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CYCLIC SIMPLE SHEAR PROPERTIES OF UNSATURATED RESIDUAL SOILS

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Abstract

In Singapore, two thirds of the land area is underlain by residual soils. One distinct characteristic of residual soils is its unsaturated nature. The relationships of shear modulus and damping ratio with shear strain for residual soils may differ from transported soils and have not been fully investigated. This study aims to investigate shear stiffness and damping ratio properties of residual soils under cyclic simple shear loading in both saturated and unsaturated conditions.

To date, there is no standard for cyclic simple shear test. Two commercial NGI-type cyclic simple shear apparatuses were evaluated using a “standard” sand, Ottawa 20-30. The evaluation revealed that there was a rocking movement on the top platen experienced by both apparatuses. As a result of rocking, the shear modulus below shear strain of 0.03% was much lower than that reported in the literature, and thus a cut-off shear strain for the cyclic simple shear apparatuses was suggested to be 0.03%. For a stiffer sand specimen tested, the rocking became more significant, resulting in lower shear modulus at shear strains ranging from 0.03% to 0.1%. A correction factor based on actual horizontal displacement ratio, R_δ , was proposed to correct the shear modulus. In addition, the evaluation also found that there was friction load and friction energy in the cyclic simple shear apparatuses. The friction load was corrected for shear modulus determination following the suggestion by ASTM D6528-07 (2007). A detailed correction procedure on friction energy was described for damping ratio determination. If the friction energy was left uncorrected, the damping ratio of the soil with shear strain will not follow an orderly trend.

Unsaturated soil specimens for cyclic simple shear tests were prepared using the pressure plate apparatus. To eliminate uncertainty with regards to the equilibrium matric suction, the equilibrium time of soil specimens in the pressure plate test was investigated. Independent matric suction measurement using a high suction tensiometer was performed during the equilibrium process. Corrected rates of gravimetric water content change of 0.15 %/day and of 0.08 %/day were suggested

as criteria for equilibrium in the pressure plate test for drying and wetting processes, respectively. Applicability of the proposed criterion for drying process was examined during the preparation of unsaturated residual soil specimens for cyclic simple shear tests. The measured matric suctions at the “equilibrium” time determined from the proposed criterion showed a maximum difference of 10% of the applied matric suctions.

Unsaturated soil zones (transition and residual zones) can be distinguished from saturated soil zone (boundary effect zone) using the soil-water characteristic curve (SWCC). To understand unsaturated soil behaviour, SWCC is therefore required to separate unsaturated soil behaviour in transition or residual zone and saturated soil behaviour in boundary effect zone. However, it is time-consuming to measure a complete SWCC. As such, a simplified method to estimate the SWCC of fine- and coarse-grained soils was developed. The proposed method has been evaluated for a total of 62 soils with 31 soils each for fine-grained and coarse-grained soils. The results showed that the proposed method is simpler and performed better than existing one-point methods.

The GCTS cyclic simple shear apparatus was modified to incorporate a suction-controlled system to enable testing of unsaturated soils. Cyclic simple shear tests were conducted on undisturbed and reconstituted residual soil specimens under saturated and unsaturated conditions. The test results showed that the volumetric threshold shear strains for unsaturated soils were larger than those of saturated soils. Once the volumetric threshold shear strain was exceeded, the number of loading cycles on shear modulus and damping ratio came into effect. Both the shear modulus and damping ratio decreased with number of loading cycles, due to a permanent microstructure change. As the excess pore-water pressure generation rate was faster in saturated soil specimens than in unsaturated soil specimens, the degradation parameter for saturated soil specimens was higher than for unsaturated soil specimens. For saturated soil specimens, the result showed that shear modulus increased and damping ratio decreased with mean effective stress. For unsaturated soil specimens, the test results showed that shear modulus increased and damping

ratio decreased with increasing matric suction. Using residual soil data from literature, a correlation to estimate G_{max} from basic soil properties was developed and evaluated. The estimated G_{max} had a discrepancy of $\pm 15\%$ from the experimental G_{max} based on data in the literature. The correlation was used to estimate the G_{max} of the residual soil specimens, thus normalized G/G_{max} can be plotted. The normalized G/G_{max} and damping ratio of residual soil specimens were compared with other studies. The comparison showed that the differences in G/G_{max} and damping ratio with shear strain of residual soil can be attributed to the different weathering conditions of the residual soils.

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LIST OF SYMBOLS

A	Constant
A_{soil}	Cross-sectional area of soil
a	Constant
B	Pore-water pressure parameter
b	Constant
C	Suction capacity
C_g	Grain characteristics, grain shape, grain size, grading and mineralogy
$C_1, C_2, C_3,$ C_4, C_5, C_6	Constant
C_c	Coefficient of curvature
C_f	Clay fraction
C_u	Coefficient of uniformity
$C(u_a-u_w)$	Correction factor for Fredlund and Xing equation
c_s	Cementation
$c(G)$	Shear modulus correction factor
D	Damping ratio
$D_{10}, D_{20}, D_{30},$ D_{50}, D_{60}, D_{90}	Soil particle diameters (in mm), corresponding to 10, 20, 30, 50, 60, and 90% passing, respectively on the cumulative grain-size distribution curve
D_{app}	Apparatus-generated damping
D_{max}	Damping ratio at large shear strain
D_{min}	Damping ratio at very small shear strain
D_r	Relative density
d	Diameter
d_e	Dominant particle size diameter
E	Young's modulus
e	Void ratio
e_0	Initial void ratio
e_{max}	Maximum void ratio

e_{min}	Minimum void ratio
F_{act}	Actual shear load
F_f	Friction load
F_m	Mean effective force
F_t	Shear load
F_{t1}	Shear load in the first loading cycle
F_{tN}	Shear load at the N^{th} loading cycle
$f(e)$	Function of void ratio
f	Frequency
f_n	Frequency at resonance
G	Shear modulus
G_1	Shear modulus in the first loading cycle
G_N	Shear modulus at the N^{th} loading cycle
G_{max}	Very small strain shear strain modulus
G_s	Specific gravity
g	Gravity
$g(OCR, PI)$	Function of overconsolidation ratio and plasticity index
H_e	Evaporation rate per unit water surface area
h	Height
K_0	Coefficient of earth pressure at rest
$K_{2,(max)}$	Function of relative density
K_t	Temperature, including freezing
L	Length
LL	Liquid limit
m	Constant
m_2^w	Coefficient of water volume change with respect to a change in matric suction
N	Number of loading cycles
n	Constant
O	Soil structure
OCR	Overconsolidation ratio
P_{200}	Percent of soil passing standard sieve #200

PI	Plasticity index
PL	Plastic limit
p	Constant
p_{atm}	Atmospheric pressure (= 100 kPa)
q	Constant
R^2	Coefficient of determination
R_δ	Ratio of actual displacement
S	Degree of saturation
T_s	Secondary effects and are a function of time
t	Time
t_g	Geologic age
t_δ	Degradation parameter
u_a	Pore-air pressure
u_b	Back pressure
u_w	Pore-water pressure
(u_a-u_w)	Matric suction
$(u_a-u_w)_{AEV}$	Air-entry value
$(u_a-u_w)_a$	Applied matric suction
$(u_a-u_w)_m$	Measured matric suction
$(u_a-u_w)_r$	Matric suction corresponding to θ_r
V	Volume
v_c	Compression wave velocity
v_s	Shear wave velocity
W	Weight
W_d	Damping energy
W_s	Maximum stored strain energy
W_{fd}	Friction damping energy
W_{fs}	Friction strain energy
W'_d	Damping energy solely due to soil
W'_s	Maximum stored strain energy solely due to soil
WEV	Water-entry value
w	Gravimetric water content

w_i	Initial gravimetric water content
w_L	Evaporation loss per unit water surface area
$\Delta\sigma'_v$	Change in vertical stress
Δu	Change in pore-water pressure
$\Delta(u_a-u_w)$	Change (increment or decrement) in matric suction
Δt	Change in time
Δw	Change in gravimetric water content
δ_{act}	Actual horizontal displacement
δ_h	Horizontal displacement
δ_i	Degradation index
$\delta_{lvd t}$	Applied horizontal displacement
δ_{mid}	Mid-height horizontal displacement
δ_n	Normal displacement
δ_{thrust}	Deflection of shear thrust
δ_{top}	Top platen horizontal displacement
η	Constant
ε_v	Vertical strain
γ	Unit weight of soil
γ_c	Shear strain amplitude
γ_{tl}	Linear cyclic threshold shear strain
γ_{tv}	Volumetric cyclic threshold shear strain
$\dot{\gamma}$	Strain rate
κ	Function of plasticity index
ν	Poisson's ratio
π	A mathematical constant, approximately equal to 3.14159
ρ	Density
ρ_b	Bulk density
ρ_{dry}	Dry density
ρ_w	Density of water
σ_l	Main principle stress

σ_3	Cell pressure (minor principle stress)
$(\sigma_1 - \sigma_3) = \sigma_d$	Deviatoric stress
$(\sigma_3 - u_a)$	Net confining stress
σ_n	Normal stress
σ'_h	Effective horizontal stress
σ'_m	Mean effective stress
σ'_v	Effective vertical stress
τ	Shear stress
τ_0	Stress and vibration history
θ_a	Residual air content
θ_r	Residual volumetric water content
θ_s	Saturated volumetric water content
θ_w	Volumetric water content

ABBREVIATIONS

AMK	Ang Mo Kio
ASTM	American Society for Testing and Materials
DOS	Disk Operating System
GDS	Geotechnical Digital Systems
GCTS	Geotechnical Consulting and Testing System
HST	High Suction Tensiometer
JM	Jalan Minyak
LVDT	Linear Variable Differential Transducer
MSE	Mean of Squared Errors
NGI	Norwegian Geotechnical Institute
NTU	Nanyang Technological University
PWD	Public Works Department
RMSE	Root Mean Squared Error
SGI	Swedish Geotechnical Institute
SSE	Sum of Squares Errors
SSNR	Sum of the Squared Normalized Residuals
SST	Sum of Squares Total
SWCC	Soil-Water Characteristic Curve
UNSODA	Unsaturated Hydraulic Database
USCS	Unified Soil Classification System.
WMO	World Meteorological Organization

Chapter 1

Introduction

1.1 Research Background

Measurement of small-strain shear stiffness and damping ratio of soils has gained increased recognition in recent years. The variation of shear stiffness with respect to shear strain levels for different geotechnical applications is shown in Figure 1.1. It can be seen that most geotechnical applications involve shear strain ranging from about 0.01% to 1%. In this shear strain range, the non-linear behaviour of soils can be studied using cyclic triaxial, cyclic simple shear and cyclic torsional shear tests. Amongst these tests, cyclic simple shear test has been considered to simulate pure shear stress condition in the most direct way (e.g. Whang 2001; Hsu 2002). The cyclic simple shear test of residual soils is the subject of this study.

