces about 40 kPa of excess pore pressure, in contrast to only about 20 kPa when α_{σ} is decreased from 45° to zero. The magnitudes of the strains induced due to this stress rotation are much smaller (<0.1%) compared to those at the initial $K_c = 2$. As noted earlier, this implies that the influence of stress rotation is less severe at low initial static shear, regardless of whether α_{σ} is increased or decreased.

Figure 6.32 now compares the undrained response of sand at identical initial state ($e_c \approx 0.898$, $\sigma'_m = 200$ kPa, $b_c = 0.5$, $K_c 2.0$ and $\alpha_{\sigma c} = 30^\circ$), subjected to identical degree of stress rotation $\Delta \alpha_{\sigma} / \Delta \sigma_{dn} = 1.75$, for increasing α_{σ} with that of decreasing α_{σ} . The response under increasing α_{σ} may be noted to be strain softening compared to strain hardening for decreasing α_{σ} . Under increasing α_{σ} the sand strain softens at $\alpha_{\sigma} \approx 35^\circ$, a rotation of merely 5°. In contrast, when α_{σ} decreases from 30°, σ'_1 progressively gets aligned with stronger vertical direction until it becomes zero. The sand in this case reached its phase transformation state at about 2% shear strain, (at $\alpha_{\sigma} \approx 10^\circ$), and on subsequent shearing it developed negative excess pore pressure due to strain hardening.

Comparative undrained tests, similar to those shown in Figure 6.32, but at an initial $\alpha_{\sigma c} = 60^{\circ}$ are shown in Figure 6.33. In contrast to the results shown in Figure 6.32, principal stress rotation induces strain softening in both cases, regardless of whether α_{σ} is increased or decreased from its initial value of 60°. The inability of the sand to carry the ambient deviator stress when sheared



Figure 6.32 : Influence of the sense of principal stress rotation on the undrained response at initial $\alpha_{\sigma c} = 30^{\circ}$.



Figure 6.33 : Influence of the sense of principal stress rotation on the undrained response at initial $\alpha_{\sigma c} = 60^{\circ}$.

undrained at a threshold effective stress ratio (perturbation in the form of excess pore pressure) is the cause for strain softening to trigger when α_{σ} is decreased. However, when α_{σ} is increased, both the threshold effective stress ratio and the lower minimum undrained strength at the new higher α_{σ} contributes to the trigger of strain softening.

The minimum undrained strength mobilised in these tests decreases with increasing α_{σ} , and its value at a given α_{σ} level is consistent with the data shown for initial $K_c = 2$ in Figure 6.17. Furthermore, the friction angle mobilised at steady/quasi steady state is also essentially identical to that observed in the tests with fixed principal stress directions.

6.5 Summary

Results of hollow cylinder torsional shear tests on water pluviated Fraser River sand are presented in this chapter. The undrained behaviour is assessed both under fixed and changing principal stress directions including simultaneous change in σ_d and α_{σ} . The initial effective stress state of the sand, characterized by K_c and α_{σ} , plays a major role in the subsequent undrained behaviour of the sand at a given confining stress level. Test results clearly demonstrate:

• The behaviour of water deposited sands transforms from being dilative to contractive as the direction of major principal stress during shear changes from the direction of deposition ($\alpha_{\sigma} = 0^{\circ}$) to the direction of bedding planes ($\alpha_{\sigma} = 90^{\circ}$) regardless of the level of initial static shear.

- Undrained conditions may trigger a sudden flow in sand, that is otherwise stable under drained conditions on account of the initial static shear. The peak shear strength and the friction angle mobilised at peak are dependent on the direction of the principal stresses. The potential for flow deformation dramatically increases with α_σ.
- Contrary to the commonly held steady state concepts, the minimum undrained strength of a sand at a given void ratio is not uniquely related to its void ratio. It is dependent on the direction of the major principal stress and to a lesser extent on the initial static shear. However, the friction angle corresponding to minimum strength is essentially constant, and appears to be a unique material property.
- Rotation of principal stress directions alone at constant magnitude can trigger strain softening deformation due to the generation of excess pore pressure. The susceptibility to liquefaction is most severe at high initial K_e and α_{re} levels.
- The undrained behaviour under simultaneous increase in deviator stress and the major principal stress direction depends on the relative magnitudes of the degree of stress rotation and the change in deviator stress. The friction angle mobilised at peak, ϕ_{CSR} due to stress rotation at a given α_{σ} is about 2 to 3° smaller than the corresponding friction angle mobilised under fixed principal stress directions. The friction angle mobilised at steady/quasi steady state, however, is not influenced by the principal stress rotation.
- The minimum undrained strength realised under rotation of principal stresses depends on the direction of major principal stress at the instant of steady state or quasi steady state.

It is also influenced by the stress path/stress history that brought the sand to its current state, but to a lesser extent.

These tests clearly demonstrate that the measured undrained properties of sands are highly dependent on their initial state characterised by effective confining and static shear stresses, void ratio and fabric. Therefore it is of critical importance to simulate conditions in the laboratory tests that duplicate those anticipated in the field problem. This will lead to a more credible assessment of the undrained response of sands.

Chapter 7

Summary and Conclusions

The influence of fabric, initial effective stress state and stress path during undrained shear on the behaviour of sands has been studied using triaxial, simple shear and hollow cylinder torsional shear tests. Triaxial and simple shear represent specific loading paths with independent control of two stress parameters. Torsional shear tests, however, enable independent control of each of the three principal stresses and the directions of two in one plane. Proper cognizance of the influence of membrane penetration on undrained response is taken in order to assess liquefaction susceptibility under truly undrained conditions.

The new method proposed for determining the magnitude of membrane penetration has several advantages over the existing methods. It enables specimen and membrane specific corrections to be made. This method does not make any assumptions regarding the constitutive behaviour of the sand, and is non destructive. The following conclusions may be drawn from this component of the investigation.

- Membrane penetration induced volume changes in a given sand was found proportional to the logarithm of the effective confining stress.
- The normalised unit membrane penetration may be approximated as a linear function of D₅₀ for average grain sizes ranging from about 0.1mm to 2.0 mm.

- The partially drained state induced if membrane penetration is not accounted for may transform a strain softening sand into a strain hardening one. The strain hardening response of the coarsest sand tested ($D_{50} = 0.90$ mm) in conventional undrained tests transformed into strain softening type if the test was truly undrained (i.e. constant volume).
- The available post correction methods exaggerate the influence of membrane penetration on undrained response. The error in the induced excess pore pressure in a conventional undrained tests is typically only about 10% for medium/fine ($D_{50} < 0.30$ mm) sands.

The influence of the fabric on undrained response was assessed using comparative tests on undisturbed in-situ frozen sand and specimens of the same sand reconstituted by different techniques. Undisturbed sand specimens were sampled from four different sites; two alluvial deposits in British Columbia, and two tailings sands from Alberta. Reconstituted specimens were prepared using bulk sands from each of these four sites, and also from sand dredged from the Fraser River. The fabric studies indicate that:

 The technique of in-situ ground freezing yields very high quality undisturbed sand specimens in saturated sands. The negligible amount of axial and volumetric strains during thawing and reinstatement of the in-situ stresses was taken as evidence in its support. Frozen specimens suffer only minimal disturbance if thawed at low effective stresses and subsequently consolidated to the in-situ stresses. In-situ ground freezing, however, is an expensive endeavour, and might not be economically justifiable except in critical/large projects.

