Post Liquefaction Behaviour of Sands under Simple Shear and Triaxial Loading Modes

Submitted by

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This thesis is dedicated to:

My beloved parents,
Azizollah and Mahnaz

My dear brother and sisters,
Omid, Armaghan and Alhan

and

All Baha'i students in Iran
who are deprived from higher education.

Abstract

An experimental study aimed at investigating the post liquefaction deformation characteristics of an initially strain softening sand is presented. The shear modulus of the liquefied sand is initially equal to zero, and it continuously increases with strain. Regardless of whether the sand is contractive or dilative in the pre-liquefaction response, its post liquefaction response is dilative if a state of zero effective stress was realised at the end of cyclic loading. Influence of relative density, consolidation stress, maximum strain prior to the post liquefaction shearing, and stress path on the characteristics of the post liquefaction response are studied using simple shear and triaxial devices. It is shown that relative density and strain level profoundly influence the post liquefaction response, and the post liquefaction response is stress path dependent. The residual strength calculated in current practice based on $N_{1,60}$ values appear conservative compared to laboratory post liquefaction data.

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Notation

B	Coefficient of saturation
b	Intermediate principal stress parameter
CSR	Critical stress ratio/Cyclic stress ratio
C _U	Uniformity coefficient
D ₅₀	Average particle size
DP	Dry pluviation method
D _r	Relative density
D _{rc}	Relative density after consolidation
e	Void ratio
I _B	Brittleness index
K _o	Consolidation stress ratio at rest= $\sigma'_{vc}/\sigma'_{hc}$
N _{1,60}	Corrected normalised standard penetration resistance
PT	Phase transformation
QSS	Quasi steady state
SS	Steady state
S _{us}	Undrained shear strength
TC	Triaxial compression mode
TE	Triaxial extension mode
WP	Water pluviation method
α	Inclination angle of major principal stress
Δu	Excess pore water pressure
ϵ_a	Axial strain

$\epsilon_{\max}$	Maximum pre-cyclic axial strain
ϕ_{CSR}	Friction angle at critical stress ratio
ϕ_{PT}	Friction angle at phase transformation
ϕ_{Failure}	Friction angle at maximum obliquity
γ	Shear strain
$\gamma_{\max}$	Maximum pre-liquefaction shear strain
σ_1	Major total principal stress
σ_2	Intermediate total principal stress
σ_3	Minor total principal stress
σ'_1	Major effective principal stress
σ'_3	Minor effective principal stress
σ'_{3c}	Effective consolidation stress (triaxial tests)
σ'_c	Effective confining stress
σ_d	Deviator stress
σ'_H	Horizontal effective stress
σ'_V	Vertical effective stress
σ'_{vc}	Effective vertical confining stress after consolidation (simple shear tests)
τ	Shear stress

CHAPTER 1

INTRODUCTION

1.1 General

Liquefaction of saturated sands has been the subject of the extensive research over the past 40 years. Under static loading, the term liquefaction is associated with a contractive strain softening type of response while under cyclic loading it is usually defined in terms of the strain criterion. Sand, under cyclic loading is deemed to have liquefied if it develops a specific level of strain (2.5% axial strain or 3.75% shear strain) regardless of the manner in which strain develops (NRC, 1985). Following liquefaction induced by cyclic loading or monotonic load/unload cycle, the sand may be at the state of zero effective stress. Regardless of whether the sand is contractive or dilative in pre-liquefaction response, its response following this state of zero effective stress is dilative.

Large deformations of earth structures and sand deposits are often caused by monotonic post liquefaction loading due to the driving shear stresses following liquefaction induced by an earthquake. Large cyclic deformations may occur if the sand undergoes excursions through the state of zero effective stress during undrained loading. Attempts to predict liquefaction induced displacements require knowledge of the nature of the stress-strain and strength characteristics of the liquefied soil.

The current analytical methods for estimating earthquake induced displacements are generally based on the models which imply modulus degradation with strain (Finn et al. 1986), and the notion that steady-state strength during the post liquefaction loading remains unaltered at the pre-liquefaction static value (Byrne et al. 1992). On the other hand, recent laboratory studies (Thomas, 1992; Vaid and Sivathayalan, 1997) indicate that the post liquefaction shear modulus increases with strain, and the post liquefaction shear strength does not approach residual shear strength equal to the pre-liquefaction steady-state strength. Little systematic research has been carried out on the post liquefaction behaviour of sands. Therefore, further studies are required to comprehensively characterize the behaviour of the liquefied soil, and to fully recognize the possible spectrum of the post liquefaction responses.

1.2 Objective and Scope of the Study

The objective of this study is to investigate the mechanics of the post liquefaction response of sand. The post liquefaction behaviour of two different sands, one rounded (Silica Sand) and the other sub-angular (Fraser River Sand), have been studied over a range of relative densities and confining stress levels to assess the influence of the dilation potential on the post liquefaction response of sand. Most of the tests were carried

out under the simple shear loading mode, which simulates the stress conditions in soil elements in-situ due to vertically propagating shear waves. A limited number of tests were carried out under the triaxial loading mode to evaluate the stress path effects on the post liquefaction response. In addition to the above variables, the influence of the maximum strain level prior to liquefaction has also been studied.

To have a better understanding of sand behaviour and its liquefaction characteristics, the pre-liquefaction monotonic response of sand is studied separately. Monotonic simple shear and triaxial tests were carried out to determine the contractive characteristics of the sands at the selected initial densities and stress states. This enables the identification of characteristics common to both virgin and post liquefaction loading.

Three different sample preparation techniques (water and dry pluviation, moist tamping) were used in this experimental work to assess the effect of soil fabric. The post liquefaction response was studied using pluviated fabric. The moist tamping method is useful to achieve very loose samples in order to study the characteristics of the steady-state type of response.

1.3 Outline of Thesis

Chapter 2 presents a review of the literature related to the monotonic and post liquefaction behaviour of sands. Advantages and limitations of the simple shear and triaxial devices, and different sample preparation techniques are discussed in chapter 3. In chapter 4, the characteristics of the monotonic pre-liquefaction behaviour of Silica and Fraser River Sand are presented. The specimens were prepared using various reconstitution methods and loaded along different stress paths. The steady-state concept is also discussed in chapter 4. The main objective of this thesis, which is the investigation

of the post liquefaction response of sand, is presented in chapter 5. The effects of the relative density, confining stress, maximum pre-liquefaction strain level and stress path on the post liquefaction response are discussed. The laboratory data from this study are compared to the empirical design approach used in current practice. A summary and conclusions of the experimentation and analysis are presented in chapter 6.

CHAPTER 2

LITERATURE REVIEW

The flow slides and damages that occurred during the 1964 Alaska and Niigata earthquake have triggered the initial interest in the phenomenon of liquefaction. Main focus has been on the monotonic and cyclic behaviour of virgin sand. Very little attention has been paid to the behaviour of liquefied sand until recently. Relative density, consolidation pressure, stress path, fabric, static shear and cyclic stress ratio are among the most important factors that influence the monotonic and cyclic behaviour of sand.

Post liquefaction behaviour of sand is a new area of research in comparison to monotonic and cyclic behaviour. Large settlement of embankments after earthquake loading (caused by dissipation of pore water pressure under static loading), and residual strength of sand after earthquake loading are directly related to the post liquefaction behaviour of sand.

2.1 Pre-liquefaction Monotonic Behaviour

Under pre-liquefaction undrained monotonic loading, sands exhibit three distinct types of response as shown in Figure 2.1 (Castro, 1969; Castro et al., 1982; Chern, 1985; Kuerbis, 1989). The change in stress-strain characteristics is generally associated with the change in relative density or consolidation pressure, and mode of loading. Type 1 and 2 are characteristic responses exhibited by loose sands, and sands at higher effective confining stress. Type 1 is generally called flow failure type of response. Type 3 is a characteristic dilative response exhibited by dense sands, and sands at lower effective confining stress.

Type 1 and 2 are strain softening (or contractive) responses, which are associated with loss of shear strength after the occurrence of a peak. Type 1 response has been called liquefaction by Castro (1969) and Seed (1979) and true liquefaction by Chern (1985). The most important characteristic of this type of response is the continued deformation at constant void ratio and constant confining and shear stresses. This constant shear resistance is called steady state strength, and because of its implications this type of response is often called flow deformation (Castro, 1975; Vaid and Chern, 1983). Type 2 response is strain softening with limited strain and has been called limited liquefaction (Castro, 1969). In this type of response, strain hardening follows strain softening after a limited amount of strain at minimum shear resistance. Type 3 of response is a strain hardening response. There is no loss of shear strength in type 3, and therefore it is inherently safe.

2.1.1 Effect of Loading Direction and Stress System on Stress-Strain Curve

Monotonic response of sand significantly depends on loading direction and stress system. Two variables; α which is the inclination of major principal stress to the vertical specimen axis, and $b = (\sigma_2 - \sigma_3) / (\sigma_1 - \sigma_3)$ which characterizes the influence of intermediate principal stress, are used to indicate the influence of stress path on sand behaviour. Miura and Toki (1982), Kuerbis and Vaid (1989), Thomas (1992) and Yoshimine et al. (1998) have reported that conventional triaxial compression tests could be completely dilative even for loose sands under high consolidation pressure. However, triaxial extension response is typically strain softening over a range of consolidation pressures and relative densities. At a given initial state, a gradual transformation of the response of Fraser River Sand occurs from dilative to strain softening while α increases from 0° to 90° at constant $b=0$ (Uthayakumar and Vaid, 1998). The strain softening response aggravates with an increase in b value at constant α value. Vaid and Sivathayalan (1996) have carried out a complete series of tests and concluded that for a given consolidation stress and relative density the potential for liquefaction depends on the stress path during loading. Triaxial extension has the least shear strength and triaxial compression has the most shear strength. The strength of sand in simple shear mode falls between them and is closer to the strength in triaxial extension than that in triaxial compression mode.

