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Effect of Initial Stress State on the Undrained Cyclic Behaviour of Sands

Submitted by

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of the requirements for the degree of Master of Applied Science

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Ottawa, Ontario

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ABSTRACT

A comprehensive experimental study was carried out to assess the cyclic resistance of a highly contractive sand under simple shear loading. A wide range of initial states characterized by static shear, confining stress and relative density are considered. Two series of comparison tests, one using a more dilative sand and the other using a different loading mode, were carried out to investigate the influence of contractiveness of sand and the influence of loading mode on the cyclic resistance of anisotropically consolidated sands.

The correction factors K_σ and K_α are routinely used in practice to modify the cyclic resistance value at a reference confining stress of 100 kPa and zero static shear to account for higher confining stress and static shear. It is shown that these values depend on several state parameters such as density, confining stress and static shear stress levels. Also, the value of K_α can be profoundly influenced by the contractiveness of sand and the loading mode. It has been demonstrated that sequential application of these correction factors is not proper because they are dependent on each other. Current state of practice does not take this dependency into consideration.

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LIST OF SYMBOLS

CRR	Cyclic resistance ratio, [0]
CSR	Critical stress ratio/Cyclic stress ratio, [0]
D_r	Relative density, %
D_{rc}	Relative density after consolidation, %
e_c	Void ratio after consolidation, [0]
e_{20}	Void ratio after consolidation to 20 kPa confining stress, [0]
K_c	Consolidation stress ratio, $\sigma'_{vc}/\sigma'_{hc}$, [0]
K_α	Empirical correction factor for static shear stress, [0]
K_σ	Empirical correction factor for confining stress, [0]
N	Number of loading cycles, [0]
PT	Phase transformation
QSS	Quasi steady state
r_u	Excess pore pressure ratio, [0]
SS	Steady state
Δu	Excess pore water pressure, kPa
α	Static shear stress to normal stress ratio (τ_c/σ'_{vc}) , [0]
ε_a	Axial strain, %
ϕ_{CSR}	Friction angle at critical stress ratio, degree
ϕ_{PT}	Friction angle at phase transformation, degree

γ	Shear strain, %
σ'_1	Major effective principal stress, kPa
σ'_3	Minor effective principal stress, kPa
σ'_{dcy}	Cyclic deviator stress, kPa
σ'_h	Horizontal effective stress, kPa
σ'_{nc}	Effective normal stress on 45° plane at the end of consolidation (triaxial), kPa
σ'_v	Vertical effective stress, kPa
σ'_{vc}	Effective vertical confining stress after consolidation (simple shear), kPa
τ_c	Static shear stress after consolidation (simple shear), kPa

Chapter 1

INTRODUCTION

1.1 BACKGROUND

Saturated granular materials, such as sands, silty sands and gravels, when subjected to cyclic loading will exhibit a tendency to contract (or densify), and thus will generate excess pore pressure if drainage is not permitted. Depending on their initial density and cyclic stress histories, these soils may develop pore pressures high enough to cause a near complete loss of effective stress, which in turn will imply zero shear strength. This may lead to excessive deformations generally termed liquefaction, with unlimited or limited strain potential (Seed et al, 1975). Both phenomena are particularly severe for loose cohesionless soil deposits such as those developed during conventional reclamation work (i.e., hydraulically placed fills). The reduction in strength (and stiffness) may result in permanent deformations and cause catastrophic damage. In addition, loss of bearing capacity of shallow foundations, and the floating or sinking of structures is often observed in soils liquefied by earthquakes.

Although many historic accounts of past earthquakes describe what we now refer to as liquefaction, modern study of the phenomena did not begin in earnest until after the Niigata and Alaska earthquakes of 1964. Since then, many researchers have carried out extensive laboratory tests and in-situ observations essentially on loose sand, which is clearly more prone to liquefaction. Those investigations provide us the first understanding of the triggering mechanisms of liquefaction.

Once the underlying mechanisms were understood and a basic theory was developed, the focus of liquefaction engineering research turned to the identification of liquefaction-prone sites. Analytical techniques were developed based on empirical analysis of case histories (Seed and Idriss, 1971; Seed et al., 1975). Work by Seed et al. (1985) provided a relationship between soil density (as inferred by corrected SPT blow count) and the liquefaction potential for a given level of cyclic loading. This methodology as incorporated into the Seed and Idriss (1971) “simplified procedure” has become the most widely used liquefaction analysis tool in practice.

Most of our fundamental understanding of sand liquefaction has been derived from controlled laboratory studies. The triaxial test is most widely used in industry and research because of its simplicity, and the popularity of the triaxial device. However, this kind of test cannot simulate the in-situ stress condition, which is due to a vertical propagating shear wave. The maximum cyclic shear stresses due to earthquakes are generally applied on the horizontal plane but the maximum cyclic shear stresses are applied at the 45° plane in a cyclic triaxial test. The cyclic simple shear test is considered the best laboratory tool for representing in-situ loading conditions during an earthquake.

Initial static shear stress conditions on horizontal plane of soil elements under sloping ground (or ground under a structure) are simulated in the triaxial test by using anisotropically consolidated specimens with principal stresses σ'_{1c} and σ'_{3c} . The static shear stress on the 45° plane is $(\sigma'_{1c} - \sigma'_{3c})/2$; and the stress ratio $\alpha = \frac{\sigma'_{1c} - \sigma'_{3c}}{\sigma'_{1c} + \sigma'_{3c}}$ is regarded as a measure of the initial static shear stress level. In simple shear loading mode,

α is defined as the ratio between the initial shear on the horizontal plane and the initial vertical effective confining stress.

Traditionally, the resistance to liquefaction in a cyclic triaxial test is defined as the uniform amplitude Cyclic Resistance Ratio $CRR = \frac{\sigma_{dcy}}{2\sigma'_{3c}}$ (σ_{dcy} = cyclic deviator stress) that causes a specified level of axial strain in a specified number of load cycles. Since most traditional tests were conducted under isotropic consolidation (hydrostatic), the normal stress on the 45° plane is equal to the confining stress σ'_{3c} . However, when anisotropic consolidation (with static shear) occurs, a better representation of the simple shear definition in triaxial test might be $CRR = \frac{\sigma_{dcy}}{\sigma'_{1c} + \sigma'_{3c}}$. This new definition is consistent with the definition of CRR used in simple shear, and with the definition of the static shear stress ratio in triaxial loading mode.

Laboratory studies have revealed that for a given sand, CRR is dependent on relative density, confining stress and initial static shear stress level.

In practice the cyclic loading resistance of sand is often assessed by cyclic triaxial tests on isotropically consolidated specimen ($\alpha = 0$), at generally one value of the confining stress σ' level (usually 100 kPa). The resistance at an arbitrary σ' and α is then estimated by applying factors K_σ and K_α as

$$(CRR)_{\sigma', \alpha} = (CRR)_{\sigma'=100 \text{ kPa}, \alpha=0} * K_\alpha * K_\sigma \quad (1.1)$$

where, $(CRR)_{\sigma'=100 \text{ kPa}, \alpha=0}$ represents the cyclic resistance of initially hydrostatically consolidated specimens at $\sigma' = \sigma'_{3c} = 100 \text{ kPa}$. K_σ depends on the level of σ' and a

relationship between K_σ and σ' has been proposed by Seed and Harder (1990). In their results, K_σ decreases substantially with increase in σ' , but no dependence of K_σ on relative density has been suggested. Experimental data by Vaid and Thomas (1995) using triaxial device together with its comparative simple shear studies by Vaid and Sivathayalan (1996) however suggest a strong dependence of K_σ on D_r . Furthermore, the K_σ - σ' data available in the literature is almost exclusively generated from initial hydrostatic stress conditions (i.e. $\alpha = 0$) in triaxial tests or corresponding to conditions with no initial static shear in simple shear tests. Little systematic research work has been done on the relationship between K_σ and α .

1.2 MOTIVATION OF RESEARCH

K_α has been suggested to depend on both D_r and α . The relationship proposed by Seed and Harder (1990), however, represents a large K_α range for a given α value for loose and medium dense sand. This large scatter of K_α values might be caused by the following factors: 1) different loading modes (simple shear, triaxial or others), 2) different confining stress, and 3) different characteristics response of sands. In other words, the value of K_α might be a function of various other parameters (in addition to D_r and α). The relationship suggested by Seed and Harder (1990) is for $\sigma' < 300$ kPa, and thus no dependence of K_α on σ' has been implied.

Stedman (1997) conducted a series of cyclic triaxial tests on anisotropically consolidated Fraser River sand and assessed the influence of confining stress on K_α . Based on Stedman's study, it was concluded that K_σ and K_α are not independent factors

as assumed in practice. The combined factor $K_\sigma * K_\alpha$ resulting from the empirical method will underestimate the cyclic resistance of Fraser River sand regardless of the initial density, confining stress and static shear stress levels. Fraser River sand is generally dilative (except at the loosest deposited density states) and whether Stedman's findings may be generalized to strain softening sands is unknown. Since contractive (strain softening) sands are more problematic in liquefaction design, there is a need to characterize the response of such sand and determine to what extent our current practice will over/under estimate the liquefaction resistance of this type of sands.

Systematic research on K_α and K_σ factors is limited in literature and available information is mostly obtained from triaxial tests. But the loading mode in a triaxial test does not properly simulate the in-situ stress condition. The simple shear loading mode on the other hand represents the stress conditions in-situ during an earthquake.

The main objectives of the research are:

- 1) to determine the dependence of K_σ and K_α on D_r , σ' and α under stress conditions representative of the field.
- 2) to assess influence of the contractiveness of sand on the value of K_α and evaluate the influence of loading mode on the value of K_α .

Two different sands were used in this research: Silica sand and Fraser River sand. Fraser River sand was only used in comparison tests to investigate the influence of contractiveness and to provide comparisons with earlier published results. The two sands used show quite different characteristic behaviours (strain softening or strain hardening)

under both monotonic and cyclic loading, and were used to characterize the dependence of the K_α factor on the contractiveness of sands

1.3 ORGANIZATION OF THE WORK

The focus of this thesis is the presentation of the results and insights attained during the simple shear and triaxial laboratory study undertaken. All the relevant technical information and the role of the testing within the framework of liquefaction research are also discussed. This initial chapter provides a relevant historical background and the motivation for research.

Chapter 2 presents the current understanding of liquefaction deformation behaviour, which has been developed largely based on laboratory testing. A review and discussion on both simple shear and triaxial loading modes are also presented in the chapter.

Chapter 3 discusses the experimental aspects of this research. The characteristic of the materials tested and a full discussion about the testing apparatus and testing procedures are presented. And the methodology for interpreting the cyclic testing results was discussed.

Chapter 4 provides a detailed description of the laboratory-testing results. An overview of available test data is provided and results are discussed in detail. Comparisons between test series are examined with emphasis on improving the understanding of generalized liquefaction behaviour of different types of soils under different loading modes.

Finally, Chapter 5 summarizes the findings of the testing series undertaken and

examines possible future research. All the cyclic loading test results are summarized in the appendix.