- All undisturbed sands tested were found to be inherently anisotropic. Even loose sands rarely strain softened in triaxial compression, but did so invariably and realised a quasi steady state, prior to the commencement of dilation, or an eventual steady state in triaxial extension.
- Loose undisturbed sands marginally strain softened in simple shear. The minimum undrained strength in simple shear was in between the values noted in triaxial compression and extension.
- Water pluviated sands also exhibit direction dependent response similar to those of undisturbed sands on account of their inherent anisotropy. Moist tamped specimens, however, strain softened and realized essentially steady state both in triaxial compression and extension.
- Even the loosest water pluviated Syncrude sand was found dilative at an initial void ratio of 0.768 and $\sigma'_{ve} = 200$ kPa in simple shear. But, after air pluviation at the same void ratio and effective stress, it marginally strain softened to an essential steady state. The same sand moist tamped to an identical initial state strain softened and realised steady state at much lower strength. This implies that the fabric effects are not destroyed at large strains as claimed by the proponents of the steady state concepts even when the sands strain soften. The fact that a sand at the same void ratio can realize steady state at two distinct values of stresses clearly indicates that the initial fabric does play an important role on undrained behaviour and the steady state strength of sands. In general, the undrained behaviour at identical void ratio and effective stress states under a given

loading mode is profoundly affected by the method of specimen reconstitution which controls the ensuing fabric.

- Specimens reconstituted by water pluviation to essentially identical void ratio and effective stress state of corresponding undisturbed specimens exhibit characteristics very similar to those of undisturbed sands. This was noted under both static and cyclic loading conditions. This equivalence was both qualitative and quantitative under triaxial compression, extension and simple shear states.
- The post liquefaction stress strain curves of undisturbed sands were found to posses three distinct phases of deformation. The post liquefaction deformation progressed along the line of maximum obliquity. This is consistent with the post liquefaction behaviour of water pluviated sands that have been reported in the literature. The minimum undrained strength under virgin loading does not appear to limit the strength of the undisturbed sand in post liquefaction loading.
- Static, cyclic and post liquefaction characteristics of water pluviated sands can be confidently assigned to alluvial or hydraulic fill in-situ sand deposits. Water pluviation thus provides an attractive economical alternative to the very expensive undisturbed samples using ground freezing techniques.

The influence of initial non axisymmetric stress state on liquefaction susceptibility was assessed using water pluviated fabric in the hollow cylinder torsional shear device. Because of the demonstrated equivalence of the behaviour of water pluviated and undisturbed sands, findings from this study can be confidently applied to alluvial and hydraulic fill in-situ sands. The undrained behaviour was assessed both under fixed and continually changing principal stress directions. The initial effective stress state of the sand, characterized by K_c and α_{σ} plays a critical role in the subsequent undrained behaviour of sand at a given confining stress level. The following conclusions may be drawn from the data presented.

- Water deposited sands, at a given initial state systematically transform from being dilative to contractive as the direction of major principal stress changes from the direction of deposition ($\alpha_{\sigma} = 0^{\circ}$) to the direction of the bedding planes ($\alpha_{\sigma} = 90^{\circ}$), when sheared undrained. This is a clear manifestation of the inherent anisotropy.
- Liquefaction potential increased with increase in static shear K_c and inclination of major principal stress to the vertical α_{σ} . Even though an increase in static shear results in marginally higher peak shear strength, the minimum undrained strength mobilised might even be lower than the initial static shear, especially under high K_c and α_{σ} . As a result, undrained loading may trigger a sudden flow in a sand deposit that is otherwise stable if fully drained. In addition to the peak shear strength, the friction angle mobilised at the trigger of strain softening is also dependent on the direction of the principal stresses.
- The minimum undrained strength of a sand is highly dependent on the void ratio, effective confining stress level, loading direction, and the fabric, and to a lesser extent on the initial static shear. This conflicts with the prevalent belief of a unique relationship between steady state strength and void ratio. The concept of a unique steady state was formulated based on triaxial compression tests on moist tamped sands, and it appears those concepts are not applicable to the inherently anisotropic alluvial and hydraulic fill sands. The

friction angle corresponding to the minimum strength, however, is essentially constant and appears to be a material property.

- A mere rotation of principal stress directions with no increase in σ_d may trigger strain softening and lead to flow failure. Both the initial static shear level and the direction of σ'₁ plays an important role on the undrained response due to principal stress rotation only. The undrained behaviour under simultaneous change in deviator stress σ_d and principal stress directions α_σ depends on the initial direction of principal stresses, shear stress level and the sense of principal stress rotation.
- Both an increase in effective stress ratio, and change in principal stress directions induce anisotropy. Principal stress rotation was noted to accelerate the evolution of strain induced anisotropy.
- The minimum undrained strength mobilised under rotation of principal stresses is primarily dependent on the direction of major principal stress at steady or quasi steady state. ϕ_{CSR} at the trigger of strain softening with principal stress rotation is about 2 to 3° smaller than the corresponding angle under fixed principal stress directions. However, the friction angle mobilised at steady/quasi steady state is not influenced by principal stress rotation, and appears a unique property of the sand.

Test data reported herein clearly demonstrate that the undrained response of in-situ and water deposited sands under a given stress path is highly dependent on initial effective stress state, void ratio and the fabric, and that under a given initial state on the undrained stress path. Therefore it is essential to simulate actual field loading conditions both prior to and during shear in laboratory tests in order to confidently characterize their in-situ response. The fabric of in-situ alluvial and hydraulic fill sands appear to be similar to that of water pluviated sands, in that they both are inherently anisotropic and possess similar characteristics under static, cyclic and post liquefaction loading modes. This opens the possibility of using reconstituted instead of very expensive in-situ frozen undisturbed samples for characteristing in-situ behaviour.

7.1 Recommendations for future research

This thesis highlights the importance of using specimen reconstitution techniques that simulate the in-situ fabric together with duplicating the in-situ stress states and loading conditions in laboratory assessment of liquefaction susceptibility of in-situ sands. Principal stress directions and the presence of static shear at the end of consolidation is shown to profoundly influence the monotonic undrained response. The influence of undrained principal stress rotation is shown to be complex and dependent on the initial state. Further research in the following related topics will complement the findings reported in this thesis.

- Extend the fabric studies to assess the relevance of moist tamped specimens to compacted fills in-situ. Issues regarding accessible density states and the associated mechanical behaviour need to be resolved.
- Assessment of the influence of initial stress state on specimen density and confining stress level. This is of particular interest since denser sands are less anisotropic than loose ones.

- Influence of initial non-axisymmetric stress states on the cyclic resistance of sands at different stress levels. This will complement the current understanding of the effects of static shear and confining stress level on liquefaction resistance. Successful modelling of the cyclic response of in-situ sands under generalised stress state requires this information.
- The effect of continuous principal stress rotation on the cyclic resistance of sands at different initial states and the relevance of cyclic triaxial and simple shear resistance to the sands in-situ.
- Influence of changing intermediate principal stress parameter during static and cyclic undrained shear.
- The effect of generalised three dimensional loading conditions on the post liquefaction response of sands in order to assess the degree of anisotropy in liquefied sands.

REFERENCES

- Airey, D. W., Budhu, M., and Wood, D. M., (1985), "Some aspects of behaviour of soils in simple shear", Developments in soil Mechanics and Foundation Engineering: Stress-strain modelling of soils, Eds: Banerjee, P. K. and Butterfield, R., Elsevier, London, England, pp. 185-213.
- Alarcon-Guzman, A., Leonards, G. A., and Chameau, J. L., (1988), "Undrained monotonic and cyclic strength of sands.", ASCE Journal of Geotechnical Engineering, Vol. 114, No. 10, pp. 1089-1109.
- Ambraseys, N. N., and Sarma, S. K., (1967), "The response of earth dams to strong earthquakes.", Geotechnique, Vol. 17, No. 3, pp. 181-213.
- Ansal, A.M. and Erken, A., (1996), "Post correction procedure for membrane compliance effects on pore pressure", Journal of Geotechnical Engineering, Vol. 122, No. 1, pp. 27-38.
- Arthur, J. R. F., and Menzies, B. K., (1968), "A new soil tesing apparatus: Correspondence", Geotechnique, Vol. 18, No. 2, pp. 271-272.
- Arthur, J. R. F., and Menzies, B. K., (1972), "Inherent anisotropy in sand", Geotechnique, Vol. 22, No. 1, pp. 115-128
- Arthur, J. R. F., Chua, K. S. and Dunstan, T., (1979), "Dense sand weakened by continuous principal stress direction rotation", Geotechnique, Vol. 29, No. 1, pp. 91-96.
- Arthur, J. R. F., Chua, K. S., Dunstan, T., and Rodriguez del C. J. I., (1980), "Principal stress rotation: a missing parameter", Journal of the Geotechnical Engineering Division, Vol. 106, No. 4, pp. 419-433.