2.1.2 Critical Stress Ratio

The peak shear stress indicates the initiation of strain softening behaviour. On the effective stress path curve, such as shown in Figure 2.2, the effective stress ratio

corresponding to the peak shear stress has been called critical stress ratio (CSR) by Vaid and Chern (1983). It should be noted that the concept of critical stress ratio is only applicable to the strain softening sands. Various researchers have shown that the CSR line is unique for sand in triaxial compression, but dependent on void ratio (Chung, 1985; Chern, 1985; Thomas, 1992) and sample preparation technique (Kuerbis, 1989) in extension loading modes. The position of CSR line could be affected by confining pressure and relative density as well. Castro (1982) and Sladen et al. (1985) have shown that the CSR line position for moist tamped specimens in triaxial compression tests is depends on the confining stress. Yang (2002) has reported that in loose sand the CSR line is not unique in triaxial compression tests and depends on both initial void ratio and consolidation pressure. Vaid and Sivathayalan (1996) have evaluated the friction angle mobilized at the peak shear stress and reported that it increases with a decrease in void ratio. This friction angle is the largest for triaxial compression mode and is smallest for triaxial extension mode. CSR friction angle in simple shear mode falls between CSR friction angle in compression and extension modes.

2.1.3 Phase Transformation State

The point of the termination of strain softening which is the end of increase in pore pressure and the start of increase in shear resistance has been called phase transformation state by Ishihara et al. (1975). The phase transformation line as shown in Figure 2.2 is a unique line that does not depends on void ratio, consolidation stress nor stress path. This has been illustrated by Chern (1985) and Thomas (1992) using triaxial tests. Vaid and Sivathayalan (1996) using simple shear tests, Uthayakumar and Vaid (1998) and Sivathayalan and Vaid (2002) using hollow cylinder torsional tests have

suggested that the friction angles mobilized at phase transformation state under triaxial and other loading modes are not too different and can be considered equal.

2.1.4 Ultimate Failure Line

On further straining beyond the PT state, the effective stress path follows the undrained ultimate failure line or the line of maximum obliquity (Ishihara et. al., 1975). The position of ultimate failure line is generally considered unique for a given sand and does not change with change in void ratio, consolidation pressure and mode of loading, although there is a discrepancy in literature about the uniqueness of failure line. Castro et al. (1982) have reported that the friction angle at failure changes with change in stress and strain level, first increasing with increase in strain and then decreases to phase transformation angle at very large strain. Miura and Toki (1982) have shown that the failure line angle varies with relative density, loading mode and sample preparation technique. On the other hand, Vaid et al. (1990), Kuerbis (1989) and Thomas (1992) have illustrated that loading mode, sample preparation technique, level of stress and strain and relative density do not affect the angle of failure line, and thus this angle is a unique property of a given sand.

2.1.5 Steady State Concept

Predominant interest has generally focused on behaviour of sand in loosest state which results in true liquefaction or flow liquefaction. It has been shown by Castro (1969) that if sand undergoes true liquefaction, the effective confining stress and shear resistance during steady state deformation are uniquely related to void ratio regardless of the initial consolidation stress condition. This unique line within the three dimensional

space of void ratio, shear stress and confining stress has been called steady state line by Castro et al. (1982) and Chern (1985).

Castro (1982) has suggested that steady state is only achieved after all soil dilation is complete. This may occur at small strain if the sand is very contractive or at very large strain if the soil is substantially dilative past phase transformation state. Chern (1985) considered stress condition at phase transformation state for limited liquefaction and found a unique relationship between void ratio and effective confining stress in triaxial compression loading tests. Vaid and Sivathayalan (1996) have shown that the normalized steady state or phase transformation strength for a given void ratio and confining stress is smaller in triaxial extension than simple shear mode. They have also mentioned that these differences become larger with an increase in confining stress level, and for a given confining stress level the difference further increases with an increase in density.

2.1.6 Effect of Sample Preparation Technique

Sample preparation method is one of the most important factors that affect the initial fabric of granular material. Oda (1972) has shown that the differences in initial fabric of granular soils usually influence the mechanical properties of the material such as mobilized strength. Ladd (1974, 1977) examined different types of sample preparation methods and concluded that moisture content at which the samples were prepared has a great influence on the cyclic strength of sand. The moist tamped sample has the largest cyclic resistance and dry vibrated specimen has the smallest cyclic resistance.

There is a big discrepancy in literature about the strength of samples prepared with different techniques. Ibrahim and Kagawa (1991) have reported that dry pluviation

technique tends to produce specimens with more random particles orientation and larger deviation of local void ratios than wet tamping method. They have shown that the specimen prepared by dry pluviation method has the least resistance to liquefaction while those prepared by wet tamping technique have the largest resistance. DeGregorio (1990), also, has shown that the moist tamped specimens have higher peak and steady state strength than the dry pluviated samples. However, Vaid et al. (1999) concluded, based on several tests, that the moist tamping technique produces much more non-uniform samples compared to the water-pluviated counterpart. They also showed that the moist tamped specimens in a certain void ratio can be contractive while the water pluviated specimens at the same void ratio and confining stress are dilative. Frost and Park (2003) supported this idea by investigating the uniformity of moist tamped specimens using X-ray and optical images methods. They have shown that the standard deviation of local relative density in the air-pluviated specimens was about half of that in the moist-tamped specimens.

2.2 Post Liquefaction Behaviour

Displacements induced by post liquefaction deformation of soil following earthquake shaking can be large enough to produce severe damage to earth structures including dams and embankments. Settlement of sand after cessation of earthquake occurs due to dissipation of excess pore pressure that developed during cyclic loading. Most of the studies that have been done in the area of post liquefaction and the settlement of sand after earthquake cessation are in situ based (Hamada, 1987; Seed and Harder, 1990). Little systematic experimental research has been carried out on post liquefaction response of sand. Such information is needed for predicting and estimating the

settlements induced by earthquake shaking in earth structures, and to validate the empirical observations.

2.2.1 General Characteristics of Post Liquefaction Response

Pre and post liquefaction stress-strain curve of a tailings sand presented by Kuerbis (1989) is shown in Figure 2.3. Prior to liquefaction the strains are small, but upon liquefaction large strain occurs. Unloading of shear stress following limited liquefaction brought about a transient state of zero shear stress. During further loading, sand deforms at zero shear stiffness which then increases gradually with level of strain and then remains constant. The post liquefaction behaviour is dilative in its entire path after passing the zero shear stress state. This type of behaviour has been reported by several researchers (Thomas, 1992; Vaid and Sivathayalan, 1997; Shamoto et al., 1997 and Koester, 1999).

Post liquefaction response of sand is divided into three distinct regions as shown in Figure 2.4 (Thomas, 1992). Region 1 starts from the first instant that liquefied soil pass through the zero effective stress and has the essentially zero shear stiffness. The stiffness gradually increases with strain at a much smaller rate. Region 2 is a parabolic region in which the stiffness increases relatively rapidly with strain. Region 3 is the linear section in stress-strain curve with constant shear stiffness.

2.2.2 Factors Affecting Post Liquefaction Behaviour

The post liquefaction response of sand depends on three major variables in each loading mode. Thomas (1992) has conducted a complete series of tests to investigate the influence of relative density, consolidation pressure, maximum pre-cyclic strain level and

loading mode on the post liquefaction response of sand. The rate of increasing in stiffness increases with relative density, however relative density does not significantly affects the deformation during first phase of deformation. The axial strain, at which the curves become practically linear, decreases with an increase in relative density as shown in Figure 2.5 (Vaid and Thomas, 1995). The same dependency of post liquefaction strength to relative density has been shown by Vaid and Sivathayalan (1997) in simple shear test and Shamoto et al. (1997) in torsional undrained test.

Another factor that affects the post liquefaction response is consolidation pressure prior to pre-liquefaction loading. By increasing the consolidation pressure at the identical relative density and pre-cyclic strain level, the stiffness of linear part increases and the post liquefaction strength increases as a consequence (Thomas, 1992; Vaid and Sivathayalan, 1997). The effect of consolidation stress is not as significant as the effect of relative density, although it affects deformations during both region 1 and region 2.

One of the most important factors affecting the post liquefaction strength appears to be the maximum pre-liquefaction strain level. Thomas (1992) has discussed the characteristics of region 1 based on the maximum pre-liquefaction strain level in triaxial compression test. Figure 2.6 shows that by increasing the maximum pre-liquefaction strain level, the strain within region 1 increases which implies the weaker behaviour. Vaid and Sivathayalan (1997), also, have shown the same effect of the maximum pre-liquefaction strain level on the post liquefaction behaviour using simple shear test. The Maximum pre-liquefaction strain influences the post liquefaction strength as well. Post liquefaction strength increases with a decrease in the pre-liquefaction strain.

The post liquefaction behaviour is also dependent on mode of loading and stress path just like the pre-liquefaction response. Vaid and Sivathayalan (1997) have presented the comparison of post liquefaction response in simple shear and triaxial compression tests. Simple shear results in weaker post liquefaction response compared to triaxial compression tests. Thomas (1992) has showed that post liquefaction response in triaxial extension mode is weaker than that in triaxial compression mode. Both Thomas (1992) and Vaid and Sivathayalan (1997) have discussed that the post liquefaction response of sand does not depend on the type of pre-liquefaction loading. Initially identical samples, whether liquefied by load/unload cycle or cyclic loading exhibited approximately the same post liquefaction behaviour.

The residual effective stress state at the end of cyclic loading affects the post liquefaction behaviour of sand. Thomas (1992) has showed that the modulus of post-cyclic loading decrease initially, but starts increasing once dilation commences if the residual effective stress is non zero. Increasing residual shear stress at the end of cyclic loading makes the post liquefaction response stronger.

The post-cyclic response appears to be profoundly influenced by the direction of post-cyclic strain relative to the sense of residual strain following liquefaction. The post liquefaction stress-strain is much stiffer with no leg corresponding to virtual zero stiffness when strain occurs in the same direction as the residual strain than when there is a strain reversal (Vaid and Sivathayalan, 1997).