Chapter 2

LITERATURE REVIEW

2.1 LIQUEFACTION PHENOMENON

Loose saturated cohesionless soils are susceptible to excessive deformation under undrained loading conditions (either monotonic or cyclic). A soil is deemed to have liquefied when it generates considerable excess pore pressure and undergoes large deformation under undrained loading. The term liquefaction, as used in current practice, encompasses all forms of undrained deformation without regard to the mechanism of strain development.

Catastrophic damage has been caused as a result of such loading conditions in several earthquakes (e.g. 1964 Niigata and Alaska, 1995 Kobe, 1999 Chi-Chi) and the geotechnical engineering profession has paid serious attention to the liquefaction phenomenon following the 1964 Alaska and Niigata earthquakes. Since then, extensive research work has been carried out on the undrained behaviour of saturated sands and a basic understanding of the mechanism of this phenomenon has been derived. Now it is generally agreed that when sudden or dynamic loading is applied as in the case of a shock or an earthquake, a transient undrained state is induced in soil masses because of the short duration of these events. Under such loading conditions the saturated granular materials exhibit a tendency to contract (or densify), and thus will generate excess pore pressure. Increasing pore pressure implies a reduction in the effective stresses in the soil element, since the total stresses will not be changed under applied shear stresses. At the time when

the effective stress in the soil element is almost zero, a nearly complete loss of shear strength and stiffness will occur.

Under level ground, liquefaction induced damage occurs on account of the reduced bearing capacity due to the increased pore pressure. However, under a sloping ground, flow failure may develop as a consequence of static gravity stresses. The interest in the static undrained response has essentially focused on potential failure leading to flow slides. Under cyclic loading, however, interest is in both flow failure and accumulation of large shear strains (due to a progressive reduction in shear stiffness on account of the increase in pore pressure).

2.2 MONOTONIC LOADING BEHAVIOUR

2.2.1 Type of Response

Monotonic (static) undrained behaviour of saturated sands has been widely studied under triaxial loading conditions (e.g. Castro, 1969; Lee and Seed, 1980; Chern, 1985; Vaid and Thomas, 1994). Castro (1969) observed three different types of stress-strain behaviour depending upon the initial state of the soil. Very loose samples collapsed at a small shear strain level and failed rapidly with large strains (type 1). Castro called this behaviour "liquefaction" - it is also commonly referred to as "flow liquefaction" (Casagrande, 1975; Seed, 1979) and "true liquefaction" (Chern, 1985) in current practice. The soil is presumed to undergo unlimited deformation at constant stress in "steady" state. Therefore, this type of response is also known as the "Steady State" response. Medium dense soils initially showed the similar behaviour as the loose samples, but after initially exhibiting contractive behaviour and the minimum shear strength, the soil

"transformed" and began exhibiting dilative behaviour (type 2). This kind of response has been called "limited liquefaction" by Castro (1969) and Chern (1985) and "partial liquefaction" by Lee and Seed (1967). Dense specimens initially contracted but then dilated with increasing effective confining pressure and shear stress (type 3).

Which type of response will occur is primarily dependent upon the initial "state" of sand, which encompasses its relative density, confining stress, shear stress, fabric, and the loading mode. If a given sand specimen was prepared in the same way (same fabric) and was loaded from a given initial stress state, the undrained response of the sand can change from type 1 (contractive) to type 3 (dilative) with the decreasing of void ratio (increasing relative density), for a given loading mode (Fig. 2.1).

For a given sand under triaxial loading mode, contractive behaviour is observed over a larger range of void ratios in triaxial extension test than in triaxial compression test (Vaid et al., 1989). This implies that at a given initial state the loading mode (i.e., stress path) plays a major role on the response of sands. The sand at a given initial state may be dilative, and hence strain hardening in compression, but could be contractive, and hence strain softening in extension. This can be deemed as a reflection of the inherent anisotropic nature of sands (Oda, 1972).

2.2.2 Initiation of Strain Softening

The instance of peak deviator stress (or shear stress) in type 1 and 2 responses indicates the initiation of strain softening. The effective stress ratio corresponding to the peak shear stress has been termed the Critical Stress Ratio (CSR) by Vaid and Chern (1983). The value of the critical stress ratio for a given sand is dependent on the loading

mode. Chern (1985), Kuerbis (1989) and Vaid et al. (1990) conclude that CSR is unique for water pluviated sand in triaxial compression. However, Castro (1969) and Sladen et al.

Fig. 2.1 Characteristic behaviour of saturated sand under undrained monotonic loading (after Vaid and Chern, 1985)

(1985) have reported that CSR varies with the void ratio and confining stress level in triaxial compression for moist tamped specimens. This apparently is a manifestation of the effect of different specimen preparation techniques. The CSR values have been found

to be dependent upon the deposition void ratio in triaxial extension (Vaid and Thomas 1994, Stedman 1997) and simple shear (Vaid and Sivathayalan, 1996) loading modes for water pluviated sands. The CSR values in triaxial extension and simple shear are considerably smaller in comparison to results obtained from triaxial compression tests. This is an indication of the influence of loading mode on the undrained response, which arises due to the inherently anisotropic fabric of water pluviated sand.

2.2.3 Phase Transformation and Steady States

Considerable interest has been paid to the study of the true liquefaction behaviour. Most of the understanding of this phenomenon has come from monotonic triaxial compression tests on moist tamped specimens (Castro, 1969; Castro et al., 1982; Casagrande, 1975). Based on tests on moist tamped specimens, Castro (1969) concluded that the effective confining stress, shear stress and void ratio are uniquely related during steady state deformation.

The instance of maximum excess pore pressure in type 2 and 3 response is called phase transformation (PT) state (Ishihara, 1975). The response of sand up to this point is contractive and beyond is dilative. The termination of contractive behaviour is characterized by a sharp turnaround in the stress path. It has been shown that the minimum shear stress state in the limited liquefaction response may be treated within the same framework as that of steady state in the effective stress space (Been and Jefferies 1985).

The mobilized friction angle at steady state (for flow liquefaction response) is equal to that at phase transformation for a limited liquefaction response (Chern, 1985; Vaid and

Thomas, 1994). They showed that PT and Steady State might be treated within the same framework in stress space, even though their characteristics can be quite different in shear stress-void ratio (τ vs. e) space. The mobilized friction angle at PT state is found to be unique for a given sand, under a variety of initial conditions (Chern, 1985; Vaid and Thomas, 1994; Stedman, 1997). Neguessy et al. (1986) have shown that the friction angle at phase transformation is dependent only upon soil mineralogy and is equal to the constant volume friction angle under drained conditions.

2.2.4 Ultimate failure envelope

On further straining beyond the PT state, the effective stress path rapidly approaches the line of maximum obliquity and further deformation progresses at a constant friction angle until a very large strain was reached, at which the stress path is postulated to return to the PT/SS line. The angle of maximum obliquity has also been found to be unique for a given water deposited sand (Vaid and Chern, 1985; Vaid and Thomas, 1994), regardless of relative density, confining stress level, loading mode, sample preparation technique or gradation (Kuerbis, 1989). Miura and Toki (1992), however, reported that this angle varies somewhat with relative density and loading mode, but the changes recorded were minor.

2.3 CYCLIC LOADING BEHAVIOUR

2.3.1 Factors of influence

The undrained response of saturated sand under cyclic loading depends on a variety of factors: relative density, confining stress level, static shear stress level and cyclic shear

stress level being the most important. Details of the influence of these factors will be discussed below.

2.3.2 Mechanisms of Strain Development

Strain development during cyclic loading can either be due to true liquefaction, limited liquefaction or cyclic mobility – type 1, type 2 or type 3 response shown in Figures 2.2 a), b), & c) respectively (Vaid and Chern, 1985). The different types of cyclic responses do generally correspond to the type 1, 2 or 3 responses noted under monotonic loading. The strain development mechanism (i.e. either type 1, 2 or 3) depends primarily on the density and to a lesser extent on the characteristics of the initial stress states. Because of the different mechanisms that are responsible for strain development, the results of cyclic loading are generally assessed in terms of a fixed level of strain developed in a given number of cycles. The initial development of strain is associated with stiffness degradation with cycles of loading. Seed (1979) found that strain development is small until the excess pore pressure ratio, $\Delta u/\sigma'_{3c}$ exceeds about 60%. In contractive sands (true or limited liquefaction type of response) however, large strains may develop without very high excess pore pressure. A transient state of zero effective stress is normally a pre-requisite for large strain development due to cyclic mobility. Such a state of zero effective stress, however, will occur only if the cyclic stresses undergo stress reversal.

Fig. 2.2 a) True liquefaction due to cyclic loading (after Vaid and Chern, 1985)

Fig. 2.2 b) Limited liquefaction due to cyclic loading (after Vaid and Chern, 1985)

Fig. 2.2 c) Cyclic mobility due to cyclic loading (after Vaid and Chern, 1985)

Castro (1969) and Vaid and Chern (1985) have demonstrated that true liquefaction and limited liquefaction type of response develop in cyclic loading in the same manner as in monotonic loading. Vaid and Chern (1985) have also shown that contractive deformation in cyclic loading is triggered at the critical value of stress ratio, CSR observed under monotonic response. Upon the triggering of contractive deformation, unlimited strains occur in true liquefaction type of behaviour at finite stress levels. In limited liquefaction however, the strain development is associated with strain softening until the PT state and the stresses increase due to dilation, beyond the PT state. Subsequent unloading cycle generally brings the sand to a state of zero effective stress ($\sigma'_{3c} = 0$). Any further cycles of loading will take the sand through transient states of zero effective stress that cause strain accumulation.

No strain softening deformation occurs in cyclic mobility type of response in dense sands. Vaid and Chern (1985) observed that strain development is small until the stress path reaches the PT line. Subsequent to PT state, the unloading pulse cause a large increase in excess pore pressure and take the sample to a very low effective stresses. The excursions of the stress path through transient states of zero confining stress with increasing load cycles cause progressive stiffness degradation. Additional load cycles beyond the PT state are generally responsible for strain accumulation.

2.3.3 Effect of Confining Stress

Earlier studies on the effect of confining stress were first undertaken under monotonic loading conditions. Castro (1969) recognized that Banding sand was more prone to true liquefaction failure under static loading as the confining stress increased.

Vaid and Thomas (1994) demonstrated that at a given density, increasing confining stress in triaxial compression loading promotes more contractive behaviour. However, increasing confining stress in triaxial extension loading was found to promote less contractive behaviour. The effect of confining stress was found to be less significant for the loosest deposited specimens (Vaid and Thomas, 1994).