- Arthur, J. R. F., Bekenstein, S., Germanie, J. T., and Ladd, C. C., (1981), "Stress path tests with controlled rotation of principal stresses", Laboratory shear strength of soil, ASTM Special Technical Publication 740, Eds. Yong, R. N and Townsend, F. C., Philadelphia 25 June 1980, pp. 516-540.
- ASTM, 1991a, Standard test methods for maximum index density of soils using a vibratory table (D4253-83), In 1991 Annual book of ASTM standards, Sect 4, Vol. 04-08, ASTM, Philadelphia, PA., pp. 589-600
- ASTM, 1991b, Standard test methods for minimum index density of soils and calculation of relative density (D4254-83), In 1991 Annual book of ASTM standards, Sect 4, Vol. 04-08, ASTM, Philadelphia, PA., pp. 601-607.
- BC Hydro Duncan Dam, (1993), "Liquefaction assessment and seismic stability", Report No. H2599, Burnaby, B.C., Canada
- Baldi, G., and Nova, R., (1984), "Membrane penetration effects in triaxial testing", Journal of Geotechnical Engineering, Vol. 110, No. 3, pp. 403-420.
- Banerjee, N. G., Seed, H. B., and Chan, C. K., (1979), "Cyclic behaviour of dense coarse-grained materials in relation to the seismic stability of dams", Earthquake Engineering Research Center Report UCB/EERC-79/13, University of California, Berkeley, CA.
- Bardet, J. P., and Davis, C. A., (1996), "Performance of San Fernando Dams During 1994 Northridge Earthquak", Journal of Geotechnical Engineering, Vol. 122, No. 7, pp. 554-564
- Bardet, J. P., and Kapuskar, M., (1993), "Liquefaction sand boils in San Francisco during 1989 Loma Prieta Earthquake.", Journal of Geotechnical Engineering, Vol. 119, No. 3, pp. 543-562

- Barends, F. B. J., Siddharthan, R., Towhata, I., Zafir, Z., Kouki, Z., and Hiroyuki, Y., (1992),
 "Oscillatory pore pressure and liquefaction in seabed induced by ocean waves.", Soils and
 Foundations, Vol. 32, No. 3, pp. 190-196.
- Beaty, M. H., and Byrne, P. M., (2000), "A synthesized approach for modeling liquefaction and displacements", Proc. International FLAC symposium on Numerical Modeling in Geomechanics, Minneapolis, USA, Eds: Detournay, C. and Hart, R., Sept 1-3, 1999,
- Been, K., (1998), "The critical state line and its application to soil liquefaction", Proc.
 International Symposium on the Physics and Mechanics of Soil Liquefaction, Eds. Lade,
 P. V., and Yamamuro, J. A., Baltimore, MD, Sept 1998, pp. 195-204.
- Been, K. and Jefferies, M. G., (1985), "A state parameter for sands", Geotechnique, Vol. 35, No. 2, pp. 99-112.
- Been, K., Jefferies, M. G., and Hachey, J., (1991), "The critical state of sands", Geotechnique, Vol. 41, No. 3, pp. 365-381.
- Bishop, A. W., (1971), "Shear strength parameters for undisturbed and remoulded soil specimens", Proc. Roscoe memorial symposium on stress-strain behaviour of soils", Cambridge University, pp. 3-58.
- Bishop, A. W. and Henkel, D. J., (1962), "The measurement of soil properties in the triaxial test", 2nd Edition, Edward Arnold Ltd, London, 227p.
- Bjerrum, L., (1972), "Embankments on soft ground", Performance of Earth and Earth supported structures, Lafayette, Indiana, pp. 1-54.
- Bjerrum, L. and Landva, A., (1966), "Direct simple shear tests on a Norweigian clay", Geotechnique, Vol. 16, No. 1, pp. 1-20.
- Boulanger R. W., Mejia, L. H., and Idriss, I. M, (1997), "Liquefaction at Moss Landing during Loma Prieta earthquake.", Journal of Geotechnical and Geoenvironmental Engineering, Vol. 123, No. 5, pp. 453-467.

- Brewer, R. (1964), Fabric and Mineral analysis of soils, John Wiley and Sons Inc. New York, pp.129-158.
- Brooms, B. B., (1980), "Soil sampling in Europe: state of the art", Journal of the Geotechnical Engineering Division, ASCE, Vol. 106, No. GT1, pp. 65-97.
- Byrne, P. M., (1998), Personal Communications.
- Byrne, P. M., Jitno, H. and Haile, J., (1992), "A procedure for predicting the seismic response of tailings impoundments", Proc. Symposium on Geotechnical and Natural hazards, GeoHazards '92, Vancouver, Canada, pp. 281-289.
- Byrne P.M., Finn, W.D.L., and Ventura, C.E., (1996), "The Hyogo-ken Nanbu (Kobe) earthquake of 17 January 1995: Seismological aspects and ground motions", Canadian Journal of Civil Engineering, Vol. 23, No. 3, pp.771-777.
- Byrne, P. M., Puebla, H., Chan, D. H., Soroush, A., Morgenston, N. R., Cathro, D. C., Gu, W. H., Philiips, R., Robertson, P. K., Hofmann, B., Wride, C. E., Sego, D. C., Plewes, H. P., List, B. R. and Tan, S., (2000), "CANLEX Full-scale experiment and modelling", Canadian Geotechnical Journal, Vol. 37, No. 3, pp. 543-562.
- Casabarian, A. O. P. and Jamal, A. K., (1963), "Study of failure envelope of soils: Discussion", ASCE Journal of the Soil Mechanics and Foundation Engineering division, Vol. 89, No. SM4, pp.243-243.
- Casagrande, A., (1975), "Liquefaction and cyclic deformation of sands: a critical review", Proc. Fifth Pan-American Conference on Soil Mechanics and Foundation Engineering, pp. 79-133.
- Casagrande, A. and Carrillo, N., (1944), "Shear failure of anisotropic materials", Journal of the Boston Society of Civil Engineers, Vol. 31, No. 4, pp. 122-135.
- Castro, G., (1969), "Liquefaction of sands", PhD Thesis, Harvard University, Cambridge, Massachusettes, .231p.