There is only limited information about the effect of sample preparation or fines content on the post liquefaction behaviour of sand. Koester (1999) has carried out some tests on different sands with different fines content and found out that increasing the fines

content causes sand to have weaker behaviour in cyclic test, and sand specimens do not dilate upon subjection to the large shear strains following liquefaction.

2.2.3 Numerical Modelling

A simple model for predicting liquefaction induced settlements has been presented by Byrne et al (1992). This method is based on single-degree-of-freedom system that incorporates the post liquefaction stress-strain of sand. The key parameter is residual strength which comes from correlation with $(N_1)_{60}$ values. In this method, it has been suggested that the steady state strength of a contractive sand is not altered during post-cyclic loading.

Another model has been proposed by Shamoto et al. (1997) based on some experimental post liquefaction data. They considered that large post-liquefaction shear strain is governed by two types of volumetric strains due to dilatancy, irreversible dilatancy component and reversible dilatancy component. Shear strain is composed of a shear strain component independent of effective stress and a shear strain depending on change in effective stress. They have suggested that the dependent shear strain to effective strain is determined only by the preceding maximum pre-liquefaction strain level for sand of given density.

2.2.4 Post Liquefaction Settlement and Volumetric Strain

When saturated sand deposits are subjected to earthquake loading, pore pressure is generated, and this leads to liquefaction or loss of strength in the sand deposits. Following the cessation of the dynamic event, the pore pressure starts to dissipate and cause volumetric change that leads to settlement of sand.

There are several factors affect volumetric strain of sand. Laboratory tests by Tatsuoka et al. (1984), Negase and Ishihara (1988) and Thomas (1992) have shown the influence of relative density, consolidation pressure and maximum pre-liquefaction strain level on volumetric strain of sand. Tatsuoka et al. (1984) and Negase and Ishihara (1988) showed that volumetric strain under undrained simple shear condition after liquefaction can be significantly influenced by the maximum shear strain developed in the soil and the soil density but is insensitive to level of effective confining pressure prior to cyclic loading. Tokimatsu and Seed (1987) developed a simplified method for predicting post liquefaction settlement based on available test data and concluded that the primary factors controlling earthquake induced settlement are the cyclic shear strain, relative density of the sand and the magnitude of the earthquake that controls the amplitude of cyclic stress. Ishihara and Yoshimine (1992) have proposed a series of curves that relate the volumetric strain to the density of sand and shear strain amplitude, but independent of cyclic stress level and confining pressure. However, Thomas (1992), based on experimental results, has shown that the volumetric strain depends on confining pressure in addition to relative density and maximum pre-liquefaction shear strain level. Also, Pillai and Salgado (1994) studied the post-liquefaction stability and deformation analysis of Duncan Dam using Seed's approach and experimental analysis on undisturbed samples. They have concluded that the Seed's residual strengths are low and not affected by large confining stresses present in the foundation soils at Duncan Dam, and consequently predicts less than adequate performance of dam while the laboratory testing method considering effective confining stresses predicts more reliable post-earthquake performance of the dam.

Figure 2.1 Stress-strain response of sandy soils (Kuerbis, 1989)

Figure 2.2 Stress path of sandy soils (Kuerbis, 1989)

Figure 2.3 Pre and Post liquefaction stress-strain response of sand (Kuerbis, 1989)

Figure 2.4 Characterization of post cyclic curve (Thomas, 1992)

Figure 2.5 Effect of relative density on post cyclic undrained monotonic response (Thomas, 1992)

Figure 2.6 Maximum strain due to cyclic or static load/unload cycle (Thomas, 1992)

CHAPTER 3

EXPERIMENTAL WORK

3.1 Material Tested

Tests were performed on two commercial sands; a Silica Sand that conforms to ASTM designation C-109, and Fraser River Sand dredged from Fraser River near Abbotsford in British Columbia.

Silica Sand is a uniform white coloured quartz sand ($D_{50} = 0.425$ mm, $C_u = 1.8$) with rounded particles. Its maximum and minimum void ratios determined by the ASTM test method D4253 and D4254 are 0.723 and 0.478 respectively. The specific gravity is determined as 2.66 based on ASTM test method D854.

Fraser River Sand is a medium uniform grey coloured sand with subangular particles ($D_{50} = 0.26$ mm, $C_u = 1.9$). The average mineral composition of the sand is

generally 40% quartz, quartzite and chert, 11% feldspar, 45% unstable rock fragments and 4% miscellaneous detritus (Garrison et. al., 1969). Its maximum and minimum void ratios determined by the ASTM test method D4253 and D4254 are 0.806 and 0.509 respectively. The specific gravity is determined as 2.73 based on ASTM test method D854. The fine particles percentage, passing sieve No. 200 (.075 mm), was reduced to less than 0.5% in order to eliminate the influence of silt material. The grain size distribution of Silica and Fraser River Sands are shown in Figure 3.1.

3.2 Testing Apparatus

Most of the tests were conducted using an NGI type simple shear apparatus and a limited number of tests were carried out using a triaxial device. Simple shear apparatus simulates the field conditions during an earthquake better than triaxial equipment. The stress condition in the simple shear samples is comparable to what occurs to an in-situ soil element when a vertically propagation of shear wave passes by it. However, the non-uniformity of stresses in simple shear specimen is relatively higher than that in triaxial specimen.

3.2.1 Simple Shear Apparatus

A schematic diagram of the simple shear device is shown in Figure 3.2, and a photograph in Figure 3.3. Simple shear specimens were approximately 20 mm high and 71 mm in diameter. Shear stress, normal stress, horizontal displacement and vertical displacement were measured using the load cells and electronic transducers connected to a data acquisition system interfaced with a PC.

The testing system consists of a simple shear cell and a loading system. Steel wire reinforced membrane was used to form the samples. Using the steel wire reinforced membrane has two advantages. First, it provides the condition for conducting a real constant volume test even at large strain with the help of vertical piston lock. Second, it provides a K_0 condition which is close to field condition and simulates the in-situ condition much better than a typical isotropic consolidation system used in conventional triaxial tests.

The vertical loading system consists of an air cylinder, which is controlled manually. Horizontal loading system consists of an air actuator and electro-pneumatic transducer. Air actuator has two separate chambers, one of them is connected to a constant air pressure and the other one is connected to an electro-pneumatic transducer. Cyclic shear stresses are applied by adjusting the air pressure using the electro-pneumatic transducer.

The original displacement measurement system consisted of non-contact transducers, which were not very linear. The actual relationship between voltage and displacement was sinusoidal, but if the appropriate range was chosen the response was quite close to linear. Each transducer was mounted on a wheel moving on a rod or plate and both of them were aligned on a system together, and this alignment mechanism caused some cross-talk. Also, because of cohesion between wheels and rod or plate caused by dirt in the air the offset of transducers were changing a lot during tests. As a result, the dependent non-linear transducers were replaced by two separate high resolution linear LVDTs, which were mounted separately using custom-made clamps.

3.2.2 Triaxial Device

A schematic diagram of the triaxial equipment is shown in Figure 3.4, and a photograph in Figure 3.5. Triaxial specimens were approximately 127 mm high and 63 mm in diameter. Axial load, cell pressure, pore pressure and axial displacement were measured using load cells and electronic transducers connected to a data acquisition system interfaced with a microcomputer. Volume change was indirectly measured using a differential pressure transducer.

The testing system consists of a triaxial cell and a loading system. The loading system is capable of strain or stress controlled loading both under monotonic and cyclic modes. Strain controlled monotonic system provides a better record of stress-strain and pore pressure response than stress controlled system especially in strain softening material as it arrests the run away strains.

3.3 Testing Procedure

3.3.1 Consolidation

In simple shear apparatus, samples were consolidated to the desired effective vertical stress under K_0 condition by using the wire reinforced membrane. Following preparation of samples outside of the cell, they were carried and installed in the cell, and vertical consolidation pressure was manually applied to them in stage. The horizontal shear stress was maintained at zero during consolidation.

In triaxial device, samples were isotropically consolidated by step wise increase of cell pressure using a constant back pressure. Prior to initiation of isotropic consolidation, both chambers of the piston were pressurized to an approximately equal

base pressure. To compensate the uplift on the loading ram during setting up of back pressure, the pressure in the lower piston was reduced appropriately.

3.3.2 Cyclic Loading

In simple shear apparatus, cyclic load was applied by means of an electro-pneumatic transducer. A sinusoidal cyclic pulse was applied to the transducers. Sinusoidal cyclic pulse was calibrated prior to setting up of the test specimens. The rate of frequency was adjustable but it was normally 0.10 Hz in the tests. Cyclic tests were carried out under constant volume condition by using wire reinforced membrane and locking the vertical piston movement. No cyclic test was carried out in triaxial device in this study.

3.3.3 Monotonic Loading

Both pre and post liquefaction monotonic tests in simple shear device were conducted under stress controlled loading system by increasing or decreasing the input voltage of the electro-pneumatic transducer. The post liquefaction tests were conducted following liquefaction induced by cyclic loading. Like the cyclic tests, constant volume conditions were maintained during the monotonic and post liquefaction tests.

In triaxial device, both pre and post liquefaction monotonic tests were conducted using the strain controlled loading mode. Following consolidation, the strain drive was lowered and connected to the upper portion of the double acting piston through a load connection ring. Pulses were applied to the motor through the computer causing the motor to move in compression or extension modes.

3.4 Resolution of Measurement

In simple shear apparatus, horizontal load was corrected for: 1) stiffness of membrane 2) friction force in the horizontal loading system, and 3) LVDT spring force on sample. A signal conditioning box was used to reduce the noise level in the system. The horizontal and vertical stresses had a resolution of ± 0.1 kPa. The resolutions of the horizontal and vertical strains were accurate to $\pm 0.01\%$.

In triaxial device, the resolution of cell and pore pressure was ± 0.01 kPa and the resolution of vertical load cell was ± 0.01 kg. Axial and volumetric strain had a resolution of $\pm 0.001\%$.