Figure 1.1 Range and applicability of laboratory tests (modified from Mair 1993).

There are two types of cyclic simple shear apparatuses: NGI-type and Roscoe-type. The NGI-type cyclic simple shear apparatus uses a cylindrical specimen confined in a reinforced rubber membrane while the Roscoe-type cyclic simple shear apparatus uses a cubic specimen enclosed by square metal rings all around. Over the years, a number of cyclic simple shear apparatuses based on either the NGI- or Roscoe-type cyclic simple shear apparatus have been developed and commercialized. Silver and Seed (1971a) used the NGI-type cyclic simple apparatus to test silica sand in the shear strain range from 0.01% to 0.5%. However, experimental results suggest that due to compliance in cyclic simple shear apparatus, determination of shear modulus and damping ratio of soil at the shear strain magnitude on the order of 0.01% was not reliable and consequently most of the shear stiffness and damping ratio obtained by cyclic simple shear apparatuses were reported in the shear strain range from 0.1% to 1% (Amer et al. 1986; Kagawa 1992; Sitar and Salgado 1994; Miller et al. 1996; Whang et al. 2004; Porcino et al. 2006; Seo 2008; Spikula and Garmon 2008; Jafarzadeh and Sadeghi 2009). However it was claimed that double specimen direct simple shear apparatus (DSDSS) was able to perform tests down to a shear strain magnitude of about 0.001% (Doroudian and Vucetic 1995). But the DSDSS is not widely used or commercialized, probably because of its manual control of shear strain and the accuracy of the test results depend on the skill of the operator (Nwaiwu and Osinubi 2005). In this study, two commercial actuator-operated NGI-type cyclic simple shear apparatuses were evaluated to check their shear strain range.

Previous studies on dynamic properties (stress-strain properties under (fast) cyclic loading conditions) of soils have focused primarily on dry and saturated transported soils, i.e. sands and clays (Thiers and Seed 1968; Silver and Seed 1971a; Kagawa 1992; Sitar and Salgado 1994; Spikula and Garmon 2008; Jafarzadeh and Sadeghi 2009). Relatively few studies were conducted on residual soils (Borden et al. 1996;

Macari and Hoyos 1996; Leong et al. 2003). Furthermore, these studies were conducted using resonant column, cyclic torsional shear and cyclic triaxial apparatus. Cyclic simple shear properties of residual soils have yet to be reported. Residual soils are soil-like materials derived from in-situ weathering, and decomposition of rock which has not been transported laterally from its original location (Blight 1997). As a product of weathering, these residual soils generally exist above groundwater table and are unsaturated in nature. Thus, their behaviour is affected by the air and water in the pore spaces which in turn influence the stress state of the soils. In the unsaturated state, a single stress state variable, i.e. effective stress, is insufficient to describe the mechanical behaviour of the soil (Bishop 1959; Skempton 1960). Both pore-air and pore-water pressure terms have to be considered with total stress which are given as two stress state variables, i.e. net stress and matric suction (Fredlund and Morgenstern 1977). Therefore, it is important to investigate the fundamental behaviour of residual soil covering the unsaturated condition under cyclic simple shear loading.

Research has shown that the soil-water characteristic curve (SWCC) of a soil can be related to a number of unsaturated soil properties. The SWCC expresses the amount of water in a soil versus soil suction. The air-entry value separates the boundary effect zone from the transition zone while the residual value separates the transition zone from the residual zone of the SWCC (White et al. 1970; Vanapalli 1994). An initially saturated soil specimen begins to desaturate when it is subjected to soil suction beyond the air-entry value and becomes fully desaturated as it enters the residual zone. Therefore, it is a “pre-requisite” to determine SWCC in a study involving unsaturated soils. However, it is time-consuming to measure a complete SWCC which has three distinct zones. Moreover, undisturbed soil samples are always limited and may not be available for SWCC determination. Therefore, a procedure is developed to indirectly estimate the SWCC to facilitate testing of

unsaturated residual soils.

Unsaturated soil specimens at a known matric suction can be prepared using the pressure plate apparatus (Edil and Motan 1979; Luh 1980) before being transferred to the test apparatus (e.g., resonant column apparatus, cyclic simple shear apparatus, and etc.) to determine its mechanical properties. The pressure plate apparatus is widely used due to its simplicity and relatively broad range of matric suction that can be applied to the soil specimen. Despite the extensive use of pressure plate apparatus, there appears little guidance for determining the equilibrium time of the soil water under an applied matric suction for a given soil. Marinho et al. (2008) reported that the equilibrium time for pressure plate test can be very long. It was generally assumed that the soil specimen reached the applied matric suction when the weight of the soil specimen shows negligible change. To eliminate uncertainty with regards to the equilibrium matric suction, this study investigates the relationship of the equilibrium time of soil specimens in pressure plate test and applied matric suction with independent matric suction measurement using a high suction tensiometer.

1.2 Objectives and Scope

The objective of this research is to study the cyclic simple shear properties of residual soils under saturated and unsaturated conditions. The objective is accomplished through the following:

- 1) Evaluating the cyclic simple shear apparatus to better understand the principles and limitations of cyclic simple shear test and adopting measures to address its limitations.
- 2) Modifying the cyclic simple shear apparatus to incorporate a suction-control system to test unsaturated soil specimens.

- 3) Investigating the equilibrium time of soil specimens in pressure plate test with independent matric suction measurement using a high suction tensiometer to facilitate unsaturated soil tests.
- 4) Developing a procedure to indirectly estimate the soil-water characteristic curve to facilitate unsaturated soil specimen preparation.
- 5) Investigating the factors affecting the cyclic simple shear properties of residual soil.
- 6) Developing a procedure to indirectly estimate the very small strain shear modulus, G_{max} .
- 7) Developing normalized shear modulus and damping ratio with shear strain relationships for residual soils.

1.3 Organization of the Thesis

The thesis is organized into seven chapters:

Chapter 1 outlines the research background and the objectives of this study.

Chapter 2 provides an overview of laboratory cyclic testing, factors affecting the shear modulus and damping ratio. This chapter also briefly describes unsaturated soils, the characteristics and dynamic properties of residual soils. A critical review of cyclic simple shear tests is also presented.

Chapter 3 describes the GCTS and GDS cyclic simple shear apparatuses. The instrumentations and characteristics of the two cyclic simple shear apparatuses are presented. The limitations of the cyclic simple shear apparatuses are discussed. Finally, a correction procedure to address the limitations is proposed

Chapter 4 presents the investigation of equilibrium time of soil specimens in

pressure plate test with independent matric suction measurement using a high suction tensiometer. Preparation of unsaturated soil specimens using the pressure plate is described.

Chapter 5 describes the soil-water characteristic curve and its relationship to the mechanical behaviour of soil. A method to indirectly estimate the soil-water characteristic is proposed. The proposed method is evaluated and compared with other methods using independent data from published literature.

Chapter 6 presents the cyclic simple shear tests of residual soils. The effects of number of loading cycles, shear strain, mean effective stress and matric suction on the shear modulus and damping ratio of residual soil are investigated in this chapter. An empirical correlation is developed to estimate the very small strain shear modulus, G_{max} . The proposed correlation is evaluated using data from published literature. Finally, the normalized shear modulus and damping ratio with shear strain relationships of residual soils are compared with other studies.

Chapter 7 presents the main conclusions drawn from the present research followed by recommendations for future research.

Chapter 2

Literature Review

2.1 General

This chapter presents the literature relevant to the research. An overview of laboratory cyclic testing is described in Section 2.2. Section 2.3 presents the factors affecting shear modulus and damping ratio. Characteristic and dynamic properties of unsaturated soils are briefly discussed in Section 2.4. Section 2.5 describes the characteristics and dynamic properties of residual soils. Finally, a review of cyclic simple shear apparatus is presented in Section 2.6.

2.2 Overview of Laboratory Cyclic Testing

There are many types of laboratory tests that can be used to determine the dynamic properties of soils over a large strain range. These tests include cyclic simple shear test, resonant column test, cyclic torsional shear test and cyclic triaxial test. The ranges of shearing strain amplitudes and their applicability in design are shown in Figure 1.1.

2.2.1 Cyclic Simple Shear Test

Cyclic simple shear test is a convenient method for determining the shear modulus and damping ratio of soils (Das 1993). A cylindrical (i.e., NGI-type) or cuboidal (i.e., Roscoe-type) specimen is subjected to a vertical stress and a shear stress as shown in Figure 2.1. The horizontal load required to deform a specimen is measured by means of a load cell and the shear deformation of the specimen is measured by a linear variable differential transformer (LVDT). A typical cyclic loop generated by cyclic simple shear apparatus is shown in Figure 2.2. From Figure 2.2, the shear modulus, G can be expressed as:

$$G = \frac{\Delta\tau}{\Delta\gamma_c} \quad (2.1)$$

and the damping ratio, D is given as:

$$D = \frac{1}{2\pi} \frac{\text{Area of hysteresis loop}}{\text{Area of triangle } OAB \text{ \& } OA'B'} \tag{2.2}$$

Figure 2.1 Stress conditions in simple shear test (after Das 1993).