Monotonic test results and a limited number of cyclic tests indicated that increased confining pressure would decrease the cyclic resistance of a saturated sand. However, the extent of this effect on cyclic loading is not well understood. Seed and Harder (1990) suggested the use of a correction factor K_σ , that is defined as the ratio between the cyclic stress ratio required to cause liquefaction at the given confining stress to that required to cause liquefaction at a reference stress state of $\sigma'_{vc} = 100$ kPa (or 1 tsf). The factor K_σ is always less than one for $\sigma'_{vc} > 100$ kPa. The data presented by Seed and Harder (1990) show a large scatter in K_σ for a given confining stress (Fig. 2.3). This apparently indicates the dependency of K_σ on factors other than the confining stress. Vaid and Thomas (1994) showed that K_σ depends, in addition to the confining stress, on the relative density of the sand. Thomas (1994) reported that the value of K_σ is smaller in dense sand, than in most liquefaction prone loose sands. Comparative cyclic simple shear studies (Vaid and Sivathayalan, 1996) show that cyclic triaxial tests tend to overestimate K_σ at higher densities.

Fig. 2.3 Proposed relationship between vertical effective stress (σ'_0) and K_σ (after Seed and Harder, 1990)

In 1997 and 1998, National Center for Earthquake Engineering Research (NCEER) led two workshops on Evaluation of Liquefaction Resistance of Soils. In these workshops participants gained a consensus that the Seed and Harder (1990) K_σ values were too conservative and that an increase is recommended for general engineering practice. Based on the test results of different researchers and also taking the influence of relative density into consideration, they recommended K_σ values represented by the curve and equation in Fig. 2.4 for engineering practice.

Fig. 2.4 Recommended curves for estimating K_σ for engineering practice (after Youd and Idriss, 2001)

It is evident from the existing laboratory data that K_σ is dependent on relative density and loading mode, in addition to the level of confining stress. The influence of the presence of initial static shear (anisotropic initial condition) on K_σ however, is generally missing from previous studies. Stedman (1997) tried to investigate this influence using triaxial testing equipment; however no clear conclusion on this issue could be drawn from the presented data.

2.3.4 Effect of Initial Static Shear

Cyclic resistance of saturated sand deposits are commonly evaluated from cyclic triaxial tests on isotropically consolidated sand samples. Lee and Seed (1967) pioneered cyclic triaxial testing of anisotropically consolidated specimens. Studies by Lee and Seed (1967), Lee, et al. (1975), Seed, et al. (1975) and Seed (1983) conclude that the presence of static shear could increase the cyclic resistance to liquefaction. Castro (1969, 1975), Casagrande (1976), Castro and Poulos (1977) and Castro, et al. (1982) have also performed cyclic triaxial tests on anisotropically consolidated saturated sands. They however, reached a conclusion that an increase in the static shear stress may decrease the cyclic resistance of sand to liquefaction. Studies by Vaid and Chern (1983) show that the effect of initial static shear on cyclic resistance depends on the relative density of the specimen, the magnitude of the initial static shear and the strain criterion used to define liquefaction.

Vaid and Finn (1978) and Vaid and Chern (1983, 1985) showed that without reference to the relative density, the dependence of cyclic resistance on static shear was not rational. This led to a conclusion that the cyclic resistance of a given sand can either increase or decrease depending on the density and the static bias, and is closely linked to the mechanism of strain development during cyclic loading.

Based on experimental information available at the time Seed and Harder (1990) incorporated the static bias effects into a convenient factor K_α (Fig. 2.5). K_α represents the ratio of the cyclic resistance under a static shear level of α to that with no static shear.

Static shear level α has been defined as the ratio between initial shear and normal effective stress as shown in Fig. 2.6

Thus the current design approach estimates cyclic resistance at arbitrary σ'_{vc} and α levels from the reference cyclic resistance at $\sigma'_{vc} = 100$ kPa and $\alpha = 0$ by the relation:

$$(CRR)_{\sigma', \alpha} = (CRR)_{\sigma' = 100 \text{ kPa}, \alpha = 0} * K_{\alpha} * K_{\sigma} \quad (2.1)$$

The K_{σ} and K_{α} factors are treated as separable independent correction factors in current practice, despite some experimental evidence to the contrary. A clear understanding of the combined effect of confining stress and static shear would thus complement our present knowledge on the cyclic response of saturated sand, which is affected by the collective initial state. Such an understanding is essential in order to address any concerns regarding the under or over estimation that may be present in the current empirical method $(CRR)_{\sigma', \alpha}$ using factors K_{σ} and K_{α} .

Fig. 2.5 Seed and Harder (1990) correction factor K_α

$$\alpha = \left| \frac{\tau_{hv}}{\sigma'_o} \right|$$

Fig. 2.6 Seed and Harder (1990) definition of α

2.4 REVIEW OF SIMPLE SHEAR TESTING EQUIPMENT

Cyclic simple shear testing equipment has been used for several decades and various designs have been proposed by different investigators over the time; each with the objective of achieving a specific goal (Roscoe, 1953; Bjerrum and Landva, 1966; Franke et al., 1979; Ishihara and Yamazaki, 1980; Boulanger et al., 1993). The most common method of simple shear testing involves imposing normal and shear stresses through end caps to a short cylindrical (NGI style) sample consolidated under anisotropic loading. There are three major benefits of simple shear testing. The primary benefit is the similarity of forces imposed on the sample to those experienced by the in-situ material, shown schematically in Fig. 2.7. It allows a smooth and continuous rotation through 90° of the principal stress directions, which is comparable to what occurs to a soil element in-situ when a vertically propagating SH wave passes by it. However, it is recognized that no laboratory equipment is capable of fully representing the in-situ stress state and deformation conditions of actual soil deposits, and the shortcomings of the simple shear device are discussed below.

The most significant problem with simple shear testing is boundary conditions that create non-uniformities in the stresses and strains across the sample. Non-uniform stress conditions may develop due to the lack of complimentary shear stresses on the lateral boundaries, radial expansion of the specimen, and pinching effects that may result from relative base/cap rocking motions. Previous researchers have investigated the effects of these factors on the uniformity of stresses and strains within simple shear test specimens and their findings are discussed in this section. As shown in Fig. 2.8, while shear stresses

Stresses under level ground with idealized seismic loading

Rotation of principal stress directions in situ

Fig. 2.7 Idealized in-situ conditions for soil elements under level ground experiencing seismic loading

Fig. 2.8 Comparison of state of shear between simple shear and pure shear

in pure shear develop at the vertical and lateral boundaries, the simple shear apparatus is incapable of producing complimentary shear stresses on the lateral boundaries. The absence of complimentary shear stresses produce a non-uniform stress distribution near the lateral boundaries, and the degree of error is dependent on the aspect ratio of the specimen.

De Alba et al. (1976) investigated the effect of stress non-uniformities by comparing results from large scale shaking table tests and smaller scale simple shear tests. The shaking table tests were performed on a large bed of sand with dimensions of 2300 mm x 1100 mm x 100 mm to minimize the portion of the sample that is affected by the

lack of complementary shear stresses. All tests were performed on saturated Monterey No. 0 sand under undrained conditions. Specimens were prepared by pluvial deposition of dry sand using a gantry mounted spreader system, which could be controlled to produce uniform beds of sand at different relative densities. Cyclic simple shear stresses were applied to the specimens to produce a sinusoidal motion at a frequency of 4 Hz. Fig. 2.9 shows the cyclic stress ratio versus the number of cycles to achieve liquefaction in these shaking table tests for $D_r = 50\%$. Similar results from cyclic simple shear testing of specimens with diameter to height (D/H) ratios of 3 to 4 are presented on the same plot and shown to be in good agreement with the shaking table results. The consistent soil response from these two specimen configurations with significantly different aspect ratios indicates that the errors due to the stress concentrations in small-scale simple shear tests may not be large.

Additional testing programs investigating the effects of aspect ratio include work by Franke et al. (1979) and Vucetic and Lacasse (1982). In the Franke et al. work, undrained cyclic simple shear tests were performed using the BAW apparatus on a saturated sand specimens with D/H ratios of 3.75 and 7.5. The shear moduli for specimens with the two aspect ratios were found to be similar. Vucetic and Lacasse (1982) performed 21 strain-controlled, constant-volume, monotonic-loading tests on undisturbed specimens of medium-stiff clay using an NGI-type simple shear device. Specimen diameters were varied while the height was kept constant to achieve D/H ratios of 3, 5 and 7. This testing enabled the effects of aspect ratio on the static shear strength and modulus to be investigated. As shown in Fig. 2.10, the test results show that the aspect ratios between 3 to 7 have no significant effect on the monotonic stress-strain behaviour of the tested soil.

Fig. 2.9 Cyclic stress ratio versus the number of cycles to achieve liquefaction in shaking table tests for $D_r = 50\%$ (after De Alba et al., 1976)

Another potential source of stress non-uniformities in simple shear testing is non-uniform specimen deformation during the application of a static vertical stress. Both lateral and vertical deformations during vertical loading may occur if the lateral boundary condition does not effectively limit radial expansion of the specimen. These lateral deformations will be distributed non-uniformly as a result of the frictional radial confining effects of the upper and lower specimen caps. The use of steel wire-reinforced membranes provides a lateral boundary condition that can essentially approximate one-dimensional specimen deformation by restraining the specimen against radial expansion while providing negligible shear stiffness. To sum up, as shown by De Alba et al. (1976),

Fig. 2.10 Effect of H/D ratio on stress-strain behaviour; test nos. 1.7, 1.9 & 1.10 have $H/D = 0.2$ and nos. 1.6 & 1.8 have $H/D = 0.32$ (after Vucetic and Lacasse, 1982)

Franke et al. (1979) and Vucetic and Lacasse (1982), for most practical engineering purposes, simple shear testing is able to reasonably approximate in situ deformations of soils subject to vertical propagating shear waves.

Simple shear tests for liquefaction studies have been carried out using wire-reinforced membrane by several researchers (Finn and Vaid, 1977; Finn et al., 1978; Finn, 1985; Kammerer, 2002). A vertical load is applied while the wire-reinforced membrane enforces an essential K_0 condition. The lateral stresses develop as needed to maintain zero lateral strain. During cyclic loading of a specimen, no effort is required to ensure saturation. Instead, the specimen height is kept constant during testing, which implies maintaining constant volume by virtue of the reinforced membrane. As the pore pressure in the sample is always atmospheric, the development of excess pore pressure is actually zero. The drop in vertical stress in such a test is equal to the excess pore pressure that would have developed in a fully saturated undrained test. The total loss of vertical (and consequently, horizontal) effective stress implies full liquefaction in dry sands.

Clearly there is a difference between the stress conditions that exist during testing on a dry sample and the fully saturated in-situ conditions it is intended to replicate. When the vertical load drops off in a dry sample, the horizontal load drops as well, but the drop is some lesser value than occurs in the vertical directions. The ratio of the two stress changes is dependent on the " K_0 , OCR " properties of the soil. This is in contrast to an increase in pore pressure, which produces an equal drop in effective stress in both the vertical and horizontal directions. If two samples, one dry and one saturated, produce the same calculated pore pressures, the dry sample will actually be confined by a larger lateral effective stress. However, the advantages of cyclic simple shear testing heavily outweigh the above note limitations. First, this method can prevent compliance issues by having no compression force on water. Secondly, it satisfies the in-situ condition of both constant vertical load and height. Finally, it simulates the stress conditions of the soil

element in-situ fairly well; at least much better than any other routine soil-testing device.