3.5 Sample Preparation and Set up in Simple Shear Apparatus

Three different sample preparation techniques were used for preparing the specimens in simple shear device: 1) dry pluviation, 2) water pluviation, and 3) moist tamping. Different sample preparation would produce specimens with distinctively different fabric (Oda, 1972; Ibrahim and Kagawa, 1991).

3.5.1 Dry Pluviation

In the dry pluviation or air pluviation method, dry sand is pluviated from a controlled height inside the sample cavity. First, the membrane was sealed around the pedestal using an O-ring. The split mould was placed around the membrane and the upper part of membrane was folded around the top part of mould, then suction was applied to make membrane lined in the mould. A brass ring was placed on top of the mould to provide a smooth surface in order to level the surface of specimen. Dry sand was pluviated through a funnel inside the sample cavity. In order to have a uniform sample,

the pouring tip was traversed laterally during deposition. To achieve minimum possible density, the drop height was kept near to zero. The surface of specimen was levelled by applying suction through a plexiglas tube. The plexiglas tube was held by another plexiglas cubic smooth rod that traversed smoothly and laterally on top of the brass ring to achieve a levelled specimen surface. Then, the top cap was placed slowly on the surface of specimen and the top part of membrane was sealed around the cap using an O-ring. The minimum possible density achieved after consolidation to 400 kPa was around 30%, and for 200 and 100 kPa consolidation stresses it was 28% and 26% respectively for Silica Sand. By tamping the table below pedestal, it was possible to reach to higher densities. A 20 kPa vacuum was applied to the specimen to prevent disturbance of sample during delivery of sample to the cell. A schematic diagram of the dry pluviation technique is shown in Figure 3.6 (a).

3.5.2 Water Pluviation

Several researchers (Finn et. al., 1971; Vaid and Negussey, 1984) have described the water pluviation technique for sample preparation. This technique simulates the deposition of alluvial soils and mechanically placed hydraulic fills.

Sample preparation technique and set up before pluviation of sand is quite similar to dry pluviation technique. Following setting up the cavity, another mould with the height of 15 cm was placed on top of the split mould. The cavity and more than half of the upper mould were filled with de-aired water. Boiled and de-aired sand was cooled down to room temperature and then pluviated in the de-aired water inside the cavity through a plexiglas nozzle. The tip of nozzle was always kept submerged in water during deposition. The nozzle tip traversed laterally and smoothly in order to provide a level and

uniform sample. To achieve minimum disturbance during levelling, the extra sand was siphoned off by using a small plexiglas tube connected to a flexible tube. The top cap was placed very slowly and without any pressure in order to prevent disturbance in the top layer of sample. Following sealing the top cap with the membrane using an O-ring, a 20 kPa vacuum was applied to samples to reduce densification of samples carrying to the simple shear cell. The schematic diagram of this deposition technique is shown in Figure 3.6 (b).

3.5.3 Moist Tamping

Moist Tamping method is probably the most widely used specimen preparation technique. This method may be used to produce samples with very loose void ratio. However, it has been heavily criticised because of its lack of uniformity compared to dry and water pluviated sands (Vaid et al., 1999; Frost and Park, 2003).

The technique essentially involves placing moist sand in several layers and tamping them to required overall density. Sand was mixed with 9% of water (Ladd, 1974). It was compacted in four layers while the lower layers were compacted looser than the final desired density so that the final density of each layer was equal to the target density. Equal amount of sand was used in each layer. After sealing the top cap with membrane using an O-ring, a volume equal to 4 to 6 times of samples void volume of de-aired water percolated through the samples from the bottom to the top. This was done to improve saturation and to break the capillary tension between the sand particles. To avoid the samples disturbance while they were carried to the cell a 20 kPa vacuum was applied to the samples. The schematic diagram of the moist tamping technique is shown in Figure 3.6 (c).

This method can provide specimen with a wide range of void ratio, from a dense value, for which considerable amount of tamping energy is necessary, up to the values well above the loosest void ratio determined using the ASTM standard.

3.6 Sample Preparation and Set up in Triaxial Device

Sample preparation in triaxial device is similar to water pluviation technique in simple shear device except the size of the cavity which is bigger than the size of simple shear specimen cavity. Also, it was possible to achieve samples with void ratio looser than maximum void ratio attainable in simple shear specimens. The reason would probably be the larger size of triaxial samples in comparison to simple shear samples and the better confinement provided while moving the specimen to the test frame. The membrane in triaxial sample preparation is not reinforced, so confinement of samples is required. A vacuum of 20 kPa was applied following the installation of top cap.

The triaxial cell was assembled, and then drainage line was connected to the pore water pressure transducer device. B values of 0.98 or better were obtained consistently by increasing the cell pressure in stage up to maximum value of 400 kPa. It should be considered that the differences between cell pressure and pore pressure should always be less than desired consolidation stress. Consolidation was conducted using a constant back pressure equal to differences between cell pressure and desired consolidation stress and opening the drainage line to volume change transducer. After the consolidation stage was completed, the drainage line was closed and the sample was ready for undrained monotonic or cyclic loading.

3.7 Test Program

3.7.1 Static Tests

Static tests in simple shear were carried out on sand with a range of confining pressure-100, 200 and 400 kPa- and a range of relative density- 30%, 50% and 70%. Static tests were conducted using three different sample preparation techniques described earlier.

In triaxial tests, limited amount of tests were conducted on loosest (relative density of 15% to 18%) samples and samples with relative density of 30% and 50%.

3.7.2 Cyclic Tests

Cyclic tests with constant $CSR=0.1$ conducted on sand with different consolidation pressures and different densities (similar to those used in static tests). The criteria for stopping the cyclic tests were based on different amount of strain level at the end of cyclic tests. Cyclic tests were conducted on dry pluviated and water pluviated samples.

3.7.3 Post Liquefaction Tests

Post liquefaction tests were carried out on specimen liquefied either by cyclic loading (simple shear tests) or by monotonic load/unload cycle (triaxial tests). Influences of various parameters (relative density, consolidation pressure and maximum pre-liquefaction strain level) were assessed by considering systematic series of tests.

Figure 3.1 Grain size distribution curve of Silica and Fraser River Sand

Figure 3.2 Schematic layout of simple shear apparatus

Figure 3.3 Simple shear apparatus at Carleton University

Figure 3.4 Schematic layout of triaxial device

Figure 3.5 Triaxial device at Carleton University

a) Dry Pluviation

b) Water Pluviation

c) Moist Tamping

Figure 3.6 Different sample preparation techniques

CHAPTER 4

PRE-LIQUEFACTION MONOTONIC BEHAVIOUR

The undrained monotonic behaviour of Silica and Fraser River Sand is presented in this chapter. Most of the tests were carried out using the simple shear apparatus and a limited number of tests were carried out using the triaxial device.

General characteristics of the undrained behaviour of the dry pluviated samples and the dependency of their monotonic response on relative density and consolidation stress level are discussed in the first section of this chapter. A comparison between response of the dry pluviated and water pluviated samples is discussed in the second section. Moist tamping method was used to produce specimens at very loose states, both in Silica and Fraser River Sand in order to investigate the concept of a unique steady state dependent on void ratio only.

4.1 General Characteristics of Dry Pluviated Specimens

The sand behaviour was investigated at specific targeted densities of 30%, 50% and 70% under simple shear mode. A range of consolidation stress levels (100, 200 and 400 kPa) were used at each targeted density to investigate the domains of contractive and dilative behaviour as influenced by the initial state characterized by consolidation stress and relative density after consolidation. The characteristics of stress path, CSR, PT and failure lines are then analyzed, and the influences of consolidation stress level and relative density on them were assessed.

Minimum void ratio that achieved in triaxial specimens was lower than that in simple shear specimens. This enables the investigation of pluviated sand behaviour at loosest possible relative density. Response of Silica Sand in triaxial compression and extension modes is compared with response in simple shear mode to explore the influence of stress path.

In simple shear tests, only the shear and normal stresses on the horizontal plane are measured. Extra information is required to determine the complete stress state in the sample. Roscoe (1970) has demonstrated that the horizontal plane is generally the plane of maximum shear stress in simple shear. Interpretation of test data in this thesis is based on the above premise.

4.1.1 Monotonic Response at Fixed Density State

The desired relative density after consolidation may be achieved by consolidating sand from different initial void ratio. Compressibility of sand decreases with decreasing initial void ratio as shown in Figure 4.1. This information used as a guideline for achieving the desired void ratio at a certain consolidation pressure.

The minimum relative density that could be achieved at 400 kPa consolidation pressure was 30%. Therefore, 30% relative density was considered as the minimum relative density used for comparison between sand response at different consolidation pressures. Relative density of 50% and 70% were used as other states that could be considered as medium and dense states.

The undrained monotonic response of Silica Sand at three fixed consolidated relative densities is shown through Figure 4.2 to Figure 4.4. This enables isolation of the effect of consolidation stress alone on stress-strain behaviour. At low relative density of 30%, all the responses are contractive. Increasing consolidation stress does not significantly affect the degree of contractive response as evidenced by an approximately constant brittleness index of $I_B=0.3\sim0.4$. At medium relative density under 100 kPa consolidation pressure, the behaviour of sand is dilative, but at 200 and 400 kPa consolidation stress the sand is strain softening with limited strain. At dense state that is 70% relative density, the sand behaves dilative in all consolidation stress level. Increasing consolidation stress generally transfers the sands behaviour into a more contractive response, but it has a minor influence on the response compared to relative density.

4.1.2 Monotonic Response at Fixed Consolidation Pressure

Testing the sand at a fixed consolidation pressure enables the effect of relative density to be isolated. The monotonic response of sand at three fixed consolidation stresses is shown through Figure 4.5 to Figure 4.7. Clearly, the relative density has much more effect than consolidation pressure on response of sand. A contractive behaviour at 100 kPa consolidation stress and 30% relative density is changed to a dilative response at the same consolidation stress but at 50% or 70% relative density. Increasing the relative

density makes an already dilative behaviour more dilative and an already contractive behaviour less contractive or even dilative.