- Castro, G., and Poulos, S. J., (1977), "Factors affecting liquefaction and Cyclic mobility", Journal of the Geotechnical Engineering division, ASCE, Vol. 103, No. 6, pp. 501-516.
- Castro, G., Poulos, S. J., France, J. W., and Enos, J. L., (1982), "Liquefaction induced by cyclic loading", Report to National Science Foundation, Washington, DC.
- Castro, G., Poulos, S. J., Leathers, F. F., (1985), "Re-examination of slide of lower San Fernando dam", Journal of Geotechnical Engineering, Vol. 111, No. GT9, pp. 1093-1107.
- Chang, N.Y., Hseih, N.P., Samuelson, D.L. and Horita, M., (1981), "Static and cyclic behaviour of Monterey #0 sand", Proc. Third Microzonation Conference, Seattle, Washington, pp. 929-944.
- Chern, J. C., (1981), "Effect of static shear on resistance to liquefaction", MASc Thesis, University of British Columbia, Vancouver, Canada, 64p.
- Chern, J. C., (1985), "Undrained response of sands with emphasis on liquefaction and cyclic mobility", PhD Thesis, University of British Columbia, Vancouver, Canada, 213p.
- Choi, J.W., and Ishibashi, I., (1992), "An experimental method for determining membrane penetration", Geotechnical Testing Journal, Vol. 15, No. 4, pp. 413-417.
- Chung, E. K. F., (1985), "Effects of stress path and prestrain history on the undrained monotonic and cyclic loading behaviour of saturated sand", MASc Thesis, University of British Columbia, Vancouver, Canada, 101p.
- Cole, E. R. L., (1967), "The behaviour of soils in the simple shear apparatus", PhD Thesis, Cambridge University, England.
- Cooling, L. F. and Smith, B. A., (1936), "The shearing resistance of soils", Journal of the Institution of Civil Engineers, Vol. 3, pp. 333-343.
- DeGregorio, V. B., (1990), "Loading systems, sample preparation, and liquefaction", Journal of Geotechnical Engineering, Vol. 116, No. 5, pp. 805-821.

- DeGroot, D. J., Germanie, J. T., and Ladd, C. C., (1994), "Effects of non uniform stresses on measured DSS stress-strain behaviour", Journal of the Geotechnical Engineering division, ASCE, Vol. 120, No. 5, pp. 892-912.
- Duncan, J. M. and Dunlop, P., (1969), "Behaviour of soils in simple shear tests", Proc. Seventh International conference on Soil Mechanics and Foundations Engineering, Vol. 1, pp. 101-108.
- Dyvik, R., Berre, T., Lacasse, S., and Raadim, B., (1987), "Comparison of truly undrained and constant volume direct simple shear tests", Geotechnique, Vol. 37, No. 1, pp. 3-10.
- Dyvik, R., and Hoeg, K., (1998), "Comparison of tests on undisturbed and reconstituted silt and silty sand", Proc. International workshop on the Physics and Mechanics of soil Liquefaction, Eds: Lade and Yamamuro, Maryland, USA, Sept 10-11, pp. 159-167
- El-Sohby, M. A. and Andrawes, K. Z., (1972), "Deformation characteristics of granular materials under hydrostatic compression", Canadian Geotechnical Journal, Vol. 9, No. 3, pp. 338-350.
- El Sohby, M. A., Andrawes, K. Z., and Menzies, B. K., (1972), "Inherent anisotropy in a sand: Discussion", Geotechnique, Vol. 22, No. 3, pp. 537-541
- Emery, J. J., Finn, W. D. L., and Lee, K. W., (1973), "Uniformity of saturated sand samples", Proc. Relative density, geotechnical projects, cohesionless soils, ASTM Special Technical Publication 523, pp. 182-194.
- Eshtehard, A. A., (1996), "Triaxial testing on very loose sands for flow liquefaction analyses", MSc Thesis, University of Alberta, Edmonton, Canada, 160p.
- Fear, C. E., (1996), "In-situ testing for liquefaction evaluation of sands", PhD Thesis, University of Alberta, Edmonton, Canada, 412p.

- Finn, W. D. L. and Vaid, Y. P., (1977), "Liquefaction potential from drained constant volume simple shear tests", Proc. Sixth World conference on Earthquake Engineering, Vol. 1, pp. 2157-2162.
- Finn, W. D. L., Pickering, J., and Bransby, P. L., (1971), "Sand liquefaction in triaxial and simple shear test", Journal of the Soil Mechanics and Foundations division, ASCE, Vol. 97, No. 4, pp. 639-659.
- Finn, W. D. L., Vaid, Y. P., and Bhatia, S. K., (1978), "Constant volume cyclic simple shear testing", Proc. Second International conference on Microzonation for safer construction, Vol. 2, pp. 839-851.
- Finn, W. D. L., Yogendrakumar, M. and Yoshida, N., (1986), "Response of 2D embankments to seismic loading - TARA3.", UBC Soil Mechanics Series, University of British Columbia, Vancouver, Canada..
- Finn W.D.L., Byrne, P.M., Evans S., and Law, T., (1996), "Some geotechnical aspects of the Hyogo-ken Nanbu (Kobe) earthquake of January 17, 1995", Canadian Journal of Civil Engineering, Vol. 23, No. 3, pp. 778-796.
- FLAC (1992), "Fast Lagrangian Analysis of Continua: Version 3.2", Itasca Consulting Group Inc., Suite 210, 1313 5th Street SE, Minneapolis, Minnesota 55414, USA.
- Frydman, S., Alpan, I. and Zeitlan, J. G. (1971), "Sress deformation behaviour of sand studied in in hollow cylinder and triaxial apparatus", Proc. Fourth Asian Regional Conference in Soil Mechanics and Foundations Engineering, Bangkok, Vol. 1, pp. 17-23.
- Frydman, S., Zeitlan, J.G., and Alpan, I., (1973), "The membrane effect in triaxial testing of granular soils", Journal of Testing and Evaluation, Vol. 1, No. 1, pp. 37-41.
- Garrison, R. E., Luternuer, J. L., Grill, E. V., Macdonald, R. D. and Murray, J.W., (1969), "Early diagenetic cementation of recent sands, Fraser River Delta, British Columbia", Sedimentology, Vol 12, No. 1, pp 27-46.

- Graham, J., and Hovan, J.,M., 1986, Stress characteristics for bearing capacity in sand using a critical state model., Canadian Geotechnical Journal, Vol. 23, No. 2, pp. 195-202.
- Green, G. E., (1971), "Strength and deformation of sand measured in an independent stress control cell", Proc. Roscoe memorial symposium on stress-strain behaviour of soils", Cambridge University, pp. 285-323.
- Hamada, M. and Wakamatsu, K., (1998), "Liquefaction-Induced Ground Displacement Triggered by Quaywall Movement", Special Issue on Geotechnical Aspects of the January 17, 1995, pp.85-96.
- Hanzawa, H., (1980), "Undrained strength and stability analysis for a quick sand.", Soils and Foundations, Vol. 20, No. 2, pp. 17-29.
- Harder, L. F. and Stewart, J. P., (1996), "Failure of Tapo Canyon Tailings Dam", ASCE Journal of Performance of Constructed Facilities, Vol. 10, No. 3, August 1996, pp. 109-114
- Haruyama, N., (1981), "Anisotropic deformation-strength characteristics of an assembly of spherical particles under 3D stresses", Soils and Foundations, Vol. 21, No. 4, pp. 41-55.
- Hatanaka, M., Sugimoto, M., and Suzuki, Y., (1985), "Liquefaction resistance of two alluvial volcanic soils sampled by in-situ freezing.", Soils and Foundations, Vol. 25, No. 3, pp. 49-63.
- Hight, D. W., Gens, A., and Symes, M. J., (1983), "The development of a new hollow cylinder apparatus for investigating the effects of principal stress rotation in soils.", Geotechnique, Vol. 33, No. 4, pp. 355-383.
- Hofmann, B. A., (1997), "In situ ground freezing to obtain undisturbed samples of loose sand for liquefaction assessment", PhD Thesis, University of Alberta, Edmonton, Canada, 485p.
- Hofmann, B. A., Sego, D. C., and Robertson, P. K., (1995), "Undisturbed sampling of loose sand using ground freezing", Proc. 48th Canadian Geotechnical Conference, Vancouver, BC. pp.197-204.