Although cyclic simple shear test can simulate behaviour of the in-situ soil element closely during dynamic excitation and pure shear stress conditions, it has limitations. The major drawbacks include non-uniform stress and strain distributions and rocking (Lucks et al. 1972; De Alba et al. 1976, Shen et al. 1978; Boulanger et al. 1993; Miller et al. 1996; Duku et al. 2007). These limitations will be discussed in details in Section 2.5.3.

Figure 2.2 Typical cyclic loop generated by cyclic simple shear apparatus.

2.2.2 Resonant Column Test

Determination of shear modulus and damping ratio of soils by resonant column test is based on theory of wave propagation in prismatic rods (Richart et al. 1970). The soil column is either excited longitudinally or torsionally (Figure 2.3), which yield longitudinal wave velocity, v_L , or shear wave velocity, v_s , respectively. The resonant column test procedure is given in ASTM D4015-07 (2007). In a resonant column apparatus, the cylindrical soil specimen (constrained either free-free end or fixed-free end) is subjected to excitation by varying the frequency until the soil specimen experiences resonance. Once the resonant frequency, f_n , is known, the wave velocity can be determined. Thus, Young's modulus, E and shear modulus, G can be determined from compression wave and shear wave velocities, respectively, as

$$E = \rho v_L^2 \quad (2.3)$$

$$G = \rho v_s^2 \quad (2.4)$$

where ρ is density of soil specimen.

Figure 2.3 Schematic diagram of a soil specimen subjected to longitudinal and torsional excitations (after ASTM D4015-07 2007).

With free-free end conditions, using longitudinal excitation, the Young's modulus, E , is given as:

$$E = 4 f_n^2 \rho L^2 \quad (2.5)$$

and using torsional excitation, the shear modulus, G , is given by:

$$G = 4 f_n^2 \rho L^2 \quad (2.6)$$

where L is length of soil specimen.

With fixed-free end conditions, using longitudinal excitation, the Young's modulus is given as:

$$E = 4\pi^2 \left(\frac{f_n^2 L^2}{\alpha_r^2} \right) \rho \quad (2.7)$$

and using torsional excitation, the shear modulus is given by:

$$G = 4\pi^2 \left(\frac{f_n^2 L^2}{\alpha_r^2} \right) \rho \quad (2.8)$$

where α_r is given as:

$$\alpha_r = \frac{A_{soil} L \gamma}{W \tan \alpha_r} \quad (2.9)$$

A_{soil} is the cross-sectional area of the specimen; γ is the unit weight of soil; W is the weight of the attachments on top of the specimen.

Using the free vibration decay method (Figure 2.4), damping ratio can be computed as follows:

$$D = \sqrt{\frac{\delta^2}{4\pi^2 + \delta^2}} \quad (2.10)$$

where π is a mathematical constant (i.e., 3.14159...) and δ is the logarithmic decrement obtained by plotting the accelerometer peak output for successive cycles on a logarithmic scale. It can be determined from the peak output as follows:

$$\delta = \frac{1}{N} \ln \left(\frac{u_1}{u_{i+1}} \right) \quad (2.11)$$

where u is the peak output of the cycle indicated by the sub index and N is the number of cycles between the chosen peak outputs (Figure 2.5).

Figure 2.4 Typical free vibration decay curves obtained from resonant column test (after Hall 1962).

Despite its versatility in measuring dynamic soil properties, the resonant column apparatus has several shortcomings as highlighted by Drnevich (1972). In resonant column apparatuses which use solid cylindrical specimen, non-uniform strain distributions and incapability of testing high shear strain amplitude are among the major disadvantages. For high shear strain test (torsional motion), incomplete coupling exists between the soil specimen (particularly for stiff soil specimen) and

end (i.e. top and bottom) platens. Consequently, slippage between soil specimen and end platens occurs, resulting in lower shear modulus and higher damping ratio of soil specimen being measured. Drnevich (1972) reported that non-uniform strain distributions can be minimized or overcome by the use of hollow cylindrical specimens. However, the use of hollow cylindrical specimen presents a challenge in specimen preparation especially for cohesionless soil and mounting it in the resonant column apparatus (Woods 1978).

Figure 2.5 Determination of logarithmic decrement (after Hall 1962).

2.2.3 Cyclic Triaxial Test

Cyclic triaxial tests have been used extensively to evaluate the cyclic behaviour of soils (Seed and Lee 1966) and are documented in ASTM D3999–91 (2003). Cyclic triaxial test can be stress-controlled or strain-controlled. In stress-controlled tests, cyclic shear stress is applied to the cylindrical soil specimen by means of a cyclic axial deviatoric stress, $\sigma_d = (\sigma_1 - \sigma_3)$, that is assumed to act on a potential failure plane which is inclined $(45^\circ + \phi/2)$ from the horizontal. Initial stresses can be introduced by consolidating the soil specimen under a mean effective stress, σ'_m .

Commonly, the test is performed by maintaining the cell pressure, σ_3 , at a constant value and the vertical deviatoric stress is cycled by $\pm \sigma_d$ (Peacock and Seed 1968). In strain-controlled tests, the soil specimen is subjected to a cyclic axial strain, $\pm \varepsilon$, while the resulting deviatoric stress, $\pm \sigma_d$ is measured. The principal stresses in cyclic triaxial test are shown in Figure 2.6. The Young's modulus, E can be calculated from the axial stress-strain loop (Figure 2.7) and is given as:

$$E = \frac{\Delta \sigma_d}{\Delta \varepsilon} \quad (2.12)$$

Shear modulus, G can be calculated from the theory of elasticity which is given as:

$$G = \frac{E}{2(1+\nu)} \quad (2.13)$$

where ν is Poisson's ratio. In undrained test, a representative ν of 0.5 can be assumed.

Damping ratio is obtained from the axial stress-strain curve (Figure 2.7) and the relationship is given as:

$$D = \frac{1}{2\pi} \frac{\text{Area of hysteresis loop}}{\text{Area of triangle } OAB \text{ \& } OA'B'} \quad (2.14)$$

Some of the major limitations of conventional cyclic triaxial tests as highlighted by Woods (1978) and Bhatia et al. (1985) are different behaviour of soil under axial compression and extension, non-uniform condition during cyclic loading (i.e., stress concentrations at the top cap and base of the specimen), rotation of the principal stress directions by 90° , void ratio redistribution within the specimen during cyclic loading, and difficulty in achieving shear strain levels below 0.01% (e.g. equipment compliance and resolution in the axial strain measurement).

Recent research has shown that advanced cyclic triaxial apparatus equipped with local transducers (e.g. local deformation transducer) can evaluate stress-strain behaviour at strains from about 0.001% to 1.0% (Tatsuoka et al. 1994). The local measurement bypasses the deformation of loading piston, load cell, and connections, and eliminates the bedding errors at the top and bottom ends of the soil specimen. However, such test is not yet standardized and thus not commonly adopted in design engineering practice but is only performed in specialized soil testing laboratories (Jardine et al. 1985; Tatsuoka et al. 1997).

Figure 2.6 Principal stresses in cyclic triaxial test (after Bhatia et al. 1985).

Figure 2.7 Test result from cyclic triaxial apparatus.

2.2.4 Cyclic Torsional Shear Test

The cyclic torsional shear apparatus was developed to overcome some of the difficulties associated with cyclic triaxial and simple shear tests (Woods 1978; Kramer 1996). Kramer (1996) pointed out that the cyclic torsional shear tests allow isotropic or anisotropic initial stress conditions and can impose cyclic shear stresses on horizontal planes with continuous rotation of principal stress axes.

In the early design of cyclic torsional shear apparatus, Ishihara and Li (1972) modified the triaxial apparatus to provide torsional straining capabilities over solid cylindrical specimen. The distinct shortcoming of this solid cylindrical specimen is that shear strain range from zero at the centre of the specimen to a maximum at the outer radius. To improve the radial uniformity of shear strains, hollow cylinder cyclic torsional shear apparatuses have been developed (Drnevich 1972; Ishibashi and Sherif 1974; Ishihara and Yasuda 1975; Sherif et al. 1977; Tatsuoka et al. 1982; Ampadu and Tatsuoka 1993).

Generally, in hollow cylinder apparatus, the specimen is enclosed within internal and external membranes on which internal and external pressures can be applied independently. Cyclic shear stress is induced by cyclic torque on the horizontal planes as shown in Figure 2.8.

Figure 2.8 Stress conditions in hollow cylinder cyclic torsional shear test
(after Kramer 1996).

A typical cyclic loop generated by hollow cylinder cyclic torsional shear apparatus is shown in Figure 2.9. From Figure 2.9, the shear modulus can be expressed as:

$$G = \frac{\Delta \tau}{\Delta \gamma_c} \quad (2.15)$$

and the damping ratio is given as:

$$D = \frac{1}{2\pi} \frac{\text{Area of hysteresis loop}}{\text{Area of triangle } OAB \text{ \& } OA'B'} \quad (2.16)$$

Figure 2.9 Typical cyclic loop generated by hollow cylinder cyclic torsional shear apparatus (modified from Sherif et al. 1977).

Wright et al. (1978) reported that shear stress is more uniformly distributed when long hollow specimen configuration is used. In spite of the advantage of having uniform shear strain distributions, preparation of hollow cylinder specimen is not easy especially for undisturbed cohesionless soils (Woods 1978).

2.2.5 Summary

Laboratory cyclic tests and their working principles, range of capabilities, advantages and limitations in determining the shear modulus and damping ratio of

soils have been described. Summary of laboratory techniques to measure dynamic soil properties and the parameters measured in dynamic laboratory tests are given in Tables 2.1 and 2.2, respectively. Literature review shows that the cyclic simple shear apparatus is one of the apparatuses which can reasonably simulate pure shear stress condition. Furthermore, it provides a more direct way to measure shear modulus. Unlike cyclic triaxial apparatus, shear modulus can only be determined if Poisson’s ratio is known. Poisson’s ratio may be assumed as 0.5 in saturated undrained case. However, it may not be applicable in unsaturated soil test and needs to be measured independently for accurate shear modulus determination. Therefore, in this study, the cyclic simple shear test is preferred to other cyclic laboratory tests.