4.1.3 Critical Stress Ratio Line

The friction angle ϕ_{CSR} mobilized at the peaks of shear stress for the samples that show contractive response is a function of relative density. The influence of relative density on ϕ_{CSR} is shown in Figure 4.8. The friction angle at CSR increases with an increase in relative density. Thomas (1992) has shown that ϕ_{CSR} is a function of placement void ratio and is smaller in triaxial extension than in compression loading mode. Data reported by Chern (1985), Chung (1985) and Kuerbis (1989) also support this idea. Vaid and Sivathayalan (1996) have shown that the friction angle at peak is a function of void ratio after consolidation both in simple shear and triaxial extension tests.

4.1.4 Phase Transformation State

Figure 4.9 shows the effective stress conditions at phase transformation state (PT) which represents the state at which the soil begins dilation. This is reflected during the test by a decrease in pore pressure with further straining. The effective stress conditions at PT state fall within a unique straight line passing through the origin regardless of relative density, initial stress and type of response (contractive or dilative). This line has a unique slope implying that the friction angles at phase transformation state for both contractive and dilative response are equal. Thomas (1992) has shown that the slope of PT line is unique in triaxial extension and compression modes and does not depend on loading mode consequently. The value of friction angle at PT line is found to be a constant and equal to 25° for the Silica Sand.

Figure 4.10 shows the relationship between shear strength at quasi steady state (S_{QSS}) and consolidated void ratio for three different consolidation pressures. As previously pointed out, limited liquefaction only occurred for samples at loose states, and medium density states with high consolidation pressure. At a given void ratio, the shear strength at QSS increases with initial consolidation stress. Kuerbis (1989) also noted that shear strength at QSS is not only a function of void ratio but also a function of consolidation stress. Figure 4.11 illustrates the normalization of quasi steady state strength by consolidation stress level. The normalization reduces the scatter in Figure 4.10 and all the QSS lines will fall within a narrow band.

4.1.5 Ultimate Failure Line

Effective stress conditions at maximum obliquity are shown in Figure 4.12. The ultimate failure line connecting the stress conditions at maximum obliquity passes through origin and has a constant slope. This straight line implies a mobilized friction angle of about 29 degrees regardless of type of behaviour (contractive or dilative), relative density and consolidation pressure level.

4.1.6 Monotonic Behaviour at the Loosest State in Triaxial

Loosest relative density for pluviated sand in triaxial device was 15% at 100 kPa consolidation stress, and 18% at 400 kPa consolidation stress. Both triaxial compression and extension responses are shown in Figure 4.13. All the responses in loosest state are contractive regardless of loading mode. In compression mode, however, samples show relatively stronger behaviour.

The degree of contractive response can be conveniently characterized in terms of brittleness index, I_B . Brittleness index is considered indicative of the flow potential of contractive sand. I_B changes from 0.33 to 0.4 in compression mode as confining stress increases from 100 to 400 kPa. In extension mode, it changes from 0.66 to 0.72 with an increase of confining stress from 100 to 400 kPa. This suggests that the loading mode has much more influence on contractiveness of sand compared to confining stress.

4.1.7 Comparison of Triaxial and Simple Shear Response

The triaxial compression and extension behaviour of Silica sand is compared with simple shear response at two different relative densities as shown in Figure 4.14. The simple shear response falls between triaxial extension and compression but is closer to triaxial extension response. Inherent anisotropy in pluviated sand has been considered responsible for the differences in compression, extension and simple shear behaviour for a given initial state.

The behaviour of pluviated Silica sand at 30% relative density is contractive in triaxial compression, even though most pluviated sands exhibit dilative responses in triaxial compression even at the loosest state (Thomas, 1992). The triaxial compression response at 50% relative density is dilative while it is contractive in extension mode.

4.2 Effect of Sample Preparation on Monotonic Behaviour of Sand

Three different sample preparation techniques were used in this study to investigate the influence of fabric on behaviour of sand. Specimens were prepared by dry pluviation, water pluviation and moist tamping techniques. However, only limited numbers of tests were carried out using moist tamping method.

4.2.1 Monotonic Stress-Strain Curve for Water Pluviated Specimens

The monotonic response of dry pluviated and water pluviated samples at three different consolidation pressures are shown in Figure 4.15. The minimum relative density in water pluviated samples, which is 34%, is achieved at 100 kPa consolidation stress. The monotonic responses are the similar for water pluviated and dry pluviated sands.

Vaid et al. (1999) have shown that water pluviation response is much stronger than dry pluviation response. The sands that they used for testing were Fraser River and Syncrude tailing sand which were angular to sub angular sands with some fines. On the other hand, the Silica Sand used in this study is a rounded to sub rounded sand without any fine particles. This suggests that the medium of pluviation does not have a major effect on the fabric for Silica Sand. The differences in response in the other sands were apparently due to the presence of fine particles. The speed of particles in air and water are different, and this just controls the density of samples but because of the spherical shape and the relatively uniform size of particles for Silica Sand the inter-particles arrangement is very similar in dry pluviated and water pluviated samples.

4.2.2 Steady state in loosest state

Failure of earth structures, which involves relatively loose sandy soils, is caused by loss of stability and strength of sand. The key parameter for estimating the failure is the minimum undrained shear strength which is called steady state strength if the mode of deformation is of type 1 or flow failure.

In dry pluviated specimens, the loosest state is 30% relative density, and no steady state type of behaviour is observed. However, using moist tamping method, it is possible to reach to very high void ratio, even higher than maximum void ratio achieved

based on ASTM. The maximum void ratio based on ASTM for Silica Sand is 0.723 which corresponds to essentially zero effective stress, while the maximum void ratio achieved in moist tamped specimens at 100 kPa consolidation stress is 0.750. The steady state type of behaviour therefore is studied using moist tamped sands. The variation of the minimum undrained strength with void ratio is shown in Figure 4.16 for Silica Sand. Comparison between quasi steady state in pluviated samples and steady state in moist tamped samples shows that the quasi steady state strength is a function of void ratio and consolidation pressure but steady state strength is a function of void ratio only.

Moist tamped specimens of Fraser River Sand were also prepared at much looser states than ASTM maximum void ratio, and they exhibited contractive steady state type of response. Figure 4.17 shows the variation of steady state strength with void ratio for Fraser River Sand. It is worthwhile noting that pluviated Fraser River Sand does not exhibit steady state type of response (Vaid and Thomas, 1995; Vaid and Sivathayalan, 1996) and the minimum shear strength normalised by consolidation pressure again falls within a narrow band in pluviated Fraser River Sand.

It could be concluded that both Silica Sand and Fraser River Sand show that S_{us} (undrained steady state strength) is a function of void ratio only at very loose state. However, the quasi steady state strength is a function of void ratio and consolidation stress for pluviated sands.

The dependency of steady state strength on void ratio has been reported by other researchers. Poulos et al. (1985) have considered the undrained steady state strength of a soil to be a function of in-situ void ratio only. It is not dependent on the soil structure or on the method or rate of loading. Also, they have concluded that the steady state strength

is very sensitive to void ratio. Re-examination of slide of lower San Fernando dam by Castro et al. (1985) also has supported this fact that the steady state strength is only a function of void ratio.

Figure 4.1 e-log p diagram for Silica Sand simple shear specimens

Figure 4.2 Effect of consolidation stress on monotonic behaviour of Silica Sand at $D_{rc} = 30\%$

Figure 4.3 Effect of consolidation stress on monotonic behaviour of Silica Sand at $D_{rc} = 50\%$

Figure 4.4 Effect of consolidation stress on monotonic behaviour of Silica Sand at $D_{rc} = 70\%$

Figure 4.5 Effect of relative density on monotonic behaviour of Silica Sand at $\sigma'_{vc} = 100$ kPa

Figure 4.6 Effect of relative density on monotonic behaviour of Silica Sand at $\sigma'_{vc} = 200$ kPa

Figure 4.7 Effect of relative density on monotonic behaviour of Silica Sand at $\sigma'_{vc} = 400$ kPa

Figure 4.8 Friction angle at CSR for Silica Sand

Figure 4.9 Friction angle at PT for Silica Sand

Figure 4.10 Quasi steady state dependent on void ratio and stress level for Silica Sand

Figure 4.11 Normalised quasi steady state line for Silica Sand

Figure 4.12 Ultimate failure line for Silica Sand

Figure 4.13 Triaxial compression and extension response for Silica Sand at loosest state

Figure 4.14 Comparison of triaxial and simple shear response for Silica Sand

Figure 4.15 Comparison of water pluviation and dry pluviation response

Figure 4.16 Steady state strength for moist tamped Silica Sand

Figure 4.17 Steady state strength for moist tamped Fraser River Sand

CHAPTER 5

POST LIQUEFACTION BEHAVIOUR

Attempts to predict liquefaction induced displacements require knowledge of the nature of the stress-strain, and strength characteristics of the liquefied soil. The post liquefaction behaviours of Silica and Fraser River Sand, in terms of stress-strain and strength characteristics are presented in this chapter. Most of the tests were conducted under simple shear loading mode and a limited number of tests were conducted under triaxial mode. Triaxial tests were carried out both in compression and extension loading modes. The post liquefaction response of sand was studied under different density and consolidation pressure states, in addition to different values of maximum shear strain prior to the post liquefaction loading.

The post liquefaction simple shear response of Silica Sand is presented in the first part of this chapter. Liquefaction was induced by cyclic loading prior to post

liquefaction monotonic loading. The post liquefaction strength is normalised by consolidation pressure to evaluate the characteristics of the response of the liquefied soil.

The post liquefaction behaviour of Fraser River Sand under simple shear condition is discussed in the second part of this chapter. Fraser River Sand is more dilative compared to Silica Sand. The objective was to study the influence of the degree of contractiveness on the post liquefaction characteristics of sand.