- Holzer, T. L., Bennett, M. J., Ponti, D. J., and Tinsley, J. C., (1999), "Liquefaction and Soil Failure during 1994 Northridge Earthquake", Journal of Geotechnical and Geoenvironmental Engineering, Vol. 125, No. 6, pp. 438-452.
- Hughes, J. M. O., (1996), "Pressuremeter investigation, data summary and preliminary interpretation", CANLEX Technical Report, Hughes InSitu Engineering Inc., Vancouver, Canada.
- Hvorslev, M. J., (1939), "Torsion shear tests and their place in the determination of shearing resistance of soils", Proc. ASTM 42nd annual meeting, Atlantic City, pp. 999-1022.
- Hvorslev, M. J, (1949), "Subsurface exploration and sampling of soils for Civil Engineering purposes", Report on a research project of ASCE, U. S. Army Engineer experiment station, Vickburg, 521p.
- Ishihara, K., (1993), "Liquefaction and flow failure during earthquakes", Geotechnique, Vol. 43, No. 3, pp. 351-415.
- Ishihara, K., Tatsuoka, F. and Yasuda, S., (1975), "Undrained deformation and liquefaction of sands under cyclic stresses", Soils and Foundations, Vol. 15, No. 1, pp. 29-44.
- Ishihara, K., Okusa, S., Oyagi, N. and Ischuk, A., (1990), "Liquefaction-induced flow slide in the collapsible loess deposit in Soviet Tajik.", Soils and Foundations, Vol. 30, No. 4, pp. 73-89.
- Ishihara, K., Yasuda, S., and and Nagase, H., (1996), "Soil characteristics and ground damage.", Soils and Foundations, Special Issue #1 on the Hyogoken-Nambu earthquake of January 17, 1995, Vol. 36, S11, pp. 109-118.
- Ishihara, K., Cubrinovski, M., and Nonaka, T., (1998), "Characterization of undrained behaviour of soils in the reclaimed area of Kobe", Soils and Foundations, Special Issue #2 on the Hyogoken-Nambu earthquake of January 17, 1995, Vol. 38, SI2, pp. 33-46.

- Jefferies, M. G., 1993, Nor-Sand; a simple critical state model for sand., Geotechnique, Vol. 43, No. 1, pp. 91-103
- Jeyapalan, J. K., Duncan, J. M. and Seed, H. B., (1983), "Analyses of flow failures of mine tailings dams.", Journal of Geotechnical Engineering, Vol. 109, No. 2, pp. 150-171.
- Kiekbusch, M. and Schuppener, B., (1977), "Membrane penetration and its effect on pore pressures", Journal of the Geotechnical Engineering Division, Vol. 103, No. 11, pp. 1267-1279
- Kjellman, W., (1936), "Report on an apparatus for consummate investigation of the mechanical properties of soils", Proc. First International Conference on Soil Mechanics and Foundations Engineering, Vol. 2, pp. 16-20.
- Kjellman, W., (1951), "Testing the shear strength of clay in Sweden", Geotechnique, Vol. 2, No. 3, pp. 225-232.
- Ko, H. Y. and Scott, R. F., (1967), "A new soil testing apparatus", Geotechnique, Vol. 17, No. 1, pp. 40-57.
- Kollbrunner, C. F., and Langer, C, (1939), "Loading tests and test borings: The obtaining of undisturbed samples", Privat-Gesellschaft fur Bodenforshung und Erdbaumechanik, Bericht No. 2, Zurich, Switzerland. (In German)
- Konrad, J. -M., & Pouliet, N., (1997), "Ultimate state of reconstituted and intact samples of deltaic sand", Canadian Geotechnical Journal, Vol. 34, No. 5, pp. 737-748.
- Kramer, S.L. and Sivaneswaran, N., (1989), "Stress-path-dependent correction for membrane penetration", Journal of Geotechnical Engineering, Vol. 115, No. 12, pp. 1787-1804.
- Kuerbis, R. H., (1989), "Effect of gradation and fines content on the undrained response of sand", MASc Thesis, University of British Columbia, Vancouver, Canada, 292p.

- Kuerbis, R., Negussey, D., and Vaid, Y. P., (1988), "Effect of gradation and fines content on the undrained response of sand", Hydraulic Fill Structures, ASCE GSP 21, pp. 330-345.
- Lacasse, S. and Vucetic, M., (1981), "State of the art: Laboratory shear testing of soils: Discussion", Symposium on Laboratory Shear Testing of Soils, ASTM Special Technical Publication 740, Eds: Yong, R. N. and Townsend, F. C., pp. 634-637.
- Ladanyi, B., (1981), "Mechanical behaviour of frozen soils", Proc. International Symposium on the Mechanical Behaviour of Structured Media, Ottawa, pp. 205-245.
- Ladd, R., S., (1978), "Preparing test specimens using undercompaction", ASTM Geotechnical Testing Journal, Vol. 1, No. 1, pp. 16-23.
- Ladd, R.S., (1974), "Specimen preparation and liquefaction of sands", Journal of the Geotechnical Engineering division, Vol. 100, No. GT10, pp. 1180-1184.
- Ladd, R.S., (1977), "Specimen preparation and cyclic stability of sands", Journal of the Geotechnical Engineering division, Vol. 103, No. GT6, pp. 535-547.
- Ladd, C. C., and Lambe, T. W., (1963), "The shear strength of undisturbed clay determined from undrained tests", Laboratory Shear testing of Soils, ASTM Special Technical Publication 361, pp. 342-271.
- Ladd, C.C., Footr, R., Ishihara, K., Schlosser, F., and Poulos, H.G. 1977. Stress-deformation and strength characteristics. Ninth International Conference on Soil Mechanics and Foundations Engineering, Vol. 2, pp. 421-482.
- Lade, P.V. and Duncan, J.M., (1973), "Cubical triaxial tests on cohesionless soil", Journal of the Soil Mechanics and Foundations Division, Vol. 99, No. SM10, pp. 793-812.
- Lade, P.V. and Hernandez, S.B., (1977), "Membrane penetration effects in undrained tests", Journal of the Geotechnical Engineering Division, Vol. 103, No. 2, pp. 109-125.

- Lade, P. V. and Wasif, U., (1988), "Effects of height to diameter ratio in triaxial specimens on the behaviour of cross anisotropic sand", Advanced triaxial testing of soil and rock, ASTM Special Technical Publication 977, Eds: Donaghe, R. T., Chaney, R. C., and Silver, M. L., Philadelphia, 706-714
- Lade, P. V. and Yamamuro, J. A., (1997), "Effects og non-plastic fines on static liquefaction of sands", Canadian Geotechnical Journal, Vol. 34, No. 6, pp. 905-917
- Lambe, T. W., (1973), "Predictions in soil engineering.", Geotechnique, Vol. 23, No. 2, pp. 151-202.
- Langer, C., (1939), "Report on undisturbed sampling of fine sand and silt by means of freezing", Paris, France.
- Lin, H. and Selig, E.T., (1987), "An alternative method for determining the membrane penetration correction curve", Geotechnical Testing Journal, Vol. 10, No. 3, pp. 151-155.
- Lo, S.-C.R., Chu, J., and Lee, I.K., (1989), "A technique for reducing membrane penetration and bedding errors", Geotechnical Testing Journal, Vol. 12, No. 4, pp. 311-316.
- Makdisi, F. I., and Seed, H. B., (1978), "Simplified procedure for estimating dam and embankment earthquake-induced deformations.", Journal of the Geotechnical Engineering Division, Vol. 104, No. 7, pp. 849-867.
- Manzari, M. T., and Dafalias, Y. F., 1997, A critical state two-surface plasticity model for sands., Geotechnique, Vol. 47, No. 2, pp. 255-272
- Marcuson, W.F. III and Gilbert, P.A., (1972), "Earthquake liquefaction potential at Patoka Dam, Indiana", US Army Waterways Experiment Station, Miscellaneous paper S-72-42.
- Martin, G.R., Finn, W.D.L., and Seed, H.B., (1978), "Effects of system compliance on liquefaction tests", Journal of the Geotechnical Engineering Division, Vol. 104, No. 4, pp. 463-479.

McClung, R., (1998), Personal Communications.