Table 2.1 Laboratory techniques for measuring dynamic soil properties
(modified from Woods 1978)

Type of techniques	Shear modulus, <i>G</i>	Young’s modulus, <i>E</i>	Damping ratio, <i>D</i>
Resonant column test	√	√	√
Cyclic triaxial test		√	√
Cyclic torsional shear test	√		√
Cyclic simple shear test	√		√

Table 2.2 Parameters measured in dynamic laboratory tests (after Silver 1981)

Parameters	Resonant column test	Cyclic triaxial test	Cyclic torsional shear test	Cyclic simple shear test
Load	Resonant frequency	Axial force	Torque	Horizontal force
Axial deformation	Vertical displacement	Vertical displacement	Vertical displacement	Vertical displacement
Shear deformation	Acceleration	Not measured	rotation	Horizontal displacement
Lateral deformation	Not usually measured	Can be measured using local gauge	Not usually measured	Often controlled
Volumetric deformation	None for undrained tests			
	Volume of fluid moving into or out of the sample for drained tests			
Pore-water pressure	Not usually measured	Measured at boundary	Measured at boundary	Measured at boundary

2.3 Factors Affecting Dynamic Properties of Soils

Hardin and Black (1968) investigated shear modulus and damping ratio of undisturbed and reconstituted soils and concluded that the functional relationship for the shear modulus can be written as:

$$G = f(\sigma'_m, e, C_g, \tau_0, O, f, \gamma_c, S, T_s, K_t) \quad (2.17)$$

where σ'_m is mean effective stress; e is void ratio; C_g is grain characteristics, 'grain shape, grain size, grading and mineralogy; τ_0 is stress and vibration history (including overconsolidation ratio, OCR , and number of loading cycles, N); O is soil structure; f is frequency (including strain rate, $\dot{\gamma}$); γ_c is shear strain; S is degree of saturation; T_s is secondary effects and is a function of time (including geologic age, t_g , thixotropy, and cementation, c_s); and K_t is temperature, including freezing effect.

Hardin and Drnevich (1972a) conducted a series of resonant column and cyclic torsional shear tests to investigate the factors affecting the shear modulus and damping ratio of cohesive and cohesionless soils (Table 2.3). From their study, they found that many factors affect the shear modulus and damping ratio of the soils. The influencing factors have been grouped into three categories: very important, less important, and relatively unimportant. From Table 2.3, it can be seen that shear strain, mean effective stress, and void ratio are very important influencing factors on shear modulus and damping ratio of cohesive and cohesionless soils. These factors will be discussed in detail in the following sub-sections. Degree of saturation is a very important factor affecting shear modulus for cohesive soil but is relatively unimportant for cohesionless soil. For damping ratio, the effect of degree of saturation is less important for cohesionless soil, but not fully known at that point of time for cohesive soil. Table 2.3 also shows that the number of loading cycles is a very important factor affecting damping ratio but is a relatively unimportant factor in affecting shear modulus for both cohesive and cohesionless soils.

2.3.1 Shear strain

Atkinson and Sällfors (1991) suggested that the stiffness-strain and damping ratio-strain curves can be divided into three strain ranges: very small strains (0.001% or less), small strains (between 0.001 and 1%) and large strains (larger than 1%). For ease of identification, the definitions of strain ranges given by Atkinson and Sällfors (1991) are adopted in the thesis.

Table 2.3 Factors affecting shear modulus and damping ratio for cohesive and cohesionless soils (modified from Hardin and Drnevich 1972a)

Factors	Degree of importance			
	Shear modulus		Damping ratio	
	Cohesion-less soil	Cohesive soil	Cohesion-less soil	Cohesive soil
Shear strain, γ_c	V	V	V	V
Mean effective stress, σ'_m	V	V	V	V
Void ratio, e	V	V	V	V
Number of loading cycles, N	R	R	V	V
Degree of saturation, S	R	V	L	U
Overconsolidation ratio, OCR	R	L	R	L
Shear stress history, τ_0	L	L	L	L
Frequency, f	R	R	R	L
Secondary effect that is a function of time (thixotropy), T_s	R	L	R	L
Grain size characteristics, C_g	R	R	R	R
Soil structure, O	R	R	R	R

Note: V: Very important; L: Less important; R: Relatively unimportant; U: unclear

Vucetic (1994) defined the three strain ranges differently: very small strains are below linear cyclic threshold shear strain, γ_{tl} , small strains are between γ_{tl} and volumetric cyclic threshold shear strain, γ_{tv} , and medium to large strains are above γ_{tv} . According to Vucetic (1994), soils behave as linearly elastic material (i.e., settlement is negligible or non-destructive behaviour) for strains below the linear cyclic threshold shear strain, γ_{tl} . Below γ_{tl} , shear modulus is constant at a maximum value, G_{max} , and material damping ratio is constant at a minimum value, D_{min} . Above γ_{tl} , soils become markedly non linear but remain to behave mainly as elastic materials below the volumetric cyclic threshold shear strain, γ_{tv} . In other words, the stress-strain relationship is curved, but the deformations are recoverable upon unloading. A decrease of normalized shear modulus and an increase of damping

ratio are observed due to non-linear behaviour of soils. Furthermore, above γ_{lv} , soils become highly non-linear and inelastic. Permanent microstructure changes significantly during cyclic loading. These permanent microstructure changes with number of loading cycles, N , and are referred to as residual cyclic pore-water pressure in fully saturated soil and permanent volume change (i.e., settlement) in dry or unsaturated soils (Vucetic 1994; Hsu 2002). At this strain, the shear modulus decreases to about 80% of G_{max} , and material damping is about 3% higher than D_{min} (Stokoe et al. 1999). Generally, the normalized shear modulus decreases whereas damping ratio increases with shear strain. The normalized shear modulus-strain and damping ratio-strain relationships are shown in Figure 2.10.

Figure 2.10 Definitions of very small strains, small strains and medium to large strains (modified from Vucetic 1994).

2.3.2 Soil Type and Plasticity Index

It is recognized that soil type is one of the most important parameters affecting dynamic non-linear soil behaviour. For transported soils, plasticity index, PI , plays a significant role on non-linear behaviour of soils. PI is a measure of the range of

water content over which a soil behaves plastically. Numerically, it is the difference between the liquid limit, *LL* and plastic limit, *PL*. Gravel and clean sand are non-plastic.

For the past four decades, the influence of *PI* on non-linear soil behaviour has been reported by numerous researchers (Zen et al. 1978; Kokusho et al. 1982; Vucetic and Dobry 1991; Stokoe et al. 1999; Darendeli et al. 2001). Based on their findings, normalized shear modulus of high plasticity soils degrade more slowly with shear strain amplitude than that of low plasticity soils. It was observed that damping ratio of high plasticity soils is lower than that of low-plasticity soils at a given small to large shear strain. The effects of *PI* on normalized shear modulus reduction and damping ratio curves are shown in Figure 2.11. For very small strains, Vucetic et al. (1998) and Stokoe et al. (1999) reported that D_{min} increases with increasing *PI*.

Figure 2.11 Effect of soil type on normalized shear modulus and damping ratio (after Darendeli 2001).

2.3.3 Mean Effective stress

The relationship of very small strain or maximum shear modulus G_{max} and mean effective stress can be approximated using a power law given as follows (Hardin and Richart 1963):

$$G_{max} = a(\sigma'_m)^b \quad (2.18)$$

where σ'_m is mean effective stress; and a and b are constants.

In general, G_{max} increases as mean effective stress increases. For most cohesionless soils, b is less than 0.5 (Seed and Idriss 1970). For residual soils, Borden et al. (1996) reported that exponent b ranges from 0.35 to 0.40 with higher values being associated to soils with higher sand content.

A number of studies have been conducted to investigate the effect of σ'_m on dynamic properties of sands and clays (Iwasaki et al. 1978; Kokusho 1980; Darendeli et al. 2001). It was found that as σ'_m increases, the normalized shear modulus (G/G_{max}) degrades at a slower rate with increase in shear strain amplitude (i.e., curve shifted to right as σ'_m increases) as shown in Figure 2.12(a). For damping ratio D , Darendeli et al. (2001) reported that for silty sand subjected to high σ'_m , D is lower than that of soils subjected to low σ'_m at a given strain (Figure 2.12b). Similar results were observed by Hardin and Drnevich (1972a) and Tatsuoka et al. (1978) for clean sands and cohesive soils and Toyoura sand, respectively. However, Iwasaki et al. (1978) and Kokusho et al. (1982) reported that low plasticity soils are more affected by σ'_m than that of high plasticity soils.

2.3.4 Void Ratio

Hardin and Black (1968) performed a series of resonant column tests for cohesive soils to investigate the effects of void ratio and mean effective stress on G_{max} . It was found that the G_{max} tends to decrease with increasing void ratio for a given mean effective stress σ'_m as shown in Figure 2.13.

Kokusho (1980) presented the effect of void ratio on shear modulus, normalized shear modulus (G/G_{max}) and damping ratio (D) for saturated Toyoura sand at different shear strains. He concluded that shear modulus degrades at a slower rate with decreasing void ratio (curve shifted to the right) as shown in Figure 2.14. Furthermore, the void ratio has less influence on the strain dependency curves of normalized shear modulus (G/G_{max}) and damping ratio (D) when compared to the effect of σ'_m . The effects of void ratio on normalized shear modulus (G/G_{max}) and damping ratio (D) are shown in Figures 2.15 and 2.16, respectively. Both curves tend to shift slightly to the left along the x-axis with increasing void ratio. Iwasaki et al. (1978) and Tatsuoka et al. (1978) concluded that void ratio has no influence on normalized shear modulus and damping ratio of sands. Using double specimen direct simple shear (DSDSS) apparatus, Hsu (2002) investigated the effect of void ratio on normalized shear modulus and damping ratio for soils with PI ranging from 0 – 44. He reported that void ratio has no effect on normalized shear modulus and damping ratio for both plastic and non-plastic soils.