In the third section of this chapter, the post liquefaction behaviour of Silica sand in triaxial compression and extension modes is compared with simple shear results. Due to preparation technique and sample size, it is possible to have looser density states in triaxial samples compared to simple shear samples. Comparison between the pre-liquefaction and post liquefaction behaviour in simple shear and triaxial compression and extension mode is presented. The experimental results are also compared with the data available in the literature.

5.1 Post Liquefaction Behaviour of Silica Sand in Simple Shear

Simple shear tests on Silica sand were conducted at three different vertical effective confining stresses (100, 200 and 400 kPa), and three different relative density (30%, 50% and 70%) levels. The dependence of the post liquefaction response on the maximum strain during cyclic loading was assessed by terminating the cyclic shearing at specified strain levels. Tests were conducted on both dry pluviated and water pluviated samples.

5.1.1 Range of Post Liquefaction Behaviour

Post liquefaction stress-strain curve can be characterized into three distinct regions as shown in Figure 5.1. Region 1 spans from the state at which the effective shear stress is zero at the end of pre-liquefaction cyclic or monotonic loading, and is characterized by deformation with essentially zero stiffness. Region 2 characterizes deformation with increasing shear modulus. Region 2 can be approximated as a parabolic curve representing continuously increasing stiffness with strain. Deformation in region 3 occurs with essentially constant modulus.

Following deformation at essentially constant modulus, some tests do exhibit a tendency to reach a residual state at very large strains (typically beyond 30%). However, stress non-uniformities in a simple shear specimen are expected to be rather serious at such large strain levels. The confidence in the measured stress-strain data at such large strains is not very high, and therefore no attempts have been made to interpret the data. It should be noted that most specimens did not realise a residual state even when the shear strains exceeded 35%.

The phenomenon of increasing stiffness with strain is opposite to the commonly assumed behaviour of soils where straining is associated with a loss of stiffness. This unusual stress-strain response is reflection of the fact that deformation during post liquefaction loading is associated with dilation, which causes effective stresses to increase under undrained conditions. Similar increase in stiffness with strain may be noted following the phase transformation phase in pre-liquefaction loading. Since the cyclic loading leading to a state of zero effective stress essentially eliminates the contractive zone (which is responsible for deformation prior to the phase transformation

state in pre-liquefaction loading) it is rational to expect modulus increase with strain in post liquefaction.

To have a common base point for comparison between different post liquefaction responses, the borders between different regions should be clearly defined. Thomas (1992) defined shear stress equal to 2.5 kPa as the end of region 1. In this study, the end of region 1 is defined based on a normalised shear stress level of 2% of effective consolidation stress.

5.1.2 Effect of Density

The influence of relative density on the post liquefaction response of Silica Sand is illustrated in Figure 5.1. The response is compared at relative density level of 30%, 50% and 70% at a fixed consolidation stress level. The maximum pre-liquefaction strain level was held constant in each group of the tests shown in Figure 5.1.

Figure 5.1 clearly indicate that the post liquefaction response is highly dependent on the relative density of the sand at all initial confining stress levels. Both the strength and the average shear modulus increase rapidly with increasing density. It may be noted that the rate of build up in shear stress beyond region 1 (region 2) increases as the relative density increases and the shear strain at which the curves become linear decreases with an increase in relative density. Stiffness in region 3, also, increases with increasing relative density.

5.1.3 Effect of Consolidation Pressure

Figure 5.2 and Figure 5.3 show the influence of consolidation pressure on post liquefaction response of Silica Sand. The amount of strain over which sample has

essentially zero stiffness decreases with an increase in consolidation pressure. The rate of increase in stiffness in region 2 increases with consolidation pressure. Also, the shear modulus at large strains (region 3) increases with an increase in consolidation pressure but the rate of increment becomes less when the consolidation stress increases to larger value. However, the effect of consolidation stress is not as significant as relative density.

5.1.4 Effect of Maximum Pre-liquefaction Strain Level

In order to study the effect of strain level on post liquefaction response, three different maximum pre-liquefaction strain levels were considered at the end of cyclic or monotonic pre-liquefaction loading. Although it was difficult to terminate the test at a certain amount of strain especially for loose sands in stress control loading, feedback control software was used to limit the maximum strains at approximately 4%, 8% and 12% shear strain.

The larger maximum pre-liquefaction strain level results in a much prolonged region of deformation at essentially zero stiffness as shown in Figure 5.4 to Figure 5.6. The stiffness in region 2 increases with a faster rate as the maximum pre-liquefaction strain decreases. The final stiffness in region 3 increases with a decrease in maximum pre-liquefaction strain level as well.

It can be clearly noted that increasing strain during cyclic loading drastically damages the post-liquefaction performance of the sand at a given confining stress and relative density. The sand at an initial confining stress of 200 kPa, and relative density 50% reaches 40 kPa shear stress at about 8% strain when the maximum shear strain during cyclic loading was about 4%. However, if the maximum strain during cyclic loading was 12%, the sand requires well over 25% shear strain to reach the same shear

stress level in post liquefaction loading. Similar influence of the maximum strain during cyclic loading can be consistently noted at all relative density and confining stress levels.

5.1.5 Characterisation of Region 1 Based on Previous Studies

As discussed earlier characteristics of region 1 may be affected by three main factors: relative density, consolidation pressure and maximum strain level prior to liquefaction. The variation of shear strain in region 1 may be defined as the strain required for developing a certain amount of shear stress or normalised shear stress following liquefaction. Liquefaction, here, means the state of zero shear stress or 100% pore pressure at the end of pre-liquefaction loading.

Thomas (1992) has done a comprehensive study on the post liquefaction behaviour of sands. He characterized region 1 based on the assumption that a constant amount of deviator stress signifies the end point of region 1. He has set $\sigma_d = 5$ kPa in triaxial test to define the end of region 1, which corresponds to $\tau = 2.5$ kPa in simple shear test. Figure 5.7 shows the characteristics of region 1 based on this definition for different consolidation stresses and relative densities as a function of the maximum pre-liquefaction shear strain level. The data indicate that the maximum pre-liquefaction strain level and the confining stress state play major roles in strain development during this region. With increasing the consolidation pressure at a given relative density and pre-liquefaction strain level the amount of strain required to mobilize 2.5 kPa shear stress decreases. Data are presented for different confining stress and density levels, and considerable scatter can be noted at the loosest density state. The data indicate that confining stress level also plays a role in strain development during this phase, especially

at loose density states. The amount of strain during region 1 significantly increases with increasing the maximum pre-liquefaction strain level.

5.1.6 Characterisation of Region 1 Using Normalised Shear Stress

Characterizing region 1 based on a constant shear stress level causes it to be dependent on various parameters including consolidation stress level. An attempt is made here to characterize region 1 based on shear stress normalised by the consolidation stress. Deformation from the instant of liquefaction to the state of normalised shear stress equal to 2% of effective consolidation stress is considered to define region 1 in this approach.

Data from the tests shown in Figure 5.7 are replotted in Figure 5.8 to Figure 5.10 with this newer definition. A significant reduction in the scatter can be noted in the normalised plot; however a strong dependency of this region on the maximum pre-liquefaction strain level still exists.

Data from both dry pluviated and water pluviated sand are shown in the figures. The loosest void ratio attainable at a given consolidation stress by water pluviation method is slightly larger than the loosest void ratio in dry pluviated sand. This results in a small difference between response of dry pluviated and water pluviated sand characteristics in region 1 at loosest state (Figure 5.8). The loosest void ratio attainable in dry pluviated and water pluviated samples correspond to about 30% and 34% relative density respectively.

The influence of relative density on the characteristic of region 1 is shown in Figure 5.11. The effect of relative density on region 1 is not as significant as the effect of strain level. The amount of strain required to mobilize a shear stress of 2% of the initial effective stress decreases with increasing density.

5.1.7 Characterization of Post Liquefaction Strength

The characteristics of the deformation subsequent to the initial phase are evaluated by considering the mobilised shear stress. The undrained post liquefaction strength is a key parameter required in evaluating liquefaction induced displacements. However, the sand did not reach a residual state in most cases, even when shear strains exceeded 30%. Therefore, the shear stress at a specific strain level is considered indicative of the shear strength of the sand. Deformation during this second phase is strongly dependent on the relative density, whereas deformation during the first phase was not significantly dependent on it. Therefore, shear strain measured from the end of the first phase is regarded as a proper measure of the performance during the second phase of deformation. The post liquefaction strength is characterized by stress level at 10% shear strain following region 1. It is fully recognized that this definition is arbitrary, but it is adopted here to provide a basis for comparison of post liquefaction strength affected by different factors like relative density, consolidation pressure and maximum pre-liquefaction strain level.

Figure 5.12 to Figure 5.14 show the post liquefaction strength at different consolidation stress levels and relative densities as a function of maximum pre-liquefaction strain level. Both dry pluviated and water pluviated sands behave in the same manner and have similar strength. Again, because of difference in the average of maximum void ratio in loosest state between dry pluviated and water pluviated samples there is a minor difference in their post liquefaction strength.

Unlike in the region 1, the post liquefaction response during region 2 is highly dependent on relative density. As it is clear from the figures, with an increase in relative

density from loose to medium dense state the post liquefaction strength is almost doubled for a constant confining stress. Although the consolidation stress does not influence post liquefaction response in second region as significant as relative density, it plays a role in the post liquefaction strength. The post liquefaction strength decreases dramatically with an increase in maximum pre-liquefaction strain level. By increasing the pre-liquefaction strain level from 4% to 12% shear strain, the post liquefaction strength drops by a factor of 1.5 to 2 at most relative density and consolidation stress states.

The data shown through Figure 5.12 to Figure 5.14 are normalized by initial consolidation stress and re-plotted in Figure 5.15. The main purpose of normalization by consolidation stress is to highlight the influence of relative density and maximum pre-liquefaction strain level on the post liquefaction strength. Normalised shear strength of sands have been reported by different researchers, both with regard to monotonic and post liquefaction loading (Vaid and Sivathayalan, 1996; Finn, 1998; Olson and Stark 2002).