- Menzies, B. K., (1988), "A computer controlled hydraulic triaxial testing system., "Advanced Triaxial Testing of Soil and Rock", ASTM Special Technical Publication 977, Eds: Donaghe, R. T., Chaney, R. C., and Silver, M. L., Philadelphia, pp. 82-94.
- Miura, S., and Toki, S., (1982), "A sample preparation method and its effect on static and cyclic deformation-strength properties of sand", Soils and Foundations, Vol. 22, No. 1, pp. 61-77.
- Miura, K., Miura, S. and Toki, S., (1986a), "Deformation behavior of anisotropic dense sand under principal stress axes rotation.", Soils and Foundations, Vol. 26, No. 1, pp. 36-52.
- Miura, K., Toki, S. and Miura, S., (1986b), "Deformation prediction for anisotropic sand during the rotation of principal stress axes.", Soils and Foundations, Vol. 26, No. 3, pp. 42-56.
- Mooney, M. A., Finno, R. J., and Viggiani, M. G., (1998), "A unique critical state for sand ?", ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 124, No. 11, pp. 1100-1108.
- Mulilis, P. J., Seed, H. B., Chan, C., Mitchell, J. K., and Arulanandan, K., (1977), "Effect of sample preparation on sand liquefaction", Journal of the Geotechnical Engineering division, Vol. 103, No. GT2, pp. 91-109.
- NRC, (1985), "Liquefaction of soils during earthquakes", Committee on Earthquake engineering Commission on Engineering and Technical Systems, National Research Council, Washington, D.C., Report No. CETS-EE-01.
- Negussey, D., (1984), "An experimental study of the small strain response of sand", PhD Thesis, University of British Columbia, Vancouver, Canada, 224p.
- Negussey, D. and Islam, M. S., (1994), "Uniqueness of steady state and liquefaction potential", Canadian Geotechnical Journal, Vol. 31, No. 2, pp. 132-139.
- Negussey, D., Wijewickreme, W.K.D., and Vaid, Y.P., (1988), "Constant volume friction angle of granular materials", Canadian Geotechnical Journal, Vol. 25, pp. 50-55.
- Newland, P.L. and Alley, B.H., (1957), "Volume changes in drained triaxial tests on granular materials", Geotechnique, Vol. 7, No. 1, pp. 17-34.
- Newland, P.L. and Alley, B.H., (1959), "Volume changes during undrained triaxial tests on saturated dilatant granular materials", Geotechnique, Vol. 9, No. 3, pp. 174-182.
- Newmark, N. M., (1965), "Effects of earthquakes on dams and embankments.", Geotechnique, Vol. 15, No. 2, pp. 139-159.
- Nicholson, P.G., Seed, R.B., and Anwar, H.A., (1993a), "Elimination of membrane compliance in undrained triaxial testing. I. Measurement and evaluation", Canadian Geotechnical Journal, Vol. 30, No.5, pp. 727-738
- Nicholson, P.G., Seed, R.B., and Anwar, H.A., (1993b), "Elimination of membrane compliance in undrained triaxial testing. II. Mitigation by injection compensation", Canadian Geotechnical Journal, Vol. 30 No.5, pp. 739-746.
- Oda, M., (1972), "Initial fabrics and their relations to the mechanical properties of granular material", Soils and foundations, Vol. 12, No. 1, pp. 17-36.
- Oda, M., (1981), "Anisotropic shear strength of cohesionless sands", Journal of Geotechnical Engineering, Vol. 107, No. GT9, pp. 1219-1231.
- Oda, M., and Koishikawa, I., (1977), "Anisotropic fabric of sands", Proc. Ninth International Conference of Soil Mechanics and Foundation Engineering, Tokyo, Vol. 1, pp. 235-238.
- Oda, M., Koishikawa, I., and Higuehi, T., (1978), "Experimental study of anisotropic shear strength of sand by plane strain test", Soils and Foundations, Vol. 18, No. 1, pp. 25-38.
- Oda, M., Nemat-Nasser, S., and Konishi, J., (1985), "Stress-induced anisotropy in granular masses", Soils and Foundations, Vol. 25, No. 3, pp. 85-97.

- Pearce, J. A., (1971), "A new triaxail apparatus", Proc. Roscoe memorial symposium on stressstrain behaviour of soils", Cambridge University, pp. 1179-1184.
- Peacock, W. H. and Seed, H. B. (1968), "Sand liquefaction under cyclic loading simple shear conditions", Journal of the Soil Mechanics and Foundation Engineering Division, ASCE, Vol 94, No SM3, pp 689-708.
- Poulos, S., J., (1981), "The steady state of deformation", Journal of the Geotechnical Engineering division, Vol. 107, No. GT5, pp. 553-562.
- Poulous, S. J., Castro, G., and France, J. W., (1988), "Liquefaction evaluation procedure: Closure", ASCE Journal of Geotechnical Engineering, Vol. 114, No. 2, pp. 251-259.
- Prevost, J. H., (1981), "Nonlinear anisotropic stress-strain-strength behavior of soils", Symposium on Laboratory Shear Testing of Soils, ASTM Special Technical Publication 740, Eds: Yong, R. N. and Townsend, F. C., pp. 431-455.
- Puebla, H., (1998), Personal Communications.
- Puebla, H., (1999), PhD Thesis, University of British Columbia, Vancouver, Canada.
- Raju, V.S. and Sadasivan, S.K., (1974), "Membrane penetration in triaxial tests on sand", Journal of the Geotechnical Engineering Division, Vol. 100, No. 4, pp. 482-489.
- Raju, V.S. and Venkatramana, K., (1980), "Undrained triaxial tests to assess liquefaction potential of sands- effects of membrane penetration", Proc. International Symposium on Soils under Cyclic Transient loading, Rotterdam, Vol 2, pp. 483-494.
- Ramana, K.V. and Raju, V.S., (1982), "Membrane penetration in triaxial tests", Journal of Geotechnical Engineering, Vol. 108, No. 2, pp. 305-310.
- Riemer, M. F., and Seed, R. B., (1997), "Factors affecting apparent position of the steady state line", Journal of Geotechnical Engineering, ASCE, Vol. 123, No. 3, pp. 281-288.

- Roscoe, K. H., (1953), "An apparatus for application of simple shear to soil samples", Proc. Third International Conference of Soil Mechanics and Foundation Engineering, Vol. 1, pp. 186-191.
- Roscoe, K. H., (1970), "Influence of strains in soil mechanics", Tenth Rankine Lecture, Geotechnique, Vol. 20, No. 2, pp. 129-170.
- Roscoe, K. H., Schofield, A. N., and Wroth, C. P., (1958), "On the yielding of soils", Geotechnique, vol. 8, No. 1, pp. 22-53.
- Roscoe, K.H., Schofield, A.N., and Thurairajah, A., (1963), "An evaluation of test data for selecting a yield criterion for soils", Shear Testing of Soils, ASTM Special Technical Publication 361, pp. 111-128.
- Rothenburg, L. and Barthurst, R. J., (1989), "Analytical study of induced anisotropy in idealized granular materials", Geotechnique, Vol. 39, No. 4, pp. 601-614.
- Rowe, P. W. and Barden, L., (1964), "The importance of free ends in triaxial testing", ASCE Journal of the Soil Mechanics and Foundations Engineering division, Vol. 90, No. 1, pp. 1-27.
- Rowe, P. W., (1971), "Theoretical meaning and observed values of deformation parameters for soil", Proc. Roscoe memorial symposium on stress-strain behaviour of soils", Cambridge University, pp. 143-194.
- Saada, A., (1988), "Hollow cylinder torsional devices: Their Advantages and Limitations", In Advanced triaxail testing of soil and rock, ASTM Special Technical Publication 977, Eds: Donaghe, R. T., Chaney, R. C., and Silver, M. L., Philadelphia, pp. 766-795.
- Saada, A. and Townsend, F. C., (1981), "State of the art: Laboratory shear testing of soils", Symposium on Laboratory Shear Testing of Soils, ASTM Special Technical Publication 740, Eds: Yong, R. N. and Townsend, F. C., Philadelphia, 25 June 1980, pp. 7-77.