2.5 REVIEW OF TRIAXIAL TESTING EQUIPMENT

Liquefaction laboratory testing was first performed by Seed and Lee (Lee and Seed, 1967; Seed and Lee, 1967) on triaxial equipment. For the next two decades, a large majority of testing was performed on this type of equipment, primarily because the equipment is widely available and the testing procedure is fairly straightforward. Triaxial testing is performed on a tall cylindrical soil sample contained by cell pressure applied through a non-reinforced latex membrane. During testing a vertical load is typically cycled around the initial isotropic consolidation pressure. Early analysis suggested that triaxial testing could be useful for assessing trends in laboratory liquefaction data. Nevertheless, care should be exercised when using this data to draw specific conclusions about overall behaviour, as the applied stress path is not representative of vertically propagating shear waves in an in-situ soil profile.

The main problem associated with the triaxial stress path can best be understood by considering the Mohr's circle analysis, as shown in Fig. 2.11, for both isotropic and anisotropic cases. It can be seen that in a triaxial test, the direction of the principal stresses instantaneously jumps 90 degrees. This differs from the smooth rotation of principal stresses that occurs in the field under seismic loading and in simple shear and torsional laboratory testing (as shown in Fig. 2.7). This is particularly important as stress reversal has a strong effect on pore pressure generation and accumulation of permanent deformations.

Stresses during triaxial tests consolidated (1) isotropically and (2) anisotropically

Rotation of principal axes in triaxial testing

Fig. 2.11 Idealized stresses during triaxial testing (after Harder, 1988)

In addition, the boundary conditions are not representative of the simple 1-dimensional seismic loading (i.e., no lateral deformation). As a result of the geometry of the sample and loading configuration (which causes a failure surface at $45+\phi/2$ degrees to horizontal), only the conditions of constant volume and full saturation are potentially well maintained. However, the sample may only maintain constant volume globally, not necessarily locally. Seed and Peacock (1971) observed the cyclic resistance in the field (τ_{hv}/σ'_o) is about 60% of the value of $\sigma_{dcy}/\sigma'_{3c}$ causing liquefaction under triaxial test conditions. Therefore, based on both analytical and experimental evidence, as well as available field data at the time they suggested most laboratory compression test data should be reduced by a factor of 0.55 to 0.7, depending on the density of the soil and the duration of the earthquake, in order to determine values of τ_{hv}/σ'_o causing liquefaction under field conditions.

Chapter 3

EXPERIMENTAL WORK

3.1 SIMPLE SHEAR TESTING

The first part of this research was aimed to investigate the dependence of correction factors K_σ and K_α on the relative density index D_r , confining stress σ'_{vc} and initial shear stress level α using the simple shear device. Details of the testing program are discussed below.

3.1.1 Material Tested

The specimens studied in this research were reconstituted using two different sands, Silica sand and Fraser River sand. These sands have distinctly different mineralogy and particle characteristics. The Silica sand used is poorly graded with sub-rounded quartz particles. The specific gravity determined following ASTM D854 is 2.66. The batch of sand used has an average particle size $D_{50} = 0.42$ mm, uniformity coefficient, $C_u = 1.63$, and the coefficient of curvature, $C_c = 0.93$. The particle size distribution for ASTM graded sand determined by U.S. standard sieve analysis, ASTM D422, is shown in Fig. 3.1. The maximum and minimum index void ratios are $e_{max} = 0.723$ and $e_{min} = 0.478$, respectively, according to ASTM standards D4252 and D4253. The Silica sand used conforms to ASTM C109 specifications.

Fraser River sand derived from Fraser River delta, B.C. was used in a limited number of comparison tests. The sand was dry sieved through a 1.00 mm sieve and the

material retained was discarded. The remaining sand was then washed through water to get rid of very fine particles. The fraction coarser than 1.00 mm was negligibly small and the fraction finer than 0.10 mm is about 7%. The maximum and minimum index void ratios are $e_{\max} = 0.806$ and $e_{\min} = 0.509$, respectively, according to ASTM standards D4252 and D4253. The grain size distribution curve is shown in Fig. 3.2. ($D_{50} = 0.28$ mm, $C_u = 2.67$ and $C_c = 1.26$)

Fig. 3.1 Particle size distribution of Silica sand

Fig. 3.2 Particle size distribution of Fraser River sand

3.1.2 Testing Apparatus

“ASK Simple Shear Tester”, Bench Model (Model No. DTC-148) was used in this research. A schematic diagram of the testing apparatus is shown in Fig. 3.3. The application of shear stress at the consolidation stage, and later at the cyclic loading stage was realized by changing the air pressure in one of the pressure chambers of a double acting piston. Vertical pressure was applied by increasing the air pressure in the vertical loading piston. An E/P transducer was connected with one horizontal pressure chamber to control its pressure numerically through a personal computer. All forces and

displacements were measured using electronic transducers interfaced to a personal computer via a signal conditioner. All stresses were measured with resolution of better than 0.2 kPa and strains with resolution of better than 10^{-4} .

In order to maintain the initial K_0 state and plane strain conditions, wire-reinforced latex membranes were used. All simple shear tests were carried out on dry pluviated sand specimens.

3.1.3 Specimen Preparation

All samples for simple shear testing were prepared by air pluviation directly into the membrane-lined cavity created by the forming split mold. During the deposition, void ratio was controlled through control of the falling height of sand particles. Before placing the top cap, the top surface of the sample was leveled with the aid of a siphon to ensure minimal disturbance to the fabric. Samples that required a slight increase in density were subjected to external vertical vibration after placing the top cap that provides a small seating load. After placement of the top cap and densifications if required, the latex membrane was carefully rolled up onto the cap and sealed by an O-ring around the perimeter of the cap. The sample was then subjected to a vacuum of about 20 kPa in order to develop some effective stress. This permitted the removal of the mold without collapsing the sample. The void ratio of the sample was determined after exposure to vacuum and recorded as e_{20} for all samples.

In order to minimize the influence of the lack of imposed complementary shear stresses, a relative large value of diameter to height (D/H) of the specimen should be used. In this study, simple shear test specimen of around 71mm diameter and 20 mm height

Figure 3.3 Schematic diagram of cyclic and monotonic loading system for simple shear test

($D/H = 3.55$) was reconstituted. The dry pluviation technique yields very uniform and repeatable specimens. (De Alba et al., 1976)

3.1.4 Consolidation

The consolidation for those samples without initial shear was done in steps (usually 4 increments) by increasing the vertical stress. The consolidation for those samples with initial shear was also done in steps: first increase vertical stress and then shear stress to maintain a constant α value. The stress path for consolidation stage of those samples with initial shear was shown in Fig. 3.4.

Fig. 3.4 Consolidation stress path for samples with initial static shear (simple shear)

3.1.5 Cyclic Loading

Cyclic shear stresses were applied by changing the pressure in the right chamber of the double acting piston in Fig. 3.3. In the normal configuration, the piston is freely floating with balanced forces on both sides. The pressure in the right chamber is controlled by an electro-pneumatic transducer. Changes in shear stresses may be made by variations in the signal voltage supplied by the signal conditioner to the electro pneumatic transducer. The determination of the output cyclic stress level required for each sample was based on the geometry of the horizontal piston, the sample area and the targeted cyclic shear stress amplitude. The electronic signals to the electro pneumatic consisted of a sine wavevariation in the voltage, and hence load, with the first quarter pulse in the same direction of static shear (if static shear exists). The frequency of the cyclic loading pulse was 0.1 Hz (Period=10 sec); this was used to achieve a reasonable degree of resolution from the instruments. The frequency of loading was not a concern because it had been shown to have little influence on the undrained response of sand (Yoshimi and Oh-Oka, 1975). The data was recorded at a rate of 64 points per cycle.

3.1.6 Liquefaction Criterion

The development of 3.75% shear strain in a single direction was taken as the definition of liquefaction in cyclic loading tests (NRC, 1985). This criterion generally corresponds to an essential pore pressure ratio $r_u = 100\%$ for contractive sands with no static shear (Ishihara, 1985). In the case of more dilative sands, the development of 3.75% shear strain may correspond to the states where only limited pore pressure has developed.

3.1.7 Testing Program

3.1.7.1 Monotonic Loading Response

Loosest deposited samples were tested undrained monotonically in simple shear testing at all selected initial stress conditions used in the cyclic testing program. This information characterized the material properties of the sand, in particular the type of response – contractive or dilative- at each initial condition and permitted the relationship between monotonic and cyclic behavior to be examined.

3.1.7.2 Cyclic Loading Response

Cyclic tests were performed on initial states both with and without static shear. Samples were prepared at various density states at a given initial confining pressure and α value. A series of such specimens were then loaded with fixed cyclic stress amplitude. Such data enable the construction of a set of curves showing D_{rc} (or void ratio, e_c) as a function of the number of cycles to liquefaction, at several constant CSR levels. This in turn permitted the development of the Cyclic Resistance Ratio curve, showing CSR as a function of D_r (or void ratio, e) for liquefaction in a fixed number (10 stress cycles was used in this study) (Fig. 3.5).

Similar data was generated at identical confining stresses but at other selected values of α . The information thus obtained at one confining stress was then developed at other vertical confining stress levels. This experimental data was then used to develop a set of K_α versus α and K_σ versus σ'_{vc} relations over a range of initial confining stresses and α values for a range of initial relative density states. Table 3.1 summarizes the

Fig. 3.5 Schematic diagram of Void Ratio vs. Number of Cycles and CSR vs. D_r showing cyclic resistance curve development at one σ'_{nc} and α

parameters used in the cyclic simple shear tests.

Table 3.1 Parameters used in cyclic simple shear tests

SIMPLE SHEAR TESTS		
Sand used	Silica sand	Fraser River sand
Relative density	Loose to dense	Loose to dense
σ'_{vc} (kPa)	100, 200 and 400	100
α	0.00, 0.06, 0.10 and 0.20	0.00, 0.10, 0.20 and 0.30

3.1.7.3 Repeatability

Due to the limited supply of material and the large number of tests (over 300) required to complete this work, the material used in tests at lower confining stresses was reused. Particle breakdown and other adverse affects on the material caused by the induced stresses affecting the results of the tests were addressed by performing monotonic loading tests on virgin and reused sand. As can clearly be seen in Figs. 3.6 there is no detectable difference in either shear stress or pore pressure response, and hence little effect on the material after one use.

The repeatability of the cyclic loading tests was also assessed by performing several tests, with identical initial condition and identical cyclic stress ratio. Figure 3.7 demonstrated excellent repeatability under cyclic loading as well.

Fig. 3.6 Monotonic simple shear response of used and virgin Silica sand at vertical confining stress $\sigma'_{vc} = 100$ kPa

Fig. 3.7 Repeatability of cyclic response of sand with no static shear at $\sigma'_{vc} = 100$ kPa

3.2 TRIAXIAL TESTING

3.2.1 Material Tested

In order to clarify the influence of loading mode on the K_α value, triaxial tests were conducted as comparison tests in this research. Tests were carried out on Silica sand whose properties and gradation are noted in section 3.1.1, which was extensively tested under simple shear.