Three distinct zones or bands, each corresponding to a certain density can be recognized in the figure. The post liquefaction strength increases with an increase in relative density as discussed earlier. The scatter in normalised data for all zones decrease with an increase in maximum pre-liquefaction strain level and the bands become closer to each other at large pre-liquefaction strain level. This suggests that the effect of density is less significant at relatively large pre-liquefaction strain levels. On the other hand, the scatter increases with density and maximum pre-liquefaction strain, especially for dense sand. The large scatter in 70% relative density zone may indicate that normalization does

not work very well for sands with dilative behaviour, especially when they have been sheared to relatively small levels of shear strain during pre-liquefaction loading.

The normalised post liquefaction strength of loose Silica Sand varies from about 0.05 to 0.18, and that for dense sand varies from about 0.3 to 0.75 depending on maximum pre-liquefaction strain. Olson and Stark (2003) have computed liquefied strength ratio values using experimental data in the literature and report values as low as 0.02. These low values generally correspond to artificially loose, negative relative density states. Specimens reconstituted by moist tamping may be consolidated to very loose states, but the relevance of such data to actual soils in-situ is questionable. Data corresponding to consolidated relative densities of as low as -20% have been reported by Olsen and Stark (2003). In contrast, the loosest relative density attainable by the pluviated sand at 100 kPa confining stress level is typically about 20%.

5.1.8 Failure Line

Figure 5.16 shows the range of ultimate failure line or line of maximum obliquity for Silica Sand. The post liquefaction deformation proceeds along the average line of maximum obliquity regardless of the relative density, consolidation stress and maximum pre-liquefaction strain level. The average value of friction angle during post liquefaction shear is equal to the failure friction angle noted in monotonic pre-liquefaction loading.

5.2 Post Liquefaction Response of Fraser River Sand in Simple Shear

In order to confirm that normalised shear stress may be a better tool to characterize the post liquefaction response of sands in general, and to study the effect of dilation potential of sands on the characteristics of the post liquefaction behaviour, a

series of tests were conducted on Fraser River Sand. Tests were carried out under simple shear mode using three different densities and three different consolidation pressures with three different maximum pre-liquefaction strain levels. Fraser River Sand is a sub-angular sand and is generally more dilative compared to Silica Sand (Thomas, 1992). Simple shear specimens of Fraser River Sand were prepared by the dry pluviation technique.

5.2.1 Characterisation of Region 1

The characteristics of region 1 for Fraser River Sand are shown in Figure 5.17 using the normalised shear stress approach proposed in 5.1.6. Three distinct lines are observed in Figure 5.17 corresponding to the three different relative densities. The characteristics of Fraser River Sand in region 1 are similar to that of Silica Sand with respect to the effects of different factors on this region. The amount of strain in region 1 increases with an increase in maximum pre-liquefaction strain level. The slope of lines in region 1 for Fraser River Sand at similar density states is very close to that of Silica Sand. This indicates that the degree of contractiveness does not affect the characteristics of region 1.

5.2.2 Characterisation of Post Liquefaction Strength

The post liquefaction strength of Fraser River Sand is displayed in Figure 5.18 for three different consolidation pressures at 27% relative density. The post liquefaction strength is evaluated by using the mobilized shear stress at an arbitrary strain level of 10% beyond region 1, as noted earlier. The general characteristics of the post liquefaction response of Fraser River Sand are similar to that of Silica Sand with respect to relative density, consolidation pressure and maximum pre-liquefaction strain level. The post

liquefaction strength increases with an increase in consolidation stress. Also, the post liquefaction strength decreases with an increase in maximum pre-liquefaction strain level.

The normalised post liquefaction strength for Fraser River Sand at 27% relative density varies from about 0.1 to 0.3 as illustrated in Figure 5.18. This indicates relatively stronger post liquefaction response compared to the normalised post liquefaction strength of Silica Sand at 30% relative density that varies from about 0.05 to 0.18. It is concluded that the differences arise on account of the characteristics of the contractiveness potential of sands.

5.3 Post liquefaction Behaviour of Silica Sand in Triaxial Test

According to several studies, monotonic behaviour of sand under simple shear mode falls between the behaviour under compression and extension modes. This characteristic is due to the differences in stress path indicated by the direction of principal stresses and value of intermediate principal stress in simple shear and triaxial compression and extension loading modes. Based on the knowledge of author, there are only limited amount of published experimental work that investigates the effect of stress path on the post liquefaction behaviour of sand.

A series of strain controlled triaxial compression and extension tests were conducted on Silica Sand to investigate the effect of stress path on the post liquefaction behaviour. The loosest density state attainable in the triaxial specimen is lower than that in simple shear sample. All triaxial samples were isotropically consolidated. Vaid and Thomas (1995) have shown that the post liquefaction behaviour doesn't depend on the type of pre-liquefaction loading that triggered liquefaction. The post liquefaction response of sand consolidated to the same confining stress at identical density resulted in

the same behaviour regardless of whether the sand liquefied as a result of cyclic loading or on account of a monotonic load/unload cycle. So in this study, the liquefaction in triaxial tests is triggered by unloading the sand following monotonic shearing to the desired strain levels.

5.3.1 Characterisation of Region 1 at Loosest Density State

Figure 5.19 shows the characteristics of region 1 for loosest density state for Silica Sand at 100 and 400 kPa consolidation pressures under both compression and extension loading modes. Samples in post liquefaction extension mode show weaker behaviour and need more strain to develop a certain amount of strength compared to samples in compression mode. Again, the amount of strain in region 1 increases with an increase in maximum pre-liquefaction strain as observed in simple shear test.

5.3.2 Comparison of Region 1 in Simple Shear and Triaxial Tests

The size and characteristic of region 1 for the samples loaded under simple shear and triaxial compression and extension modes is compared at identical relative densities in Figure 5.20. The shear strain needed to develop 2% of consolidation stress in region 1 in compression is smaller than that in extension mode both at 30% and 50% relative densities. However, this amount of strain in simple shear mode is much larger than that in triaxial extension and compression mode, and it increases with a decrease in relative density. This indicates that the deformation during first region is stress path dependent.

5.3.3 Characterisation of Post Liquefaction Strength at Loosest Density State

Post liquefaction strength of Silica Sand in compression and extension loading modes is shown in Figure 5.21 at loosest density state. Post liquefaction strength is

estimated based on the previous guidelines noted in 5.1.7. Two different consolidation pressures are used in these tests.

The dependency of the post liquefaction strength to the consolidation pressure and the maximum pre-liquefaction strain level is similar to what observed in simple shear tests. Post liquefaction strength in compression mode is higher than that in extension mode for both consolidation pressures. This is consistent with widely reported anisotropic nature of pluviated fabric.

5.3.4 Comparison of Post Liquefaction Strength in Triaxial and Simple Shear Test

A series of tests at 30% and 50% relative densities were conducted under 100 kPa consolidation pressure both in compression and extension loading modes. Liquefaction is triggered by a monotonic load/unload cycle. To have post liquefaction shear in extension, pre-liquefaction loading is in compression and vice versa.

Figure 5.22 and Figure 5.23 show the post liquefaction strength at 30% and 50% relative densities for compression, extension and simple shear loading modes. The post liquefaction strength in triaxial compression and extension -as in simple shear- increases with an increase in relative density and decreases with an increase in maximum pre-liquefaction strain level. As it is clear from the figures, the strength in compression mode is much higher than that in the simple shear and extension mode, but the difference become smaller when the maximum pre-liquefaction strain becomes larger. This suggests that the large strains that developed during liquefaction have diminished the material anisotropy. Also, the post liquefaction strength in extension mode is larger than that in the simple shear. The data suggest that the deformation during the post liquefaction loading is also stress path dependent.

5.3.5 Differences between Simple Shear and Triaxial Post Liquefaction Response

The fact that the post liquefaction extension behaviour is stronger than simple shear is opposite to observation of pre-liquefaction results in which the simple shear behaviour is stronger than triaxial extension. The differences between different stress paths-compression, extension and simple shear- arise from the differences in the direction of principal stresses and the orientation of maximum and minimum principal stresses in triaxial compression, extension and simple shear loading mode in addition to differences in the magnitude of intermediate principle stress.

Two distinct variables which determine the direction of principle stress and the magnitude of intermediate stress are α which is the angle between the major principal stress and the vertical axis of sample, and $b=(\sigma_2-\sigma_3)/(\sigma_1-\sigma_3)$ which represents the influence of intermediate principal stress. These values for triaxial compression are $\alpha=0^\circ$ and $b=0$, and for triaxial extension are $\alpha=90^\circ$ and $b=1$. In plane strain simple shear, these values are $\alpha=45^\circ$ and $b=0.3$ to 0.4 based on Yoshimine et al. (1998). It has been demonstrated using hollow cylinder torsional device that as α increases from 0° to 90° the sand behaviour becomes more contractive (Uthayakumar and Vaid, 1998). So, these values explain why the pre-liquefaction response in simple shear mode is between triaxial compression and extension mode.

Material anisotropy and the effect of intermediate principal stress do not explain the observed post liquefaction response, where simple shear seems weaker than triaxial extension response. The actual reason for such a response is unknown at this time, but it further suggests that the initial material anisotropy is altered during the cyclic/monotonic loading leading to liquefaction.

5.4 Comparison of Pre-liquefaction and Post Liquefaction Behaviour

Pre-liquefaction and post liquefaction stress-strain response of the Silica Sand at the relative density of 50% consolidated to an effective stress of 100 kPa in triaxial tests, and 100 and 400 kPa in simple shear test are compared in Figure 5.24 and Figure 5.25. The stiffness of the sand decreases with an increase in strain until the phase transformation state for the sand subjected to pre-liquefaction static loading, but it continuously increases with strain during post liquefaction monotonic loading. The pre-liquefaction response could be contractive or dilative depend on stress path, consolidation stress and density. However, Post liquefaction stress-strain response is always dilative (if the sand is at the state of zero effective stress at the end of pre-liquefaction loading). The deformation in post liquefaction occurs at a mobilized friction angle equal to the friction angle at the maximum obliquity obtained for the pre-liquefaction monotonic loading. The post-liquefaction stiffness is very small during initial phase of loading, but with an increase in strain level, it can be noted that the stiffness essentially becomes equal to the pre-liquefaction value in the post phase transformation region of the pre-liquefaction response.