- Sarma, S. K., (1975), "Seismic stability of earth dams and embankments.", Geotechnique, Vol. 25, No. 4, pp. 743-761.
- Sayao, A. S. F., (1989), "Behaviour of sand under generalised stress paths in the hollow cylinder torsional device", PhD Thesis, University of British Columbia, Vancouver, Canada, 196p.
- Sayao, A. S. F. and Vaid, Y. P., (1989), "Deformations due to principal stress rotation", Proc. Twelfth International Conference on Soil Mechanics and Foundation Engineering, Vol 1, pp. 107-110.
- Seed, H. B., (1979), "Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes", Journal of the Geotechnical Engineering division, ASCE, Vol. 105, No. 2, pp. 201-225.
- Seed, H. B. and Peacock, W. H., (1971), Test Procedure for Measuring Soil Liquefaction Characteristics", Journal of the Soil Mechanics and Foundations Division, Vol. 97, No. SM8, pp. 1099-1119.
- Seed, H. B., Makdisi, F. I., Idriss, I. M., Lee, K. L., (1975), "The Slides in the San Fernando Dams During the Earthquake of February 9, 1971", Journal of the Geotechnical Engineering Division, Vol. 101, No. 7, pp. 651-688
- Seed, H.B., Mori, K., and Chan, C.K., (1977), "Influence of seismic history on liquefaction of sands", Journal of Geotechnical Engineering division, Vol. 103, No. 4, pp. 257-269.
- Seed, H. B., Singh, S., Chan, C. K., and Vilela, T. F., (1982), "Considerations in undisturbed sampling of sands", Journal of the Geotechnical Engineering Division, ASCE, Vol. 108, No. GT2, pp. 265-283.
- Seed, H. B., Seed, R. B., Schlosser, F., Blondeau, F. and Juran, I., (1988), "The landslide at the Port of Nice on October 16, 1979", Report - Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley, 68p.

- Seed, R. B., Anwar, H. A., and Nicholson, P. G., (1989), "Elimination of membrane compliance effect in undrained testing.", Proc. Twelfth International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, pp. 111-114.
- Seed, R. B., and Harder, L.F., (1990), "SPT-based analysis of cyclic pore pressure generation and undrained residual strength", H. B. Seed memorial symposium, BiTech Publishers, Vancouver, Canada., pp. 351 -376.
- Shibuya, S., & Hight, D. W., (1987), "A bounding surface for granular materials", Soils and Foundations, Vol. 27, No. 4, pp. 123-136.
- Shibuya, S. and Hight, D. W., (1989), "Predictions of pore pressure under undrained cyclic principal stress rotation.", Proc. Twelfth International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, pp. 123-126.
- Singh, S., Seed, H. B., and Chan, C. K., (1982), "Undisturbed sampling of saturated sands by freezing", Journal of the Geotechnical Engineering Division, ASCE, Vol. 108, No. GT2, pp. 247-264.
- Sivathayalan, S., (1994), "Static, cyclic and post liquefaction simple shear response of sands", M.A.Sc. Thesis, University of British Columbia, Vancouver, BC, Canada, 139p.
- Sivathayalan, S. and Vaid, Y. P., (1998), "Truly undrained response of granular soils with no membrane penetration effects", Canadian Geotechnical Journal, Vol. 35, No. 5, pp. 730-739.
- Sivathayalan, S., and Vaid, Y. P., (1999), "Ultimate state of reconstituted and intact samples of deltaic sand: Discussion", Canadian Geotechnical Journal, Vol. 36, No. 1, pp. 173-175.
- Sivathayalan, S., Vaid, Y. P., and Uthayakumar, M., (2000), "Characterisation of undrained behaviour of soils in the reclaimed area of Kobe: Discussion.", Soils and Foundations, Vol 40, No. 2, pp. 119-120.

Sladen, J.A., D'Hollander, R.D. and Krahn, J., (1985), "The liquefaction of sands - a collapse surface approach", Canadian Geotechnical Journal, Vol. 22, No. 4, pp. 564-578.

×. ,

- Stark, T. M. and Contreras, I. A., (1998), "Fourth Avenue Landslide During 1964 Alaskan Earthquake", Journal of Geotechnical and Geoenvironmental Engineering, Vol. 124, No. 2, pp. 99-109
- Stokoe, II, K.H., Lee, N.J., and Lee, S.H., (1991), "Characterization of soil in calibration chambers with seismic waves", Calibration chamber testing, Ed: Huang, A. B., First International Symposium on Calibration Chamber Testing, Potsdam, NY, pp. 363-376.
- Sukumaran, B., Leonards, G. A., and Fox, P. J., (1996), "Liquefaction and post liquefaction behaviour of sand", Journal of Geotechnical Engineering, Vol. 121, No. 2, pp. 502-503.
- Symes, M. J. P. R., Gens, A., and Hight, D. W., (1984), "Undrained anisotropy and principal stress rotation in saturated sand", Geotechnique, Vol. 34, No. 1, pp. 11-27.
- Symes, M.J.P.R., Shibuya, S., Hight, D.W. and Gens, A, (1985), "Liquefaction with cyclic principal stress rotation", Proc. Eleventh International Conference on Soil Mechanics and Foundation, San Francisco, Vol. 4, pp. 1919-1922.
- Symes, M. J., Gens, A., and Hight, D. W., (1988), "Drained principal stress rotation in saturated sand", Geotechnique, Vol. 38, No. 1, pp. 59-81.
- Terzaghi, R. and Peck, R. B., (1948), "Soil Mechanics in Engineering Practice", 2nd edition, John Wiley and sons, New York.
- Thomas, J., (1992), "Static, cyclic and post liquefaction behaviour of Fraser River sand",M.A.Sc. Thesis, University of British Columbia, Vancouver, Canada, 117p.
- Tokimatsu, K., and Nakamura, K., (1987), "A simplified correction for membrane compliance in liquefaction tests", Soils and Foundations, Vol. 27, No. 4, pp. 111-122.

- Tsukamoto, Y., Ishihara, K., and Nonaka, T, (1998), "Undrained deformation and strength characteristics of soils from reclaimed deposits in Kobe", Soils and Foundations, Special issue #2 on Geotechnical Aspects of the January 17, 1995 Hyogoken-Nambu Earthquake, Vol. 38, SI. 2, pp. 47-55.
- Tystovitch, N. A., (1952), "Principals of the mechanics of the frozen soils", Academy of Sciences, U. S. S. R.
- Uthayakumar, M., (1996), "Liquefaction of sands under multiaxial loading", PhD Thesis, The University of British Columbia, Vancouver, Canada, 198p.
- Uthayakumar, M. and Vaid, Y. P., (1998), "Static liquefaction of sands under multiaxial loading", Canadian Geotechnical Journal, Vol. 35, No. 2, pp. 273-283.
- Vaid, Y. P. and Finn, W.D.L., (1979), "Static shear and liquefaction potential", Journal of Geotechnical Engineering division, ASCE, Vol. 105, No. GT10, pp. 1233-1246.
- Vaid, Y. P. and Negussey, D., (1984), "A critical assessment of mebrane penetration in the triaxial test", ASTM Geotechnical Testing Journal, Vol. 7, No. 2, pp. 70-76.
- Vaid, Y. P. and Chern, J. C., (1983), "Mechanism of deformation during undrained loading of saturated sand", International Journal of Soil Dynamics and Earthquake Engineering, Vol. 2, No. 3, pp. 171-177.
- Vaid, Y. P. and Chern, J. C., (1985), "Cyclic and monotonic undrained response of sands", Proc. Advances in the art of testing soils under cyclic loading conditions, Detroit, pp. 120-147.
- Vaid, Y. P. and Negussey, D., (1988), "Preparation of reconstituted sand specimens", Advanced Triaxail Testing of Soils and Rock, ASTM Special Technical Publication 977, Eds: Donaghe, R. T., Chaney, R. C., and Silver, M. L., Philadelphia, pp. 405-417.
- Vaid, Y. P. and Pillai, V. S., (1992), "Critical state of sands: Discussion", Geotechnique, Vol. 42, No. 4, pp. 655-663.