3.2.2 Testing Apparatus

The triaxial testing device used in this study is capable of conducting both stress controlled and strain-controlled tests. A schematic diagram of the testing apparatus is shown in Fig. 3.8. All triaxial specimens had a nominal diameter of 63 mm and a nominal length of 127 mm. End restraint is minimized by using polished cap and pedestal made of anodized aluminum and with a central 21 mm drainage stone. Like the simple shear testing device, all pressures, forces, and displacements and volume changes were measured using electronic transducers interfaced to a computer via a signal conditioner. The resolution of all pressure transducers is better than 0.01 kPa, vertical load cell better than 0.01 kg, and axial strain better than 10^{-5} .

The triaxial cells used for this program were standard cells and consisted of solid bases and a low friction (less than 0.1N) air bleed bushing seal for the loading ram. The reduction of the friction to a negligible amount is achieved by balancing the cell pressure against an equal pressure in the clearance between the ram and the bushing. This permits accurate measurement of axial forces outside of the cell.

Fig. 3.8 Schematic diagram of the triaxial testing apparatus (modified from Stedman, 1997)

The physical continuity of the loading ram from the sample in series through the vertical loading piston permits the strain controlled monotonic loading or post cyclic monotonic loading following stress controlled cyclic loading without undue disturbance to the specimen. The piston also permits compressive force to be applied to the ram during consolidation to compensate the loss of vertical pressure because of the vertical ram during hydrostatic consolidation or to obtain a desired deviator stress during anisotropic consolidation.

3.2.3 Specimen Preparation

All samples for triaxial testing were prepared by water pluviation. This technique has been well established by several researchers (e.g. Vaid and Negussey, 1988) to yield repeatable uniform test samples. The ensuing fabric has been shown to duplicate the fabric of natural alluvial sands, and hydraulic fills (Oda, 1972). The method of depositing the sand in place through water also ensures complete elimination of air and hence yields saturated samples. Consistent loosest void ratio can be achieved by a careful elimination of vibration during sample preparation. Denser states can be easily achieved by applying vertical vibration by tapping the cell base during and after pluviation of the sand. Water pluviation techniques for reconstituting homogenous specimens are restricted to poorly graded sands. Particle segregation will occur with this technique when applied to well graded sands. Since the Silica sand is poorly graded, water pluviation was considered to be a very suitable technique for this research.

The sand was first boiled, cooled to room temperature and kept under vacuum, and then pluviated through de-aired water into the membrane lined mold cavity. Consistent

deposition density was achieved by moving the tip of the flask in a circular or spiral manner while allowing the sand to rain out of the flask into the mold cavity. A 60 mm by 150 mm membrane, 0.3 mm thickness was employed. During deposition, the tip of the flask was kept submerged until all of the sand had passed out of the flask. The top of the sample was then leveled with the aid of a siphon to ensure minimal disturbance to the fabric. Samples that required increase in density were subjected to external vertical vibration after placing the top cap that provide a small seating load.

The internal mean area of the mold cavity was determined prior to the start of the testing program and checked from time to time by the used of a differential pressure transducer and the mass of the water expelled from the cavity. This method removes the potential error of circumferential measurement and minimizes errors in void ratio calculations (Vaid and Sivathayalan, 1996). Determination of the initial loosest deposition was made in all cases as soon as the top cap was placed. Loosest void ratios were close to the ASTM $e_{\max}$ as previously stated. After placement of the top cap and densification if required, the latex membrane was carefully rolled up onto the cap and sealed by an O-ring around the perimeter of the cap. The samples were then subjected to a vacuum of about 20 kPa in order to provide confinement during setup. During the exposure to vacuum, the volume of water expelled from the sample was recorded by permitting it flow only into a graduated burette. This coupled with measurements of the height of the samples (measured using a reference dial indicator) and the mean area of the sample section permitted an accurate determination of the sample volume at any stage in the preparation and testing phase. The void ratio of the sample was determined after exposure to vacuum and recorded as e_{20} for all samples. The act of pluviating the sand

from the flask directly into the water in the mold cavity resulted in consistently high B values, which indicated high degree of saturation.

After the sample was prepared and the cell assembled, the cyclic load pulse was calibrated based on the area of the sample at this point for cyclic tests. The cell was then moved to the loading frame and securely clamped in place under the loading ram. Application of a slight cell pressure to overcome the previous applied vacuum permitted a small bead of water to exit the drainage line to expel any air. The degree of saturation (B value) determination was accomplished by increasing the cell pressure gradually and measuring the pore pressure response under undrained conditions. All the B values obtained before consolidation are higher than 0.99 in this research. As soon as a suitable B value had been achieved the sample was then consolidated incrementally either isotropically or anisotropically.

3.2.4 Consolidation

Since the entire triaxial specimen must be fully saturated, samples were consolidated to the target confining stress state (hydrostatic or anisotropic) after B value measurements. The consolidation procedures consist of: 1) setting the back pressure to its post saturation state, 2) opening the drainage valve, 3) bring the sample to the required ratio K_c by increasing the vertical stress and then moving smoothly either up a constant K_c line, or horizontally ($K_c = 1$) along the mean normal stress axis until the targeted mean effective stress σ'_{nc} was reached by altering the cell pressure (Fig 3.9). Drainage was permitted to continue under the final stress state, until no volume change was noticed. This was further confirmed by temporarily closing the drainage line and briefly

monitoring the pore pressure for any increase above the pre-set back pressure.

Fig. 3.9 Stress path for samples with isotropic or anisotropic consolidation (triaxial)

3.2.5 Cyclic Loading

Cyclic shear stresses were applied by changing the pressure in the top chamber of the double acting piston in Fig. 3.4. In normal configuration, the piston is freely floating with balanced forces on both sides. The pressure in the top chamber is controlled by an electro pneumatic transducer. Changes in deviator stresses may be effected by variations in the signal voltage supplied by the signal conditioner to the electro pneumatic transducer. The determination of the output cyclic stress level required for each sample

was based on the geometry of the vertical piston, the sample area and the targeted cyclic shear stress amplitude. The electronic signals to the electro pneumatic transducer consisted of a sine wave variation in the voltage, and hence load, with the first quarter pulse in the triaxial compression mode. This applied cyclic stress was superimposed on top of the vertical stress already applied either to overcome the hydrostatic ram lift or the vertical stress needed to achieve an anisotropic consolidation. The frequency of the cyclic loading pulse was 0.1 Hz; this was used to achieve a reasonable degree resolution from the instruments. The data was recorded at a rate of 64 points per cycle.

3.2.6 Liquefaction Criterion

The development of 2.5% axial strain in a single direction was taken as the definition of liquefaction in triaxial cyclic loading tests. This value is equivalent to 3.75% shear strain in simple shear tests. (NRC, 1985)

3.2.7 Testing Program

Only cyclic tests were performed in triaxial testing program. These tests were performed on both isotropically and anisotropically consolidated samples. Samples were prepared at controlled densities (loosest and medium dense states) under a controlled initial normal confining pressure of 100 kPa and constant K_c values. A series of such specimens were then loaded with different cyclic stress amplitude. Such data enable the construction of a set of curves showing CSR as a function of the number of cycles to liquefaction, at several constant K_c levels. This in turn permitted the development of a set of curves for each set of conditions (constant D_r and constant σ'_{nc}) showing CSR as a function of K_c for liquefaction in a fixed number, say, ten stress cycles (Fig. 3.10). By

using the proposed relationship between K_c and α , this data was then used to develop a set of K_α versus α relations under one controlled normal confining stress (100 kPa). Only two initial relative density states (loosest and medium dense) were considered in this comparison series. Table 3.2 summarizes the parameters used in the triaxial tests.

Table 3.2 Parameters used in triaxial tests

TRIAXIAL TESTS	
Sand used	Silica sand
Relative density	Loosest and medium dense (15-20%, 45-50%)
σ'_{nc} (kPa)	100
K_c	1.00, 1.25 and 1.50

Fig. 3.10 CSR vs. Number of Cycles to liquefaction

Chapter 4

TEST RESULTS AND INTERPRETATIONS

4.1 OVERVIEW

The undrained behaviour of Silica sand under static and cyclic simple shear loading is presented in this chapter. In addition, the undrained behaviours of: 1) Fraser River sand in simple shear and; 2) Silica sand in cyclic triaxial tests are also presented for comparison. The tests conducted encompass a range of initial states represented by the state variables e_c (or relative density D_{rc}), confining pressure σ'_{vc} (or σ'_{3c} in triaxial tests) and static shear stress ratio α (or K_c in triaxial tests). Monotonic loading simple shear tests were conducted on Silica sand in the loosest deposited state. This was done with the objective of delineating the initial states that may be susceptible to contractive deformation. Cyclic tests involving both stress reversal and non-reversal were conducted in order to study the characteristics of strain development under different mechanisms, which cause liquefaction, namely (a) Contractive deformation with excursion through states of transient $r_u = 1$; (b) Cyclic mobility with excursion through states of transient $r_u = 1$; (c) Cyclic mobility without stress reversal and hence no transient $r_u = 1$.

Comparison between the cyclic and static behaviours is explored. The cyclic resistance of the sand is characterized and its dependency on the initial state variables is

identified. In particular, the effect of the static shear and confining stress on liquefaction resistance is examined. The influence of static shear on the confining stress correction factor K_σ is highlighted.

Comparison between test results on different types of sand (Silica and Fraser River) was carried out and the influence of the contractiveness of sand on the K_α value is discussed. Similarly, tests results from triaxial testing were used to identify the influence of loading mode on the K_α value. Test results from Fraser River sand were also compared with data in the literature to provide more insight into the influence of loading mode.

4.2 PARAMETER DEFINITIONS

Numerous terms and parameters have previously been developed to describe both the state of a sample and the loading applied. Whenever possible, commonly used definitions are applied in this discussion. Among the traditional parameters used are initial normal effective confining stress (σ'_{nc}), initial static shear stress ratio for simple shear (α), anisotropic consolidation ratio K_c (triaxial), normalized excess pore pressure (r_u), cyclic stress ratio (CSR), axial strain (ϵ_a), and shear strain (γ). Cyclic resistance ratio (CRR) is defined as the cyclic stress ratio (CSR) required for developing 3.75% shear strain in 10 cycles in simple shear testing. While in triaxial testing, CRR is defined as the cyclic stress ratio (CSR) required for developing 2.5% axial strain in 10 cycles. All parameters used are detailed in the list of symbols at the beginning of the thesis. The

relative density is defined as $D_r = \frac{e_{\max} - e_c}{e_{\max} - e_{\min}}$.