Figure 2.12 Effect of mean effective stress on (a) normalized shear modulus; and (b) material damping ratio (after Derendeli et al. 2001).

Figure 2.13 Effect of void ratio on $G_{\max}$ (modified from Ishihara 1996).

Figure 2.14 Effect of void ratio on shear modulus (modified from Kokusho 1980).

Figure 2.15 Effect of void ratio on normalized shear modulus (modified from Kokusho 1980).

Figure 2.16 Effect of void ratio on damping ratio (modified from Kokusho 1980).

2.4 Unsaturated Soils

Soils located above the groundwater table have negative pore-water pressures and these soils are referred as unsaturated soils. Soils desaturate through gravity flow, evaporation and evapotranspiration processes. Unsaturated soil behaviour cannot be explained using classical saturated soil mechanics. Therefore, this section describes unsaturated soil characteristics such as soil-water characteristic curve (SWCC), SWCC equations and measurements of SWCC. The dynamic properties of unsaturated soils are also presented.

2.4.1 Soil-Water Characteristics Curve (SWCC)

The soil-water characteristics curve (SWCC) expresses the amount of water in a soil versus soil suction. Commonly, three variables are used to define the amount of water in the soil (i.e., gravimetric water content, w , volumetric water content, θ_w and degree of saturation, S). Fredlund (2006) summarized the advantages and disadvantages of using each of these three variables to describe the amount of water present in the soil as shown in Table 2.4.

Table 2.4 Summary of advantages and disadvantages of the variables used in SWCC (modified from Fredlund 2006)

Variables	Advantages	Disadvantages
Gravimetric water content, w	Consistent with usage in classical soil mechanics Does not require volume measurement	Does not allow differentiation between change in degree of saturation Does not yield correct air-entry value when the volume change upon drying
Volumetric water content, θ_w	Commonly used in soil science Is the basic form that emerges in the derivation of transient seepage in unsaturated soils	Requires a volume measurement Rigorous definition requires a volume measurement at each soil suction
Degree of saturation, S	Most clearly defines the air-entry value (AEV) Appears to be the variable most closely controlling unsaturated soil behaviour	Requires a volume measurement Does not indicate when the soil undergoes volume change

Figure 2.17 shows a typical SWCC. There are two SWCCs, the main curve is a drying or desorption curve, whereas the other curve is a wetting or adsorption curve. At a given matric suction, the water content for the drying SWCC is always higher than the water content for the wetting SWCC. This phenomenon is known as hysteresis.

Figure 2.17 Definitions of terms for a typical SWCC
(modified from Fredlund and Xing 1994).

For drying SWCC, the matric suction where air starts to enter the largest pores in a soil is known as air-entry value (AEV) of the soil, $(u_a - u_w)_{AEV}$. Residual water content, θ_r , of a soil can be defined as the water content where a large increase in suction is required to remove additional water from the soil. The matric suction corresponding to the residual water content is called residual matric suction, $(u_a - u_w)_r$. Typical drying SWCCs for three major soil types are shown in Figure 2.18.

Figure 2.18 Typical drying SWCCs for sandy soil, silty soil, and clayey soil (after Fredlund and Xing 1994).

In the wetting process, water is adsorbed steadily by the dry soil which results in a decrease of matric suction until it reaches the water-entry value (WEV), $(u_a - u_w)_w$. The water-entry value is defined as the matric suction at which water first enters the pores of the soil in the wetting process. Beyond water-entry value, the water content increases significantly with decrement of matric suction until it reaches the residual air content, θ_a , that is the air content trapped in the soil in an occluded form and does not permit replacement by water. Hysteresis is observed whereby the wetting path of SWCC does not follow the drying path of the SWCC. The main causes for the difference in water content for drying and wetting processes are the non-uniformity of pore-size distribution of the soil, the presence of entrapped air in the soil, and the difference in contact angle between an advancing interface during the wetting process and a receding interface during the drying process (Fredlund and Rahardjo 1993).

2.4.2 Equations for SWCC

Most soils have unimodal SWCC which is a sigmoid. There are also SWCCs which are bimodal. Bimodal SWCCs are associated with soils having bimodal pore-size distribution (e.g. gap-graded and certain structured soils). As only certain soils have

bimodal SWCC, a representative unimodal SWCC is discussed in the following, instead. For the past five decades, numerous closed-form and empirical equations have been proposed to best fit the SWCC data. However, only some equations e.g. van Genuchten (1980), McKee and Bumb (1987) and Fredlund and Xing (1994) give a sigmoid (Leong and Rahardjo 1997). From all the proposed equations, Fredlund (2006) pointed out that there are three variables controlling the curve fitting procedures. The first variable and second variable are related to the air-entry value and rate of soil desaturation, respectively. The third variable allows the low suction range near the air-entry value to have a shape which is independent of the high suction range near residual water content.

Of the sigmoid equations, van Genuchten (1980) and Fredlund and Xing (1994) equations give good fits to the SWCC for a wide range of soil types (Leong and Rahardjo 1997). The van Genuchten (1980) equation is expressed as:

$$\theta_w = \theta_r + \frac{\theta_s - \theta_r}{\left[1 + a(u_a - u_w)^n\right]^m} \quad (2.19)$$

and the Fredlund and Xing (1994) equation is given as:

$$\theta_w = C(u_a - u_w) \frac{\theta_s}{\left\{ \ln \left[e + \left(\frac{u_a - u_w}{a} \right)^n \right] \right\}^m} \quad (2.20)$$

where a , n and m are curve-fitting parameters, and $C(u_a - u_w)$ is a correction factor given by

$$C(u_a - u_w) = 1 - \frac{\ln \left[1 + \frac{u_a - u_w}{(u_a - u_w)_r} \right]}{\ln \left[1 + \frac{1000000}{(u_a - u_w)_r} \right]} \quad (2.21)$$

where $(u_a - u_w)_r$ is matric suction corresponding to the residual volumetric water content, θ_r .

Leong and Rahardjo (1997) conducted a detailed review of the SWCC equation and concluded that Fredlund and Xing (1994) equation gave the best fit to the experimental data compared to the other equations reviewed. Leong and Rahardjo (1997) also suggested that $C(u_a - u_w)$ can be taken as unity.

2.4.3 Measurement of the SWCC

Soil suction can range from 0 to 1 GPa (Fredlund 2006). Over the years, numerous apparatuses have been developed to measure the SWCC. Typically, Tempe cell is used for low suction up to 100 kPa while pressure plate apparatus is used for suction up to 1500 kPa. For suction values greater than 1500 kPa, equilibrium in a fixed relative humidity environment method (i.e., vacuum desiccators) is employed (Agus et al. 2001; Fredlund 2006).

The principle of Tempe cell and pressure plate is based on the axis-translation technique proposed by Hilf (1956). Air pressure is applied to the soil specimen that is placed on a high-air-entry ceramic disk in a pressure chamber. The use of a high-air-entry disk allows the air phase to be separated from the water phase. During the test, the air pressure in the chamber is applied to a value above atmospheric pressure while the water pressure inside the compartment is usually kept at atmospheric (i.e., $u_w = 0$) by maintaining the water level inside the compartment through the use of burette that is filled with water. The difference between the air and water pressures is known as matric suction. Pore water is forced to drain out from the specimen through the pores of ceramic disk when air pressure is applied to the specimen. At equilibrium, the soil has a water content corresponding to the controlled matric suction. A soil-water characteristic curve is obtained by plotting the equilibrium water content of the specimen at different matric suctions with matric suction.

Various ASTM standards are available for determination of soil-water characteristic curve:

- 1) ASTM D5298-03 (2003). Standard Test Method for Measurement of Soil Potential (Suction) using Filter Paper.
- 2) ASTM D6836-02 (2008). Standard Test Methods for Determination of the Soil Water Characteristic Curve for Desorption using Hanging columnn, Pressure Extractor, Chilled Mirror Hygrometer, or Centrifuge.

ASTM D6836-02 (2008) supersedes ASTM D2325-68 (Reapproved 2000) for pressure plate method and ASTM D3152-72 (Reapproved 2000) for pressure membrane method.

The working principle for pressure plate and pressure membrane is almost the same. The main difference is that the pressure plate uses a ceramic porous disk while the pressure membrane uses a cellulose membrane for suction measurements. Furthermore, a qualifier is needed for pressure membrane and that is it is not easy to use. Therefore, in this study, pressure plate is preferred to pressure membrane for determining soil-water characteristic curve.

2.4.4 Dynamic Properties of Unsaturated Soils

To date, dynamic behaviour of unsaturated soil has not been investigated to a satisfactory extent. Hardin and Drnevich (1972a) reported that the effect of degree of saturation on shear modulus and material damping of cohesionless soils is small. The lack of influence of degree of saturation on cohesionless soil has also been documented by Richart et al. (1970). However, degree of saturation was identified as a very important parameter affecting the shear modulus of cohesive soils, as it is difficult to determine the effective stress of cohesive soils in unsaturated condition (Hardin and Drnevich 1972a). The shear modulus of cohesive soils increases rapidly with decreasing degree of saturation. At that time, the effect of degree of saturation on damping ratio was not fully established, but was believed to be a relatively important parameter affecting the damping ratio of cohesive soils.