- Vaid, Y. P. and Sivathayalan, S., (1996a), "Static and cyclic liquefaction potential of Fraser Delta sand in simple shear and triaxial tests", Canadian Geotechnical Journal, Vol. 33, No. 2, pp. 281-289
- Vaid, Y. P. and Sivathayalan, S., (1996b), "Errors in estimates of void ratio of laboratory sand specimens", Canadian Geotechnical Journal, Vol. 33, No. 6, pp. 1017-1020.
- Vaid, Y. P. and Sivathayalan, S., (1997), "Post liquefaction behaviour of saturated sand under simple shear loading", Proc. Fourteenth International Conference on Soil Mechanics and Foundation Engineering, Hamburg, 6-12 Sept 1997, Vol. 1, pp. 221-224.
- Vaid, Y. P. and Sivathayalan, S., (1998), "Fundamental factors affecting liquefaction susceptibility of sands", Proc. International Symposium on the Physics and Mechanics of Soil Liquefaction, Eds. Lade, P. V., and Yamamuro, J. A., Baltimore, MD, Sept 1998, pp. 105-120.
- Vaid, Y. P. and Thomas, J., (1994), "Post liquefaction behaviour of sand", Proc. Thirteenth International Conference on Soil Mechanics and Foundation Engineering, New Delhi, India, pp.1305 - 1310
- Vaid, Y. P. and Thomas, J., (1995), "Liquefaction and postliquefaction behavior of sand", Journal of Geotechnical Engineering, Vol. 121, No. 2, pp. 163-173.
- Vaid, Y. P., Chung, E. K. F., and Kuerbis, R. H., (1989), "Preshearing and undrained response of sands", Soils and Foundations, Vol. 29, No. 4, pp. 49-61.
- Vaid, Y. P., Chung, E. K. F., and Kuerbis, R. H., (1990a), "Stress path and steady state", Canadian Geotechnical Journal, Vol. 27, No. 1, pp. 1-7.
- Vaid, Y.P., Sayao, A., Hou, E., and Negussey, D., (1990b), "Generalized stress-path-dependent soil behaviour with a new hollow cylinder torsional apparatus", Canadian Geotechnical Journal, Vol. 27, No. 5, pp. 601-616.

- Vaid, Y. P., Uthayakumar, M., Sivathayalan, S., Robertson, P. K., and Hoffman, B, (1995),
 "Laboratory testing of Syncrude sand", Proc.48th Canadian Geotechnical Conference,
 Vancouver, B.C., Vol. 1, pp. 223-232.
- Vaid, Y. P., Sivathayalan, S., Eliadorani, A., and Uthayakumar, M., (1996), "Canadian liquefaction experiment - sand characterizing for static and dynamic liquefaction", Report: Canlex Laboratory testing at U.B.C., Report No. CANLEX 04/96.
- Vaid, Y. P., Sivathayalan, S., and Stedman, D., (1999), "Influence of specimen reconstitution method on the undrained response of sand", ASTM Geotechnical Testing Journal, Vol. 22, No. 3, pp. .187-195.
- Vaid, Y. P., Eliadorani, A., Sivathayalan, S., and Uthayakumar, M., (2000a), "Laboratory characterization of stress strain behaviour of soils by stress and/or strain path loading", submitted for review and possible publication in ASTM Geotechnical Testing Journal.
- Vaid, Y. P., Stedman, D., and Sivathayalan, S., (2000b), " K_{σ} and K_{α} factors in cyclic liquefaction,", submitted for review and possible publication in Canadian Geotechnical Journal.
- Voitkovsky, K. F., (1967), "The relaxation of stresses in ice", Proc. International Conference on Low Temperature Science: Physics of Snow and Ice, Ed. Oura, H., Sapporo, Japan, Aug. 14-16, 1966. Vol. 1, pp.329 - 337.
- Whitman, R. V., (1985), "On liquefaction", Proc. Eleventh International Conference on Soil Mechanics and Foundation Engineering, Vol. 4, pp. 1923-1926.
- Whitman, R. V., (1987), "Are the Soil Depositions in Mexico City Unique?", Proc. The Mexico Earthquake 1985: Factors Involved and Lessons Learned. Eds: Cassaro, M. And Martinez-Romero, E., pp.163 - 177
- Wijewickreme, D. and Vaid, Y. P., (1991), "Stress nonuniformities in hollow cylinder torsional specimens", ASTM Geotechnical Testing Journal, Vol. 14, No. 4, pp. 349-362.

- Wijewickreme, W. K. D., (1990), "Behaviour of sand under simultaneous increase in stress ratio and principal stress rotation", PhD Thesis, University of British Columbia, Vancouver, Canada, 225p.
- Wijewickreme, W. K. D., Uthayakumar, M. and Vaid, Y.P., (1994), "Automatic stress path control system for hollow cylinder torsional testing of soils", Transportation Research Record #1432, National Research Council, Washington, DC.
- Wong, R.K.S., and Arthur, J.R.F., 1985, Induced and inherent anisotropy in sand., Geotechnique, Vol. 35, No. 4, pp. 471-481.
- Wong, R.T., Seed, H.B. and Chan, C.K., (1975), "Cyclic loading liquefaction of gravelly soils", Journal of the Geotechnical Engineering Division, Vol. 101, No. 6, pp. 571-583.
- Wride, C. and Robertson, P. K., (1997), "CANLEX: Data Review Reports" (3 Volumes)-Mildred lake and J-Pit sites & Kidd and Massey sites, CANLEX technical reports, Bi Tech Publishers, Richmond, BC, Canada.
- Wu, H.-C., and Chang, G.S., (1982), "Stress analysis of dummy rod method for sand specimens",Journal of Geotechnical Engineering, 108(9): 1192-1197
- Yamada, Y. and Ishihara, K., (1979), "Anisotropic deformation characteristics of sand under three dimensional stress conditions", Soils and Foundations, Vol. 19, No. 2, pp. 79-94.
- Yamamuro, J. A., Covert, K. M., and Lade, P. V., (1998), "Static and cyclic liquefaction of silty sands", Proc. International Symposium on the Physics and Mechanics of Soil Liquefaction, Eds. Lade, P. V., and Yamamuro, J. A., Baltimore, MD, Sept 1998, pp. 55-66
- Yasuda, S., Yoshia, N., Masuda, T., Nagasse, H., Mine, K., and Kiku, H., (1995), "Stress-strain relationship of liquefied sands", First International Conference on Earthquake Geotechnical Engineering, Eds: K. Ishihara, pp. 811-816.

- Yoshimi, Y., Hatanaka, M., and Oh-Oka, H., (1977), "A simple method for undisturbed sampling by freezing", International Conference on Soil Mechanics and Foundation Engineering, Vol. 9, No. 1, pp. 23-28.
- Yoshimi, Y., Hatanaka, M., and Oh-oka, H., (1978), "Undisturbed sampling of saturated sands by freezing.", Soils and Foundations, Vol. 18, No. 3, pp. 59-73.
- Yoshimine, M., Ishihara, K. and Vargas, W., (1998), "Effects of Principal Stress Direction and Intermediate Principal Stress on Undrained Shear Behavior of Sand", Soils and Foundations, Vol. 38, No. 3, pp. 179-188.

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