4.3 ACCESSIBLE DENSITY STATES

Fig. 4.1 shows the compressibility characteristics of Silica sand in the form of e - $\log(\sigma'_{vc})$ relationships. Compressibility is shown at three different initial deposition void ratios (loose to dense). The loosest state e_{20} correspond to the void ratio following the application of a confinement of approximately 20 kPa after the top cap was put on following deposition. It may be noted that the loosest density states that are accessible depend on the normal effective confining stress level. The loosest density states corresponding to $\sigma'_{vc} = 100, 200$ and 400 kPa are 25%, 30% and 35% respectively. It seems that at a certain vertical stress the presence of initial static shear will generally decrease the void ratio, and this influence is more prominent for loose sand. D_{rc} is thus not an independent variable but rather intimately linked to the stress state.

Clearly, density states looser than $D_{rc} = 35\%$ are not accessible to the sand at $\sigma'_{vc} \geq 400$ kPa and states looser than $D_r = 25\%$ are not accessible to sand at $\sigma'_{vc} \geq 100$ kPa.

Fig. 4.1 Compressibility of Silica sand at different deposition densities

4.4 MONOTONIC LOADING BEHAVIOUR

4.4.1 Influence of Confining Stress and Static Shear

Fig. 4.2 and 4.3 show the monotonic loading behaviour in simple shear for the loosest deposited sand at $\sigma'_{vc} = 100, 200$ and 400 kPa and α values of $0.00, 0.10, 0.20$ and 0.30 . It can be noted that the sand is highly contractive and thus strain softens under all initial stress states considered. Both σ'_{vc} increase at constant α , and α increase at constant σ'_{vc} promote more contractive behaviour, despite the associated increase in density. The relative density of the specimen varied between $25\% - 40\%$ in the loosest state for the confining stress range of $100- 400$ kPa and static shear stress ratio range of $0.0- 0.2$. The sudden drop of the shear stress after the critical state indicates the collapse of the structure of sample when flow liquefaction occurs. It is clear that for those tests with high initial static shear and high confining stress, stress control loading system could not have full control on the behaviours of loosest samples. However, by taking advantage of the high-speed data scanning capability of the system, control can be regained on samples of limited liquefaction. The straight line portions noted in the stress-strain curves do not represent actual response of the sand. Rather it's a manifestation of the fact that only a limited number of data points are available during flow deformation.

In order to investigate the influence of initial static shear on the contractiveness of sand, monotonic shear tests were also conducted on samples under 100 kPa confining

Fig. 4.2 Shear stress vs. shear strain behaviour of loosest deposited Silica sand in simple shear monotonic loading

Fig. 4.3 Stress path of loosest deposited Silica sand in simple shear monotonic loading

stress, different initial static shear ($\alpha = 0.00, 0.03, 0.06$ and 0.10) and also with 3 different densities representing loose, medium dense and dense states. Figures 4.4 and 4.5 show the stress strain curves for these tests. Figure 4.5 shows the responses when undrained shear was imposed on both the same direction as the static shear and to the opposite direction.

The brittleness index is regarded as a measure of the potential for strain softening in

a sand. The original definition of $I_B = \frac{S_{peak} - S_{min}}{S_{peak}}$ (Bishop, 1971), which does not incorporate the initial static shear, has been modified to take account of the presence of initial static shear. The modified brittleness index is defined as $I_{\bar{B}} = \frac{S_{peak} - S'_{min}}{S_{peak} - S_{static}}$ (Sivathayalan and Vaid, 2002), where $I_{\bar{B}} = 0$ corresponds to no strain softening, and $I_{\bar{B}} = 1$ represents the state in which the minimum undrained strength equals to the initial static shear stress. The variation of I_B and $I_{\bar{B}}$ with D_r and α are shown in Fig. 4.6. The influence of static shear on the potential for flow deformation is more prominent for loose sand. The low $I_{\bar{B}}$ for shear stress reversal indicated that a much larger stress increment is required to trigger strain-softening deformation. Bishop's original characterization is not intended for initial states with static shear, and hence the I_B values do not represent a measure of the actual potential for strain softening.

Fig. 4.4 Stress strain behaviour of Silica sand with initial static shear

Fig. 4.5 Stress strain behaviour of monotonic loading to the same direction as static shear and opposite

Fig. 4.6 Influence of initial static shear on the brittleness index, I_B (or $I_{\bar{B}}$)

4.4.2 Triggering of Contractive Deformation

Data points corresponding to the peak stress states of test results shown in Fig. 4.3 were picked up to find the locations of the monotonic Critical Stress Ratio (CSR) lines, as is shown in Fig. 4.7. It can be seen that the stress ratio at which strain softening is triggered is dependent on the value of the initial static shear. The friction angle mobilized at the critical state (ϕ_{CSR}) is essentially identical for α values of 0.0 and 0.1, but it increases as α increases further. Generally, an increase of initial shear stress ratio (α) will appear to increase ϕ_{CSR} . This behaviour is different from tests results on water pluviated sand in triaxial compression (Vaid and Chern, 1983 and Stedman, 1997). However the test data from Hollow Cylinder Torsional device by Sivathayalan and Vaid (2002) indicate the same influence of static shear on the ϕ_{CSR} . Also, it should be noted that ϕ_{CSR} in simple shear and triaxial extension loading modes is dependent on the relative density (Vaid and Thomas 1994, Vaid and Sivathayalan 1996). As mentioned before an increase of static shear will increase the relative density of the sample especially for the loosest deposited sand. This increase of density can partially explain why ϕ_{CSR} increases with the static shear stress level. It has also been shown that ϕ_{CSR} is dependent on the direction of principal stresses during consolidation (Sivathayalan and Vaid, 2002). And the presence of static shear in the simple shear loading mode will change the direction of the principal stresses.

The relationship between ϕ_{CSR} and α for the loosest deposited Silica sand on simple

shear is shown in Fig. 4.8. It should be noted that although ϕ_{CSR} increases to a high 19.7° when $\alpha = 0.3$, the peak shear stress is actually not much higher than the static shear. This indicates that the sample is already in an unstable state, and only a minor perturbation is required to trigger strain softening.

Fig. 4.7 CSR lines derived from monotonic loading tests on loosest deposited Silica sand using simple shear device

Fig. 4.8 Relationship between ϕ_{CSN} and α for loosest deposited Silica sand in monotonic simple shear loading

4.4.3 Phase Transformation and Steady State

Effective stress states corresponding to PT or SS or QSS in monotonic shearing tests is shown in Fig. 4.9. All tests regardless of density or type of response (i.e. contractive/dilative) are shown in the figure. As mentioned before, the stress-controlled loading system could not properly capture the post peak stress-strain response, if rapid run-away strains develop due to strain softening. As a result, data corresponding to the steady state were not chosen from those tests.

Fig. 4.9 Effective stress states at phase transformation/steady state

The loci of phase transformation or steady states lie on a straight line passing through the origin. The friction angle $\phi_{PT/SS}$ at these states is about 26° . This uniqueness of $\phi_{PT/SS}$ has also been reported by various researchers, regardless of the stress path and initial state prior to undrained loading. However it should be noted that the equivalency between PT and SS, QSS states is only valid in the stress space. The shear strength at different states (i.e., PT, SS, QSS states in stress-density space) is quite different depending on the mode of response.

Figure 4.10 shows the shear strength at Quasi-Steady/Steady state (i.e., $S_{SS/QSS}$) as a function of void ratio in monotonic simple shear ($\sigma'_{vc} = 100$ kPa). It appears that the presence of initial static shear has a minor influence on $S_{SS/QSS}$ at a given density state, but the available data are rather limited to draw definite conclusions. The shear strength

increases almost linearly with density. Such characteristics have been reported in the literature for various sands (Vaid and Sivathayalan, 1998).

Fig. 4.10 Undrained strength at Quasi Steady State/Steady State as a function of void ratio in monotonic simple shear ($\sigma'_{vc} = 100$ kPa)

4.5 CYCLIC LOADING BEHAVIOUR

The cyclic loading behaviour was investigated at three levels of vertical confining stresses: 100, 200 and 400 kPa. For each σ'_{vc} four initial shear stress ratios levels ($\alpha = 0.0, 0.06, 0.10, 0.20$) have been used. Three distinct mechanisms were responsible for the

development of large strains, depending on the initial state as demonstrated below.

Figure 4.11 shows contractive deformation in simple shear as the cause of liquefaction. The initial state of the specimen corresponds to $\sigma'_{vc} = 100$ kPa, $\alpha = 0.10$, and $e_c = 0.643$. No cyclic shear stress reversal occurs, and contractive deformation develops in the 6th stress cycle. PT/SS lines noted in the static tests are also shown, and it should be noted that the stress ratio at PT is essentially identical between monotonic and cyclic tests. However, the stress ratio at the initiation of contractive deformation in the cyclic tests is somewhat larger than that noted for the loosest deposited sand at $\sigma'_{vc} = 100$ kPa and $\alpha = 0.10$. The friction angle mobilized at the critical state in the cyclic test was about 15° , whereas the corresponding angle in the static tests was 13.5° .

A typical test where strain development due to excursion through transient state of zero effective stress as the cause of liquefaction is shown in Fig. 4.12. The initial state of the sand is $\sigma'_{vc} = 400$ kPa, $\alpha = 0.06$, and $e_c = 0.526$. Cyclic loading induces shear stress reversal. Little strain develops until the stress path crosses the PT line (as noted in static tests) in the 7th cycle. Subsequent loading cycles made the stress path travel through transient state of $r_u = 1$ (zero effective stress) and resulted in the development of progressively larger shear strain.

Fig. 4.11 Liquefaction due to contractive deformation in simple shear loading

Fig. 4.12 Liquefaction due to cyclic mobility with transient states of $r_u = 1$ during cyclic loading

The third type of failure, in which neither the occurrence of contractive deformation nor excursions through transient state of $r_u = 1$ is responsible for liquefaction due to cyclic loading is shown in Fig. 4.13. The initial state of the sand corresponding to $\sigma'_{vc} = 400$ kPa, $\alpha = 0.10$, and $e_c = 0.555$. The cyclic loading pulse does not involve stress reversal. But the second cycle leads the stress path to cross the CSR line causing significant shear strain. The shear strain after that point accumulates at a much smaller rate with each cycle. Strain increases every time the stress state makes excursions outside the PT line. Even though the strain level defined as liquefaction is reached in the third cycle, further stress cycles (i.e. post liquefaction loading) do not result in excessive straining. Because of the high value of α (no shear stress reversal), no uncontrolled deformation occurs. It should be noted that here $\phi_{CSR} = 17.5^\circ$ is not equal to the $\phi_{CSR} = 13.5^\circ$ measured in static shear for $\alpha = 0.10$, in which the sand is in the loosest state. This is consistent with the findings of Vaid and Sivathayalan (1996), as they reported that the ϕ_{CSR} value is a function of relative density in simple shear. And for denser sand this value should be larger.