Luh (1980) performed resonant column test to investigate the influence of matric suction on the very small shear strain modulus, G_{max} , and damping ratio, D_{min} , for kaolinite clays. It was found that the G_{max} increases significantly from the fully saturated state up to a critical matric suction (about 400-800 kPa). Beyond the critical matric suction, G_{max} tends to decrease with matric suction (Figure 2.19). Luh (1980) explained that the reduction of shear modulus is probably due to the formation of microcracks at high suctions. Similar conclusion was drawn by Marinho et al. (1995) for high plasticity clay using bender element tests. However, Mendoza et al. (2005) performed bender element tests and reported that G_{max} increases with decreasing degree of saturation for clay specimens and the clay specimens did not experience any microcracks formation. No critical degree of saturation was observed in their study. For damping ratio, the influence of matric suction is inconclusive due to the highly scattered data (Luh 1980). However, damping ratio seems to increase slightly with matric suction (Figure 2.20).

Figure 2.19 Influence of matric suction on very small strain shear modulus
(modified from Luh 1980)

Figure 2.20 Influence of matric suction on damping ratio (modified from Luh 1980)

Capillary effect on very small strain shear modulus, G_{max} , for silt was examined by Wu et al. (1984) using resonant column tests. In their study, they quantified the capillary effect by the degree of saturation instead of matric suction. The value of G_{max} increases with decrease of degree of saturation until a critical degree of saturation value which is around 5% to 20%. Thereafter, G_{max} decreases with decreasing degree of saturation (Figure 2.21). For completely dry or nearly fully saturated specimens, Wu et al. (1984) found that capillary effect has negligible influence on G_{max} . Similar trend was observed by Qian et al. (1993) for sands.

Mancuso et al. (2002) performed suction-controlled resonant column-torsional shear tests to study the very small strain behaviour of a silty sand. Matric suction ranging from 0 to 400 kPa and mean net stresses of 100, 200, and 400 kPa were used in their study. Mean net stress is defined as the difference of mean stress and pore-air pressure. It was found that the effect of matric suction on G_{max} is more pronounced in the matric suction range from 0 to about 200 kPa, where G_{max}

increases with matric suction. At higher matric suctions, G_{max} tends toward a threshold value depending on the mean net stress level as shown in Figure 2.22.

Figure 2.21 Effect of degree of saturation on shear modulus of Glazier Way silt at different mean effective stress (modified from Wu et al. 1984)

Figure 2.22 Influence of matric suction on very small strain shear modulus at mean net stress of 400 kPa (modified from Mancuso et al. 2002)

Soils with PI ranging from 0 to 44 have been tested by Hsu (2002) using cyclic simple shear tests to investigate the effect of degree of saturation on normalized shear modulus and damping ratio. However, no conclusive relationship between degree of saturation and normalized shear modulus and damping ratio can be established. The effects of degree of saturation on normalized shear modulus and damping ratio of non-plastic soils are presented in Figures 2.23 and 2.24, respectively. The effects of degree of saturation on normalized shear modulus and damping ratio of plastic soils are presented in Figures 2.25 and 2.26, respectively.

Zhang (2004) reported that it is difficult to investigate the effect of degree of saturation on shear modulus of soils at different shear strain level. He doubted that degree of saturation is an important parameter for normalized shear modulus, G/G_{max} . Zhang (2004) pointed out that if the degree of saturation is a function of mean effective stress, the dynamic behaviour of soils will not be affected by degree of saturation as mean effective stress effect has already been accounted for.

Figure 2.23 Effect of degree of saturation on normalized shear modulus of non-plastic soils (modified from Hsu 2002).

Figure 2.24 Effect of degree of saturation on damping ratio of non-plastic soils (modified from Hsu 2002).

Figure 2.25 Effect of degree of saturation on normalized shear modulus of plastic soils (modified from Hsu 2002).

Figure 2.26 Effect of degree of saturation on damping ratio of plastic soils
(modified from Hsu 2002).

From the literature, it can be seen that most of the dynamic properties of unsaturated soils are performed using resonant column and bender element tests. Hence, only very small strain behaviour (i.e., G_{max} and D_{min}) of unsaturated soils was investigated. A broader range of shear strain behaviour of unsaturated soils was investigated by Hsu (2002) using double specimen direct simple shear apparatus. In his study, degree of saturation was used as the influencing factor, instead of matric suction although it is well known that matric suction is a stress state variable. This study aims to investigate the small strain shear behaviour of unsaturated soils using a suction-controlled cyclic simple shear apparatus which will be presented in Chapter 6.

2.5 Residual Soils

Except for organic soils, soils can either be residual or transported in origin. Residual soils develop in situ while transported soils are surficial deposits which accumulate due to transportation of weathered residual soil by wind, water or snow. The characteristics of residual soils may differ from those of transported soils and are presented in Section 2.5.1 while their dynamic properties are briefly described in Section 2.5.2.

2.5.1 Characteristics of Residual Soils

Blight (1997) defines residual soils as soil-like material derived from in-situ weathering, and decomposition of rock which has not been transported laterally from its original location. The major agencies of weathering are attributed to physical, chemical and biological processes. In the processes of weathering, physical and chemical weatherings are closely interrelated where the rock is comminuted by physical process (i.e., erosion) and fresh surfaces with high permeability are exposed to chemical attack. Hydrolysis, a main chemical weathering process alters and breaks down the rock minerals to form more stable clay minerals through cation exchange and oxidation processes. Both physical and chemical actions are included in the biological weathering. In general, the amount of weathering depends principally on the climate and parent rock material as well as localized influences such as drainage, topography and vegetation.

According to Blight (1997), residual soils have distinct characteristics from those of transported soils in term of engineering properties. For instance, the conventional concept of a soil grain is not applicable to residual soils. It is due to the particles of residual soil usually consisting of aggregates or crystals of mineral matter that break down and become progressively finer if the soil is reconstituted. Borden et al. (1996) highlighted that the particle size distribution for a residual soil is more directly related to engineering behaviour rather than Atterberg limits. In addition, the permeability of a residual soil may not be related to its granulometry, an effect which the permeability of a transported soil is closely related to. On the contrary, permeability of residual soils are commonly governed by its micro and macro fabric, jointing and by superimposed features such as slickensiding, termite and other bio-channels.

It is recognized that the process of formation of a residual soil profile is complex and very difficult to model and generalize (Blight 1997; Leong et al. 2002). Therefore, a simplified weathering profile which contains materials of different grades are usually used to describe the degree of weathering and the extent to which the original structure of the rock mass is destroyed, varying with depth from the

ground surface. Typically, a residual soil profile with material grade classification system as illustrated in Figure 2.27 consists of three zones namely lower, intermediate and upper zone (Little 1969). The upper zone consists of completely weathered soils with grading of V and VI while the intermediate zone which is graded III and IV consists of moderately to highly weathered material. However, this material exhibits some features of the structure of parent rock. At the lower zone, it is classified as grade I and II with fresh rock and slightly weathered material.

Figure 2.27 Typical weathering profiles of residual soils (after Little 1969).

In Singapore, two thirds of the total land area is underlain by residual soils. These residual soils consist mainly of the Bukit Timah Granite residual soils and the Jurong Formation residual soils with each of these formations occupying approximately one-third of the land area. The Bukit Timah granite and Jurong Formation residual soils are found in central and western parts of Singapore, respectively, as shown in Figure 2.28. As a product of weathering, these residual soils generally exist above groundwater level and therefore are unsaturated in nature (Yong et al. 1985).

Figure 2.28 Geological map of Singapore (modified from PWD 1976).

In term of composition, the variability of the grain size distribution of both residual soils from Bukit Timah Granite and Jurong Formation is shown in Figures 2.29 and 2.30, respectively (Leong et al. 2002). The wide band of the grain size distributions are observed as they are subjected to different degree of weathering and mineral composition of the parent rock. For index properties, Leong et al. (2002) had plotted the Atterberg limit for both Bukit Timah Granite and Jurong Formation residual soils on the plasticity chart as shown in Figure 2.31 and 2.32, respectively. It was found that most of the data for Bukit Timah Granite residual soils plot below the A-line, signifying that they consist mainly of silt. Similarly, this finding agrees well with those obtained by Poh et al. (1985) who had categorized the Bukit Timah Granite residual soils into two groups with Group I – clayey silt, and Group II – silty clay. On the other hand, the Atterberg limits of the Jurong Formation residual soils plot mostly above A-line signifying that the dominant clay minerals for this formation are kaolinite and illite.

For unsaturated properties of Singapore residual soils, Agus et al. (2001) established an envelope of soil-water characteristics curves for both Bukit Timah Granite and Jurong Formation using Fredlund and Xing (1994) equation (Equation 2.20) with correction factor equal to unity as suggested by Leong and Rahardjo

(1997). It was found that the average fitting parameters for a , n , and m are 36 kPa, 0.565, and 1.147, respectively, for Bukit Timah Granite residual soils as shown in Figure 2.33. For Jurong Formation residual soils, the average fitting parameters for a , n , and m are 299 kPa, 0.554, and 1.869, respectively, as shown in Figure 2.34.

Figure 2.29 Grain size distribution envelope for Bukit Timah Granite residual soils (after Leong et al. 2002).

Figure 2.30 Grain size distribution envelope for Jurong Formation residual soils (after Leong et al. 2002).

Figure 2.31 Plasticity chart for Bukit Timah Granite residual soils (after Leong et al. 2002).

Figure 2.32 Plasticity chart for Jurong Formation residual soils (after Leong et al. 2002).

Figure 2.33 SWCC for Bukit Timah Granite residual soils (after Agus et al. 2001).

Figure 2.34 SWCC for Jurong Formation residual soils (after Agus et al. 2001).

2.5.2 Dynamic Properties of Residual Soils

Compared to the plentiful dynamic properties of clays and sands in the literature, there is little research works on dynamic properties of residual soils. Nevertheless, some investigations of dynamic properties of residual soils have been reported by Borden et al. (1996) and Macari and Hoyos (1996). Borden et al. (1996) pointed out that engineering properties of residual soils differ from those of transported sands, silts and clays. Therefore, the empirical correlations, and design parameters of “conventional soils” may not be applicable for residual soils.