5.5 Comparison of Experimental Data with Field Observations

By back-analyzing embankments which had undergone significant displacement during earthquakes, Seed (1987) determined representative values of residual strength. In (1990), Seed and Harder developed a chart which characterizes the residual strength as a function of the normalised Standard Penetration Resistance, $(N_1)_{60}$. A re-evaluation of the liquefaction induced failure of the San Fernando dam indicated that the 35 percentile

residual strength based on laboratory data would predict failure of the San Fernando dam (Seed et al., 1989). This suggests that, on the average, laboratory triaxial compression tests overestimate the residual strength.

The results of simple shear post liquefaction strength are compared with in-situ data published by Seed and Harder (1990) in Figure 5.26. The values of $(N_1)_{60}$ corresponding to different densities are estimated using the correlations proposed by Skempton (1986). The results show that the simple shear strength measured in the laboratory is higher than the computed in-situ residual strength. The lower bound of the measured simple shear response is close to upper bound values of the Seed and Harder chart. It should be noted that the value of post liquefaction residual strength in simple shear tests is conservative, because in some tests the post liquefaction response did not reach to an actual residual strength and the shear stresses at the end of tests were considered as the representative of the residual strength. So, in reality the actual residual strength would even be higher than those plotted in Figure 5.26. This suggests that universal applications of the Seed and Harder chart may lead to conservative designs, if the field loading conditions are close to simple shear.

Residual strength of Fraser River Sand which is a more dilative sand compared to Silica Sand is higher than residual strength of Silica Sand at the identical density as shown in Figure 5.26. Again, this suggests that the contractiveness of sand in pre-liquefaction behaviour has a direct influence on the post liquefaction response of sands. Also, it may be recognized that the effect of contractiveness is minor at looser density as the boundaries for Silica Sand and Fraser River Sand data are close to each other.

Comparison of residual strength in compression and extension mode with in-situ data (Seed and Harder, 1990) is shown in Figure 5.27 for Silica Sand. Clearly, the residual strength under compression loading is much higher than both extension and in-situ residual strength. Measured residual strength under extension loading is also higher than in-situ data and is closer to upper bounds of the Seed and Harder data. Figure 5.26 and Figure 5.27 clearly show that Seed and Harder chart, which is widely used in practice, is conservative in each of the three loading modes considered in this study. It should be noted that Seed and Harder chart presents an average value of the shear strength to be assigned to the boundary value problem that exists in the field, while the laboratory tests represent the element response under a given loading mode at a specific stress and density level.

Figure 5.1 Effect of density on post liquefaction response

Figure 5.2 Effect of consolidation stress on post liquefaction response $D_{rc}=50\%$

Figure 5.3 Effect of consolidation stress on post liquefaction response $D_{rc}=70\%$

Figure 5.4 Effect of maximum pre-liquefaction strain on post liquefaction response
 $D_{rc}=30\%$

Figure 5.5 Effect of maximum pre-liquefaction strain on post liquefaction response
 $D_{rc}=50\%$

Figure 5.6 Effect of maximum pre-liquefaction strain on post liquefaction response
 $D_{rc}=70\%$

Figure 5.7 Region 1 based on Thomas (1992) definition

Figure 5.8 Region 1 based on new definition for the loosest deposited sand

Figure 5.9 Region 1 based on new definition $D_{rc}=50\%$

Figure 5.10 Region 1 based on new definition $D_{rc}=70\%$

Figure 5.11 Effect of density on region 1

Figure 5.12 Post liquefaction strength for the loosest deposited sand

Figure 5.13 Post liquefaction strength $D_{rc}=50\%$

Figure 5.14 Post liquefaction strength $D_{rc}=70\%$

Figure 5.15 Normalised post liquefaction shear strength

Figure 5.16 Ultimate failure line

Figure 5.17 Region 1 for Fraser River Sand

Figure 5.18 Post liquefaction strength for Fraser River Sand $D_{rc}=27\%$

Figure 5.19 Region 1 in triaxial tests for loosest state

Figure 5.20 Comparison of region 1 in triaxial and simple shear tests

Figure 5.21 Post liquefaction strength in triaxial tests for loosest state

Figure 5.22 Comparison of post liquefaction strength in triaxial with simple shear tests for $D_{rc}=30\%$

Figure 5.23 Comparison of Post liquefaction strength in triaxial with simple shear tests for $D_{rc}=50\%$

Figure 5.24 Comparison of pre and post liquefaction behaviour in simple shear

Figure 5.25 Comparison of pre and post liquefaction in triaxial compression and extension

Figure 5.26 Comparison of in-situ with simple shear post liquefaction data

Figure 5.27 Comparison of in-situ with triaxial post liquefaction data

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 Summary

An experimental study has been carried out to investigate the post liquefaction deformation characteristics of sands. Knowledge of the mechanics of post liquefaction response is necessary for evaluating the settlement, and deformation of liquefied soil deposits. A series of pre-liquefaction and post liquefaction tests were carried out on Silica Sand and Fraser River Sand under simple shear and triaxial loading modes. Test specimens were prepared by employing different reconstitution techniques.

Pre-liquefaction monotonic tests were carried out using three different consolidation stresses (100, 200 and 400 kPa) and three different relative densities (30%, 50% and 70%). Moist tamping method was used to study the behaviour of the Silica and Fraser River Sand in loosest state.

Post liquefaction monotonic tests were conducted on both sands following liquefaction induced by either cyclic loading (in simple shear) or monotonic load/unload cycle (in triaxial). Both pre and post liquefaction tests were conducted at similar relative density and consolidation stress states. In addition, the effect of the maximum pre-liquefaction strain at the end of pre-liquefaction loading was evaluated. The influence of loading mode on the post liquefaction response was investigated by comparing the response under triaxial and simple shear loading modes. The residual shear strength evaluated by laboratory data was compared with the in situ residual strength values used in current practice.

6.2 Conclusions

The following observations and conclusions are drawn from the data presented in the thesis:

- Silica Sand generally behaves in contractive manner under simple shear loading mode. As expected, the response generally transforms into a dilative one as relative density increases. Increasing confining stress level also promotes more contractive response, but at a smaller rate.
- The friction angle ϕ_{CSR} that corresponds to the triggering of strain softening deformation increases with increasing relative density.
- Pre-liquefaction monotonic response of sand in simple shear is weaker than that in triaxial compression and stronger than triaxial extension. Generally simple shear response is closer to triaxial extension than compression. As noted in the literature, this is a manifestation of the inherently anisotropic fabric in pluviated sands.

- The moist tamping method enables specimen reconstitution at very loose void ratios (even looser than ASTM maximum void ratio). This permits an investigation of the steady state strength of sand, but its relevance to hydraulic fills or alluvial sands is questionable.
- Steady state strength is dependent only on the void ratio and not consolidation stress, while the quasi steady state strength is dependent on both consolidation stress and void ratio.
- The post liquefaction response of liquefied sand is dilative regardless of whether the sand is contractive or dilative in pre liquefaction response, if a state of zero effective stress was realised at the end of pre-liquefaction shear.
- Post liquefaction response may be characterized by three distinct regions as originally noted by Thomas (1992). The sand deforms with essentially zero stiffness during the initial phase of the post liquefaction deformation (region 1). During the second phase of deformation, the modulus increases relatively rapidly with strain (region 2). The modulus remains essentially constant during the third phase of deformation (region 3).
- Initial modulus in post liquefaction loading is several orders of magnitude smaller compared to initial modulus under virgin loading. But the modulus of liquefied sand at large strains appears essentially similar to that of the stiffness in the post phase transformation state of the monotonic test in all loading modes.
- Relative density and maximum pre-liquefaction strain level are the main factors that affect the post liquefaction response of sand. Increasing relative density and decreasing maximum pre-liquefaction strain makes the post liquefaction response

stronger. The rate at which modulus increase in region 2 is highly affected by the relative density, however, the deformation during region 1 is not significantly influenced by the relative density. The influence of relative density on post liquefaction strength decreases with an increase in maximum pre-liquefaction level.

- Increasing consolidation stress makes the post liquefaction response stronger but has less effect than relative density or maximum pre-liquefaction strain. However, it influences deformation during both region 1 and region 2.
- The normalised strength ratio of the loose Silica Sand varies from about 0.05 to 0.18, and that for the dense sand varies from about 0.3 to 0.75 depending on the maximum pre-liquefaction strain. The normalised strength ratio of the Fraser River Sand varies from about 0.1 to 0.3 at its loosest deposited state. This implies that the dilation potential has an effect on post liquefaction strength of sands.
- The post liquefaction response under triaxial compression is stronger than that under extension mode. Measured post liquefaction response in simple shear was weaker than that in triaxial. This suggests that the post liquefaction behaviour is also stress path dependent.
- The mobilised shear stress gradually increases with strain, and no tendency to approach a residual state is generally observed in post liquefaction loading even if the sand strain softens and reaches an essential residual state under virgin loading.
- The empirical data obtained from back analyses, and widely used in current practice appear to be conservative. The current practice of assigning a normalised strength to liquefied soils based only on relative density (or $N_{1,60}$) seems rather simplistic. A more rational characterization would specify contours of normalised strength

dependent on relative density and maximum cyclic strain for different levels of tolerable post liquefaction deformation.

6.3 Further Studies

More comprehensive experimental research is required to fully investigate the effects of sample preparation technique, fabric and stress path on post liquefaction behaviour of sand. Studies using different sands with different grain sizes and shapes would enable the effect of dilation potential on post liquefaction loading to be isolated. Numerical modelling techniques for estimating the deformation of soil deposits under earthquake loading should be developed based on actual post liquefaction response rather than simply relying on traditional stress-strain curves and steady state concepts.

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