Fig. 4.13 Liquefaction due to cyclic mobility without transient states of $r_u = 1$ during cyclic loading

4.6 COMPARISON BETWEEN STATIC AND CYCLIC LOADING BEHAVIOUR

Comparison of the features of the undrained response in static and cyclic loading enables evaluation of the effect of the type of loading on the undrained response of sand. Fig. 4.14 illustrates by data points the effective stress conditions at which the contractive deformation initiates under cyclic loading. Only data of the loosest deposited samples were presented. As we can see from Fig. 4.14, the ϕ_{CSR} values measured from cyclic loading showed almost the same trend as the values measured in monotonic loading (CSR lines under static loading are shown as solid), although some scatter is present in the data, which is a reflection of the effect of density on ϕ_{CSR} .

The friction angle $\phi_{PT/SS}$ mobilized at PT/SS during cyclic loading may be noted to be generally equal to that observed under static loading, regardless of the initial stress state and density (Fig. 4.15). Typical data for initial states that were not contractive under static loading, and hence would not be so under cyclic loading (Vaid and Chern, 1985), are also shown in Fig. 4.15 in addition to those that responded contractively. Thus the $\phi_{PT/SS}$ of a sand appears to be a unique property of the material, independent of either the initial stress states or density of the sample, as have been demonstrated in earlier studies on Fraser River sand (Vaid and Thomas, 1995 and Stedman, 1997) and other sands (Vaid and Chern, 1985; Vaid et al., 1990). The tests results support the contention that Steady State (SS) and Quasi Steady State (QSS) may be treated within the same framework in the stress space.

Fig. 4.14 Effective stress states at triggering of strain softening during cyclic loading

Fig. 4.15 Effective stress states at Steady State (SS)/ Quasi Steady State (QSS) or phase transformation during cyclic loading

4.7 CYCLIC RESISTANCE

Figure 4.16 shows cyclic resistance data at $\sigma'_{vc} = 100$ kPa and $\alpha = 0.10$ in the form of e_c vs. number of cycles to liquefaction. Each contour represents constant amplitude of cyclic stress ratio. Cycle stress amplitudes were selected such that liquefaction occurs in less than about 100 cycles. The mechanism of strain development leading to liquefaction is identified. In the figure, the letter “C” besides a data point indicates that contractive deformation was responsible for liquefaction.

Essentially linear behaviour between e_c and $\log(N)$ may be noted at each amplitude of constant CSR. Relative density and corresponding cyclic resistance pairs at a specified number of load cycles (which depend on the characteristics of a given earthquake; 10 selected herein) were picked up from the figure to yield the dependence of cyclic resistance on relative density at the chosen σ'_{vc} and α values. Contours similar to those in Fig. 4.16 at $\sigma'_{vc} = 100$ kPa, but at α values of 0.00, 0.06 and 0.20 are shown in Fig. 4.17 to Fig. 4.20. Data similar to that at $\sigma'_{vc} = 100$ kPa in Figs. 4.16 to 4.20 are included in appendix A for higher values of $\sigma'_{vc} = 200$ kPa and $\sigma'_{vc} = 400$ kPa.

Fig. 4.16 Cyclic resistance data plot, $e_c (D_r)$ vs. N for $\sigma'_{vc} = 100 \text{ kPa}$, $\alpha = 0.10$

Void Ratio vs. Number of Cycles to Failure
 $(\alpha = 0.00, \sigma'_{vc} = 100 \text{ kPa})$

Fig. 4.17 Cyclic resistance data plot, e_c (D_r) vs. N for $\sigma'_{vc} = 100 \text{ kPa}$, $\alpha = 0.00$

Void Ratio vs. Number of Cycles to Failure
 $(\alpha = 0.03, \sigma'_{vc} = 100 \text{ kPa})$

Fig. 4.18 Cyclic resistance data plot, $e_c(D_r)$ vs. N for $\sigma'_{vc} = 100 \text{ kPa}$, $\alpha = 0.03$

Void Ratio vs. Number of Cycles to Failure
 $(\alpha = 0.06, \sigma'_{vc} = 100 \text{ kPa})$

Fig. 4.19 Cyclic resistance data plot, $e_c (D_r)$ vs. N for $\sigma'_{vc} = 100 \text{ kPa}$, $\alpha = 0.06$

Fig. 4.20 Cyclic resistance data plot, e_c (D_r) vs. N for $\sigma'_{vc} = 100$ kPa, $\alpha = 0.20$

Contractive deformation was the cause of liquefaction during cyclic loading if for the selected initial state and cyclic stress level τ_{cyc} the following conditions were satisfied:

- (i) The sand was contractive under static loading
- (ii) The cyclic shear stress amplitude ($\tau_{cyc} + \tau_{st}$) exceeded its $S_{PT/SS}$
- (iii) The number of stress cycles applied is enough to carry the effective stress state to the CSR line.

The above requirements were first recognized and demonstrated by Vaid et al. (1989) from cyclic triaxial test results. If any of the conditions stated above was not satisfied, then strain during cyclic loading developed due to cyclic mobility associated with or without encountering transient states of zero effective stress, depending on whether shear stress reversal occurred or not.

The resulting cyclic resistance curves (i.e., the cyclic stress ratio required to trigger liquefaction in 10 cycles as a function of density) for Silica sand derived from the data in Figs. 4.16 to 4.20 and in Appendix A, at $\sigma'_{vc} = 100, 200$ and 400 kPa and at several levels of α (static shear) are plotted in Figs. 4.21 to 4.23. These figures completely describe the dependency of cyclic resistance on both confining stress and static shear stress levels, in addition to its dependence on relative density.

As pointed out previously, the minimum D_{rc} accessible to the sand in the loosest

Fig. 4.21 Cyclic resistance CRR vs. D_r for Silica sand at $\sigma'_{vc} = 100$ kPa

Fig. 4.22 Cyclic resistance CRR vs. D_r for Silica sand at $\sigma'_{vc} = 200$ kPa

Fig. 4.23 Cyclic resistance CRR vs. D_r for Silica sand at $\sigma'_{vc} = 400$ kPa

deposited state, and after the application of confining stress and static shear stresses used was about 40% under the highest stress level ($\sigma'_{vc} = 400$ kPa, $\alpha = 0.20$). Thus the effect of static shear and confining stress levels on cyclic loading can only be considered for $D_{rc} \geq 40\%$.

At any given confining stress level, CRR vs. D_{rc} relationship is profoundly influenced by the level of static shear. Generally in all situations, the presence of static shear decreases the cyclic resistance of this sand. The measured K_α is more than one only for the densest state under highest static shear and lowest confining stress. This is especially important since the Silica sand used in this study is highly contractive and is thus very liquefiable during an earthquake. Current design practice follows the methodology suggested by Seed and Harder (1990). The test data clearly suggest that the Seed and Harder correction factors would result in an unsafe design in this Silica sand. However, it is important to note that Seed and Harder factors have been found to be conservative in dilative Fraser River sand under cyclic triaxial conditions (Stedman, 1997).

This implies that the characteristics of the sand (i.e., dilative or contractive) and/or the loading mode do play a major role on how increased confining stress and static shear affect the cyclic resistance of a sand.

4.8 EFFECT OF STATIC SHEAR AND CONFINING STRESS LEVELS

The correction factor K_α can be obtained by normalizing the cyclic resistance at a

given α (Fig. 4.21 to 4.23) by the corresponding cyclic resistance under zero static shear ($\alpha = 0$). The computed K_α values are shown in Fig. 4.24- 4.26 as a function of the level of static shear α . It can be seen that D_r plays a major role on K_α at a given confining stress level. Increasing static shear makes looser sands much more prone to liquefaction. This is consistent with the monotonic test results reported earlier in Fig. 4.4 to 4.6.

In addition, it can be noted that the confining stress plays a role on the K_α value at a given density. At the higher density state of 75%, $K_\alpha \approx 0.9$ at 100 kPa confining stress level, but it drops to about 0.6 at 400 kPa. However, the effect of confining stress is not that serious at looser density states (the K_α value at $D_r = 40\%$ varies from about 0.25 at 100 kPa to 0.22 at 400 kPa)

It is also important to assess the influence of static shear on the K_σ vs. σ relations. This can be done by normalizing the cyclic resistance data at 200 and 400 kPa confining stresses (Fig. 4.22 and 4.23) by the corresponding data (the same α value) at 100 kPa (Fig. 4.21). For clarity the influence of static shear on the K_σ vs. σ relation is only plotted at three different densities ($D_{rc} = 40\%$, 60% and 70%), since K_σ is also a function of relative density. The results are shown in Fig. 4.27 to 4.29. It is clear that the K_σ factor is dependent not only on the confining stress level, but also on the level of initial static shear and relative densities. The K_σ value for loose sand ($D_{rc} = 40\%$) at 400 kPa confining stress is about 0.91 when no static shear stresses are present in the sand. However, it drops to about 0.82 when a static shear stress amounting to 20% of the vertical confining

Fig. 4.24 K_α vs. α for Silica sand at several density states and $\sigma'_{vc} = 100$ kPa

Fig. 4.25 K_α vs. α for Silica sand at several density states and $\sigma'_{vc} = 200$ kPa

Fig. 4.26 K_α vs. α for Silica sand at several density states and $\sigma'_{vc} = 400$ kPa

Fig. 4.27 Influence of initial static shear on the confining stress factor K_σ at $D_r = 40\%$

Fig. 4.28 Influence of initial static shear on the confining stress factor K_σ at $D_r = 60\%$

Fig. 4.29 Influence of initial static shear on the confining stress factor K_σ at $D_r = 70\%$

stress is present in the sand. And for dense sand ($D_{rc} = 70\%$) at 400 kPa confining stress the drop of K_σ is from 0.90 to 0.53. These results clearly indicate that the K_σ and K_α factors are not independent of each other as assumed in current practice, and thus the sequential application of the empirical factors is not rational.

In order to evaluate the combined influence of static shear and confining stress on cyclic resistance, the data in Fig. 4.22 and 4.23 are normalized by the cyclic resistance corresponding to no static shear ($\alpha = 0$) and at 100 kPa confining stress (Fig. 4.21). The data are plotted as $K_{\alpha\sigma}$ vs. α at several constant values of relative densities, loose to dense. Since $\sigma'_{vc} = 100$ kPa in Fig. 4.21 ($K_\sigma = 1$) and thus the combined correction factor $K_{\alpha\sigma}$ degenerates to K_α (Fig. 4.24). The resulting $K_{\alpha\sigma}$ vs. α data at 200 and 400 kPa confining stresses are shown in Fig. 4.30 and 4.31.

For the sand tested, the correction factors proposed by Seed and Harder (1990) grossly overestimated the cyclic resistance at all relative density states, and the discrepancies were largest at dense states. For example, for sand at $D_{rc} = 75\%$, the observed K_α is around 0.9 (at $\sigma'_{vc} = 100$ kPa) compared to the suggested value of around 2.0 at α value of 0.2 or higher.

A direct comparison of measured and predicted cyclic resistance based on Seed and Harder (1990) is illustrated in Fig. 4.32 and 4.33. The shaded region in these figures represent the predictions using Seed and Harder (1990) correction factors together with

the reference data shown at $\sigma'_{nc} = 100$ kPa and $\alpha = 0$. For a fixed α , the measured resistance at $\sigma'_{nc} = 200$ kPa and $\sigma'_{nc} = 400$ kPa is smaller than predicted regardless of the D_r level. The discrepancy is especially large when $\alpha = 0.20$.