The influence of mean effective stress σ'_m has been investigated by Borden et al. (1996) using resonant column and cyclic torsional shear tests. From their study, it was found that the very small strain shear modulus, G_{max} , increases with σ'_m as shown in Figure 2.35. However, the influence of σ'_m was found to be less pronounced on damping ratio D than on shear modulus G .

Figure 2.35 Influence of mean effective stress on G_{max} for Piedmont residual soils (modified from Borden et al. 1996).

Number of loading cycles was found to have no significant effect on shear modulus of Piedmont residual soils (Borden et al. 1996) for shear strain range from 0.001%

to 0.098%. However, Leong et al. (2003) conducted cyclic triaxial tests to determine the dynamic properties of Singapore residual soils for shear strain range from 0.005% to 1% and found that the shear modulus and damping ratio decreases and increases slightly with number of loading cycles, respectively, as shown in Figure 2.36 and 2.37

Figure 2.36 Influence of number of loading cycles on shear modulus for Singapore residual soils (after Leong et al. 2003).

Figure 2.37 Influence of number of loading cycles on damping ratio for Singapore residual soils (after Leong et al. 2003).

Macari and Hoyos (1996) studied Puerto Rico residual soils using resonant column apparatus. They concluded that the degree of weathering affects the dynamic properties for both undisturbed and reconstituted residual soils. For highly weathered (Grade IV) residual soils (i.e., deep from the ground surface), the very small strain shear modulus is higher than those of completely weathered (Grade V and VI) residual soils whereas the very small strain damping ratio is lower than those of completely weathered (Grade V and VI) residual soils. The effect of degree of weathering decreases with increasing effective confining pressure for both very small shear strain shear modulus and damping ratio.

In general, the shear modulus of Piedmont residual soil decreases with increase of shear strain and the linear threshold shear strain was observed to be around 0.002% (Borden et al. 1996). Macari and Hoyos (1996) reported that the linear threshold shear strain of Puerto Rico residual soils was 0.004%. Volumetric threshold shear strain was not determined in both studies.

From the literature, it can be seen that the dynamic properties of residual soils are scarce. Since formation of residual soils is largely dependent on weathering condition, there is a need to investigate dynamic properties of residual soil in Singapore to establish the factors affecting them.

2.6 Review of Cyclic Simple Shear Tests

This section describes the historical development of the cyclic simple shear apparatus and its working principle. Capabilities of cyclic simple shear apparatuses are presented and problems associated with the cyclic simple shear tests are also discussed.

2.6.1 Historical Development of Cyclic Simple Shear Apparatus

The earliest recorded use of simple shear apparatus can be attributed to Swedish Geotechnical Institute (SGI) as described by Kjellman (1951). At that time, the shear strength of clay in Sweden was tested using the SGI-type simple shear apparatus. The SGI simple shear apparatus test a cylindrical specimen, 20 mm in

height and 60 mm in diameter, placed between a lower and an upper grooved plates. To prevent a change of diameter (i.e., radial deformation), the specimen is confined with rubber hose and surrounded by aluminium rings as shown in Figure 2.38. Vertical load is applied by means of lead weights on the top of a piston while the horizontal force is imposed by weights suspended from a wire running over a pulley. The corresponding horizontal displacement of the upper grooved plate is measured by a dial gauge. The SGI-simple shear apparatus has some advantages over other direct shear apparatus (Kjellman 1951): the deformation of the specimen is uniform and constant volume test can be performed with high degree of accuracy.

Figure 2.38 SGI routine simple shear apparatus (after Kjellman 1951).

Roscoe (1953) introduced a Roscoe-type simple shear apparatus enabling the sample to be tested in the shear box as depicted in Figure 2.39. The cuboidal specimen, 60 mm in length and width and 22 mm in height, is confined by rigid sidewalls in the hope to provide a more uniform shear strain of the specimen. The sides of the apparatus are fixed relative to the base and cap while the ends are attached by hinges to base and cap. The top cap is parallel to the base and can move vertically. The base is supported on a low-friction bearing way which can move horizontally. Since the specimen is not enclosed in a pressure chamber, the test is

carried out under atmospheric pressure condition. The Roscoe-type simple shear apparatus is only suitable for rapid undrained tests, but it could easily be adapted for other types of test such as drained test (Roscoe 1953).

Figure 2.39 Roscoe-type simple shear devices (after Roscoe 1953).

Bjerrum and Landva (1966) developed the Norwegian Geotechnical Institute (NGI) direct simple shear testing apparatus. The specimen size is 80 mm in diameter and 10 mm in height. Based on the SGI design, modification was made by confining the cylindrical specimen with rubber membrane reinforced by spiral wires. The use of spiral wires reinforced rubber membrane is to restrict the specimen from bulging (i.e., radial strain) under the influence of vertical loading. The top cap is fixed to the specimen carriage which prevents the specimen from tilting. The vertical load is applied to the top cap of the specimen through the roller bearings while the horizontal load is imposed to the top cap of the specimen by a variable speed motor. Similar to Roscoe-type simple shear apparatus, the specimen is not enclosed in a pressure chamber and therefore the test was carried out under atmospheric pressure condition. The NGI-type apparatus is best suited to tests where pore-water pressure is less than or equal to atmospheric pressure. This is because pore-water pressure larger than atmospheric pressure tends to cause bulging of the specimen although the membrane is reinforced by wires (Duncan and Dunlop 1969). The NGI-type direct simple shear device is shown in Figure 2.40.

Figure 2.40 NGI direct simple shear apparatus (after Bjerrum and Landva 1966).

The difference between the NGI and Roscoe-type simple shear apparatuses has been described by Duncan and Dunlop (1969). The former used a cylindrical specimen confined in a reinforced rubber membrane while the latter used a cuboidal specimen enclosed by square metal rings all around. Figure 2.41 shows the difference between the NGI and Roscoe-type simple shear apparatus. In the early design of simple shear apparatus as described by Kjellman (1951), Roscoe (1953) and Bjerrum and Landva (1966), the NGI-type simple shear apparatus appeared to be most popular due to its simplicity and is the design basis of modern cyclic simple shear apparatus (Finn et al. 1982).

Figure 2.41 Difference between NGI and Roscoe-type simple shear apparatus (after Duncan and Dunlop 1969).

In early designs of simple shear apparatus, all the tests were performed only in monotonic condition. Silver and Seed (1971a) modified the NGI-type simple shear apparatus by adding a lever arm in the apparatus as shown in Figure 2.42 to perform small shear strain amplitude cyclic test. The arm which is connected through a flexible coupling to the base plate is actuated by a cam. The applied horizontal displacement is varied by moving the cam location to change the lever arm ratio while the waveform shape and loading frequency are varied by changing the shape of the cam and speed of the cam drive motor, respectively. The other features of the modified NGI-type simple shear apparatus include the base which is fixed to a moving base plate supported by frictionless bearings while the top is rigidly held by the loading head. Simple shear deformation is induced by the relative movement between the base plate and the loading head.

Figure 2.42 Schematic diagram of cyclic simple apparatus
(after Silver and Seed 1971a).

Since then, many other simple shear apparatuses have been developed by modifying the classical simple shear apparatuses as mentioned above. Over the years, dynamic properties of soils obtained by simple shear apparatuses based on NGI-design have been reported by numerous researchers (Silver and Seed 1971a; Doroudian and Vucetic 1995; Miller et al. 1996; Evgin and Fakharian 1998; Mohajeri and Towhata

2003; Hsu and Vucetic 2004). On the other hand, a number of researchers (Peacock and Seed 1968; Duncan and Dunlop 1969; Finn et al. 1971, 1982; Bhatia 1981) conducted sand liquefaction study using Roscoe-type simple shear apparatus.

Ishihara and Yamazaki (1980) developed a bi-directional simple shear apparatus as opposed to the classical simple shear apparatuses which are uni-directional shearing devices. The bi-directional simple shear device (Figure 2.43) consists of three parts, namely, an assemblage for specimen placement; a horizontal load carriage which can apply loading in two perpendicular directions; and a vertical loading system. The disk-shaped specimen is confined within a rubber membrane and surrounded by a stack of Teflon coated annular plates. Horizontal loads are applied at the top cap by two perpendicular pneumatic actuators that can be operated independently. On the other hand, vertical loads are applied upwards by using a pneumatic actuator located underneath the specimen. Similarly, a multi-directional direct simple shear apparatus as depicted in Figure 2.44 has been developed by DeGroot et al. (1993).

Figure 2.43 Bi-directional simple shear apparatus
(after Ishihara and Yamazaki 1980).

Figure 2.44 Multi-directional simple shear apparatus

(after DeGroot et al. 1993).

More recently, Doroudian and Vucetic (1995) developed a double specimen direct simple shear (DSDSS) device which purportedly eliminates the friction of the load-transfer mechanism by reducing the effects of mechanical compliance. At very small strain testing, the shear stress induced by the roller-bearing friction can be quite large compared to the shear resistance of soil, and may even overshadow the shear resistance of soil. The use of two soil specimens and direct application of horizontal load to the middle cap between them eliminates the roller-bearing friction problem (Figure 2.45). The mechanical compliance of the apparatus is minimal as the body of the apparatus was made of thick stainless steel components. Vertical load is applied pneumatically via a piston. The magnitude of the vertical load is adjusted by an air pressure regulator and measured by a load cell. The vertical settlement of the specimen is measured by an LVDT bridged between the top plate of the device and the piston rod, which is connected directly to the top cap of the specimen. Horizontal cyclic load is measured by a load cell connected directly to the middle cap. For very small strain testing, in order to avoid the vibration induced by the pneumatic or hydraulic cyclic loading system, the horizontal load is applied manually by a micrometer connected to the horizontal piston via a soft spring, which provides a better control of applied small loads and displacements. The horizontal displacement is measured by means of a non-contact

proximity transducer bridged between the bottom cap and the middle cap. The non-contact proximity transducer is capable of measuring a displacement of 0.0001 mm or even smaller. With such a small displacement measurement, the transducer was calibrated using a high-precision micrometer. In addition, the output voltage signal of the proximity transducer was greatly amplified such that the increment of the displacement corresponding to a digital voltage step of the data acquisition system of less than 0.00001 mm. The horizontal load cell has a precision of 0.0001 N and it was calibrated for loading and unloading conditions. However, the method of calibration was not reported. The validity of the simple shear apparatus has been examined by performing four undrained cyclic tests on kaolinite clays (Doroudian and Vucetic 1995).