Fig. 4.30 Combined factor $K_{\alpha\sigma}$ vs. α for Silica sand at several density states and $\sigma'_{vc} = 200$ kPa

Fig. 4.31 Combined factor $K_{\alpha\sigma}$ vs. α for Silica sand at several density states and $\sigma'_{vc} = 400 \text{ kPa}$

Fig. 4.32 Measured versus predicted cyclic resistance at fixed $\alpha = 0.10$ level

Fig. 4.33 Measured versus predicted cyclic resistance at fixed $\alpha = 0.20$ level

4.9 TEST RESULTS OF COMPARISON TESTS

4.9.1 Simple Shear Test on Fraser River Sand

In order to investigate the influence of the characteristics of different types of sand on the static shear correction factor K_α , a series of tests were conducted on Fraser River sand, which normally behaves more dilatively (Stedman, 1997) compared to the Silica sand tested earlier. Tests were conducted at a vertical confining stress of 100 kPa, over a range of densities and static shear stress ratios. The cyclic resistance curves at different α values, shown in Fig. 4.34, are derived from the raw data presented in Appendix B. The computed K_α values for Fraser River sand are shown in Fig. 4.35. The measured K_α values for Silica sand at 100 kPa confining stress are also shown in Fig. 4.35 for comparison. It can be noted that even though both sands were tested at the same stress level, and in the same simple shear loading mode, the K_α of Fraser River sand is much higher than that of Silica sand (at a given density). The K_α values for Fraser River sand are generally greater than unity (except at the loosest relative density state). This implies that the presence of static shear increases the cyclic strength in Fraser River sand, whereas it drastically decreased the cyclic strength in Silica sand. The primary reason for such drastically different responses is the degree of contractiveness. Silica sand is highly contractive at the tested initial states, whereas Fraser River sand is dilative at the considered initial states. Increasing static shear resulted in more damaging response in contractive sand, and less damaging response in dilative sand.

Fig. 4.34 Cyclic resistance CSR vs. D_r for Fraser River Sand at $\sigma'_{vc} = 100 \text{ kPa}$

Fig. 4.35 Comparison of test data from different sands at several density states and $\sigma'_{vc}=100\text{kPa}$

These results clearly highlight the fact that the inherent characteristics of the sand will play a major role on the effect of static shear on the liquefaction potential. Thus, using one single correction factor for different sand without regard to their inherent characteristics is not rational. Unfortunately current practice adopts such a single correction factor for all sands, and the consequences could be drastic if the sand present at a site is highly contractive.

Although test data from Fraser River sand agree very well with the K_α vs. α chart proposed by Seed and Harder (1990), the use of that chart for practical design purpose would not be rational as other sands may have different characteristics than Fraser River sand.

The K_α values computed from the cyclic simple shear tests are compared with the K_α values reported in the literature based on cyclic triaxial tests (Stedman, 1997) for Fraser River sand (Fig. 4.36). The Fraser River sand used by Stedman has a similar gradation, and is from the same geologic source as the sand used in this study.

The actual triaxial K_α values are somewhat larger than those of simple shear, but increasing density increases K_α in both cases. Increasing D_r makes a sand more dilative, and its effect is consistent both in simple shear and triaxial. The differences arise on account of the different loading mode. The K_α values reported by Stedman were based on the traditional definition of $CRR = \frac{\sigma_{dcy}}{2\sigma'_{3c}}$ in triaxial tests. This definition was originally

proposed to assess the cyclic strength of hydrostatically consolidated sands. It is postulated that CRR defined by $\frac{\sigma_{dcy}}{\sigma'_{1c} + \sigma'_{3c}}$ would be a better measure of the cyclic resistance, as it employs the shear and normal stresses in one plane, and accounts for anisotropic consolidation.

Fig. 4.36 Comparison of test data from different loading modes on Fraser River

sand, $CRR_{Triaxial} = \frac{\sigma_{dcy}}{2\sigma'_{3c}}$

The triaxial data in Fig. 4.36 is replotted in Fig. 4.37 using this new definition together with the simple shear data. The differences between triaxial and simple shear K_α values are now much smaller.

Fig. 4.37 Comparison of test data from different loading modes on Fraser River

sand, $CRR_{\text{Triaxial}} = \frac{\sigma_{dcy}}{\sigma'_{1c} + \sigma'_{3c}}$

4.9.2 Triaxial Test on Silica sand

Cyclic triaxial tests were carried out on Silica sand in order to validate the finding of the influence of loading mode. Specimens of two different relative densities (one loosest and the other medium dense) were prepared and cyclic tests were conducted under normal confining stress σ'_{nc} (normal effective stress on the 45° plane) of 100 kPa. The relationships between cyclic resistance ratio (CRR) and number of cycles to liquefaction are plotted in Fig. 4.38 and 4.39 for the loosest and medium dense specimen respectively. (CRR in triaxial tests are defined as $\frac{\sigma_{dcy}}{\sigma'_{1c} + \sigma'_{3c}}$). Cyclic triaxial tests were carried out on water pluviated sand, and the loosest relative density attainable at 100 kPa normal stress was between 15-20% (depending on K_c).

Relationship between K_α and α was calculated based on the data in Fig. 4.38 and 4.39. The result is presented in Fig. 4.40 together with the computed K_α values from simple shear tests. The K_α values in cyclic triaxial tests are much higher than that derived from simple shear tests for Silica sand. For example, for sand of relative density of around 50% under 100 kPa confining stress and $\alpha = 0.20$, the value of K_α is around 0.3 in simple shear, however in triaxial test the value is around 1.3. The triaxial value is around 4 times of the simple shear value. Similar observations are noted at looser deposited states as well. This indicates that cyclic triaxial tests would yield higher K_α values regardless of the inherent dilation potential of the sand.

Fig. 4.38 Cyclic resistance CRR vs. N for sample of loosest deposition at $\sigma'_{nc} = 100$ kPa

Fig. 4.39 Cyclic resistance CRR vs. N for sample of medium dense at $\sigma'_{nc} = 100 \text{ kPa}$

Fig. 4.40 K_α vs. α for Silica sand at $\sigma'_{nc} = 100$ kPa under triaxial loading mode

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

Constant volume static and cyclic simple shear tests on dry pluviated Silica sand have been conducted over a range of confining stresses, relative densities and initial static shear. The cyclic loading response was determined at three levels of confining stresses and four levels of static shear stress. The response was examined for each initial stress state over a broad range of densities and cyclic stress ratios. Two series of comparison tests have also been carried out by testing: 1) Fraser River sand in the simple shear and; 2) Silica sand in triaxial. The results of these tests lead to the following conclusions.

1. The static simple shear response of Silica sand is highly contractive over a large range of densities. Increasing static shear stress level will increase the degree of contractiveness, as will increasing confining stress. Also, increasing of static shear stress level will increase the value of ϕ_{CSR} in both monotonic and cyclic loading.
2. An increase in the static shear stress at a given confining stress will generally decrease the cyclic resistance in Silica sand. K_α value as small as 0.2 was measured when $\alpha = 0.2$ for sand at the loosest deposition density.

3. K_σ value is profoundly influenced by the presence of static shear. Under a given confining stress, increase of static shear will decrease the value of K_σ , and this drop is especially large for dense sands.
4. Different sands can give quite different values of K_α even when sheared under the same loading mode. This indicates that the dilation potential of sand influences the K_α value. Generally, dilative sands will give higher K_α values than contractive sa 118
5. Loading modes will influence the K_α values. Generally, cyclic triaxial tests with compression consolidation will give a higher K_α value than simple shear. Although a modified definition of CRR in triaxial test ($CRR = \sigma_{dcy} / (\sigma'_{1c} + \sigma'_{3c})$) gives a better agreement with simple shear data for Fraser River sand, the differences are much higher for the more contractive Silica sand.
6. The K_σ and K_α factors proposed by Seed and Harder (1990) grossly overestimate the combined correction factor $K_{\alpha\sigma}$ for Silica sand at all relative density states, regardless of the confining stress and static shear stress levels. However, in case of Fraser River sand the K_α factors measured in simple shear generally agree well with the values proposed by Seed and Harder (1990). The triaxial K_α values for Fraser River sand, however, do not.

Based on the results of this experimental work, the following suggestions are given for design and future research:

1. Seed and Harder's K_σ and K_α charts though conservative in dilative sands, may lead to unsafe designs in contractive strain softening sands. Use of these empirical correction factors without regard to the actual stress-strain response of the sand should be avoided in practice.
2. The K_α and K_σ are not independent factors as assumed in current practice. More experimental and modeling work should be undertaken to investigate in quantity the combined factor $K_{\alpha\sigma}$ and the influence of contractiveness of sand on $K_{\alpha\sigma}$.
3. Appropriate correction should be considered if cyclic triaxial tests with compression consolidation were employed for determining the effect of static shear. And this correction factor should be a function of static shear.

Chapter 6

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Appendix A

(Silica Sand Data)

Fig. A1 Cyclic resistance curves e_c (D_r) vs. N for $\sigma'_{vc} = 200$ kPa and $\alpha = 0.00$

Fig. A2 Cyclic resistance curves e_c (D_r) vs. N for $\sigma'_{vc} = 200$ kPa and $\alpha = 0.06$

Fig. A3 Cyclic resistance curves e_c (D_r) vs. N for $\sigma'_{vc} = 200$ kPa and $\alpha = 0.10$

Fig. A4 Cyclic resistance curves e_c (D_r) vs. N for $\sigma'_{vc} = 200$ kPa and $\alpha = 0.20$

Fig. A5 Cyclic resistance curves $e_c (D_r)$ vs. N for $\sigma'_{vc} = 400 \text{ kPa}$ and $\alpha = 0.00$

Fig. A6 Cyclic resistance curves $e_c (D_r)$ vs. N for $\sigma'_{vc} = 400 \text{ kPa}$ and $\alpha = 0.06$

Fig. A7 Cyclic resistance curves $e_c (D_r)$ vs. N for $\sigma'_{vc} = 400 \text{ kPa}$ and $\alpha = 0.10$

Fig. A8 Cyclic resistance curves e_c (D_r) vs. N for $\sigma'_{vc} = 400$ kPa and $\alpha = 0.20$

Appendix B

(Fraser River Sand Data)

Fig. B1 Cyclic resistance curves e_c (D_r) vs. N for $\sigma'_{vc} = 100 \text{ kPa}$ and $\alpha = 0.00$

Fig. B2 Cyclic resistance curves e_c (D_r) vs. N for $\sigma'_{vc} = 100$ kPa and $\alpha = 0.10$

Fig. B3 Cyclic resistance curves e_c (D_r) vs. N for $\sigma'_{vc} = 100$ kPa and $\alpha = 0.20$

Fig. B4 Cyclic resistance curves e_c (D_r) vs. N for $\sigma'_{vc} = 100$ kPa and $\alpha = 0.30$