Figure 2.45 Configurations of DSDSS (after Doroudian and Vucetic 1995).

For very small strain, below the order of 0.001%, the result of simple shear test is greatly affected by the vibration transmitted through the laboratory floor into the apparatus. The stress-strain curve loops could not be clearly recorded, and only an

approximate range of shear modulus could be obtained. The following three suggestions have been given to improve the performance of DSDSS (Doroudian and Vucetic 1995):

- 1) DSDSS apparatus should be placed on a vibration isolation table to minimise the vibration problem,
- 2) A high-precision proximity transducer should be used to measure the horizontal displacement,
- 3) A better signal conditioner which has a more adequate frequency range and low pass filter should be used.

Despite the versatility of the DSDSS in measuring small-strain shear modulus and damping ratio, Nwaiwu and Osinubi (2005) doubted the manual-controlled DSDSS would yield accurate results as accuracy of the test result is largely dependent on the skill of the operator. Consequently, random testing error would occur and lead to even more uncertainties in the measured parameters.

2.6.2 Summary

From the literature review, state-of-the-art cyclic simple shear apparatus as presented by Doroudian and Vucetic (1995) appears to be the most successful cyclic simple shear apparatus in capturing small strain cyclic soil behaviour (i.e., Lanzo et al. 1997; Vucetic et al. 1998; Lanzo and Vucetic, 1999; Matešić and Vucetic 2003; Vucetic and Tabata 2003). However, back-pressure saturation is not possible for DSDSS. Moreover, Nwaiwu and Osinubi (2005) doubted the manually operated DSDSS would yield accurate results as accuracy will depend on the skill of the operator. In this study, a commercial actuator-operated cyclic simple shear apparatus manufactured by GCTS is used. Back-pressure saturation is possible for GCTS cyclic simple shear apparatus as cell pressure, back pressure can be applied independently while pore-water pressure can be measured at the boundary of the soil specimen. Table 2.5 summarizes the parameters investigated in cyclic simple shear test and Table 2.6 presents the capabilities of existing cyclic simple shear apparatuses.

Table 2.5 Experimental parameters investigated in cyclic simple shear test

Reference	Specimen size		Material tested	Test condition	Effective vertical stress σ'_v (kPa)	Number of loading cycles, N	f (Hz)	Waveform
	Diameter/Length (mm)	Height (mm)						
Finn et al. (1971)	50.8	3.2-25.4	Fully saturated sand	Undrained	200 - 600	1000	2	Square
Silver and Seed (1971a,b)	80	20	Dry sand	Drained	25 - 200	300	1	Triangular
Moussa (1975)	80	15	Dry sand	Drained followed by undrained	200	400, 4000	Estimated 0.08	Similar to trapezoidal
Ohara and Matsuda (1988)	75	20	Fully saturated kaolinite clay	Undrained followed by drained	50 - 300	10, 50, 100, 200	0.5	Sinusoidal
Chu and Vucetic (1992)	79.5	22	Partially saturated compacted clay	Drained	550	80	0.3-0.4	Not reported
Boulanger et al. (1993)	102	25.4	Fully saturated sand	Undrained	200	Until failure	0.2	Not reported
Doroudian and Vucetic (1995)	66	20	Fully saturated kaolinite clay	Undrained	30 - 150	≥ 5	Not reported	Not reported
Miller et al. (1996)	101.6	25.4	Fully saturated sand	Undrained	69	10	0.5	Sinusoidal
Whang et al. (2004)	102	23	Partially saturated compacted soils	Drained	100	25	1	Sinusoidal
Duku et al. (2007)	102	23	Dry sand	Drained	100	Not reported	Not reported	Not reported
Spikula and Garmon (2008)	63.5	19	Saturated Malco soils	Undrained	50 to 580	30	0.1 and 1	Sinusoidal
Jafarzadeh and Sadeghi (2009)	70	22	Saturated and partially saturated sands	Undrained and drained	50 to 150	1 to 200	0.5	Sinusoidal

Table 2.6 Capabilities of cyclic simple shear apparatuses

Reference	Stress or strain-controlled	Ability to apply cell pressure, σ_3	Shear load precision (N)	Shear displacement precision (mm)	Shear strain measurement (%)
Finn et al. (1971)	Stress-controlled	No	Not reported	Not reported	Not reported
Silver and Seed (1971a, b)	Strain-controlled	No	Not reported	Not reported	0.01 – 0.5
Moussa (1975)	Stress-controlled	No	Not reported	Not reported	Not reported
Ohara and Matsuda (1988)	Strain-controlled	No	Not reported	Not reported	0.05 – 3.0
Chu and Vucetic (1992)	Strain-controlled	No	Not reported	Not reported	0.008 – 4.6
Boulanger et al. (1993)	Stress-controlled	Yes	± 3.5	± 0.006	Not reported
Doroudian and Vucetic (1995)	Stress-controlled	No	0.0001	0.00001 ^a	0.003 ^b – 0.02
Miller et al. (1996)	Stress-controlled	No	Not reported	0.0005	0.002 – 0.03
Whang et al. (2004)	Strain-controlled	Yes	Not reported	Not reported	0.1 – 1.0
Duku et al. (2007)	Strain-controlled	Yes	Not reported	Not reported	Not reported
Spikula and Garmon (2008)	Strain-controlled	No	Not reported	Not reported	> 0.1
Jafarzadeh and Sadeghi (2009)	Strain-controlled	No	Not reported	Not reported	1 – 1.5

^a After amplification of the output voltage signal of proximity transducer.

^b Both G_s and D were evaluated with confidence.

2.6.3 Problems Associated with Cyclic Simple Shear Tests

It is recognized that no laboratory testing equipment is capable of duplicating the in-situ stress state under seismic event (Lee 1976). Although cyclic simple shear test can, arguably, simulate in-situ soil element during dynamic excitation closely, it has its shortcomings.

The major drawback of cyclic simple shear that has been discussed for the past four decades is non-uniform stresses and strains across the specimen. Non-uniform stress and strain distributions across the specimen are attributed to lack of complementary shear stresses on the lateral boundaries (i.e., vertical faces). The lack of complementary shear stresses on the lateral boundaries results in cap rocking motions due to unbalanced moment (Miller et al. 1996). The difference between pure shear and simple shear state of stresses is depicted in Figure 2.46. It can be seen that significant stress and strain are concentrated along the edges of the cyclic

simple shear specimen which contribute to non-uniform stress distributions at the boundaries.

Figure 2.46 Simple shear and pure shear stress state
(after Duncan and Dunlop 1969).

Non-uniform stresses in cyclic simple shear specimen are partly due to the apparatus itself. Finn et al. (1971) pointed out that adequate friction must be present between soil specimen and both top and bottom platens in order to have a more uniform stress under simple shear condition. In order to prevent slippage between the specimen and apparatus, sand grains were epoxied to the cap and base of the simple shear apparatus (Silver and Seed 1971a, b).

The effect of insufficiently roughened top and bottom platens has been examined by testing striped plasticine specimens as shown in Fig 2.47 (Finn et al. 1971). The specimens deformed relatively uniform (i.e., no relative motion) when sheared under rough plates conditions as shown in Figure 2.47(b). On the other hand, specimens deformed less uniformly when they were placed on smooth plates. It was observed that the centre part of the specimen seems to resist the shear load only as it deformed much more than near the specimen end surfaces as shown in Figure 2.47(c). This further implies that the apparent strength of soil specimen tested under this condition would be much less than the actual strength of the soil specimen as shown in Figure 2.48.

Figure 2.47 Effect of plate roughness (a) un-deformed state; (b) deformed under rough both top and bottom plates; (c) deformed under smooth plates (after Finn et al. 1971).

Figure 2.48 Effect of plate roughness in simple shear apparatus. (modified from Finn et al. 1971).

The consequences of boundary slippage at the top and bottom of a simple shear specimen by using isotropic and elastic analyses have been reported by Prevost and Hoeg (1976). It was found that the distributions of shear and normal forces are greatly disturbed by the slippage as there is coupling of shear and normal strains in soil, which is a result from shear or normal stresses and vice versa. Similar problem has been raised by Miller et al. (1996) who suggested that the end caps must be roughened in order to provide better coupling between end caps and specimen.

Other issues arising from the cyclic simple shear apparatus are the compliance of the machine loading system and carriage system friction (Boulanger et al. 1993; Miller et al. 1996; Whang 2001; Seo 2008). The friction induced by the roller-bearing can be quite significant especially for small shear strain testing (Doroudian and Vucetic 1995). Consequently, horizontal shear stress obtained from the load cell is the addition of shear resistance of soil and roller-bearing. When stiff materials are tested, an extremely rigid loading system is needed to prevent rocking effect which will result in non-uniform stress concentrations in the specimen. Due to the above outstanding issues, a tri-post frame with high performance track bearing has been used by Duku et al. (2007) in order to reduce the rocking effect induced by the unbalanced moment as a result of lack of complementary shear stresses in simple shear condition.

Several researchers investigated stress uniformity in the simple shear test theoretically. Based on a three-dimensional elastic model, Lucks et al. (1972) showed that 70% of the cylindrical specimen in the NGI apparatus is uniformly stressed under simple shear loading. Shen et al. (1978) performed a three-dimensional finite element analysis of a cyclic simple shear test to investigate the effect of height to diameter ratio and membrane stiffness on stress-strain distribution. Based on the study, it was found that the shear strain distribution becomes more uniform as the specimen height to diameter ratio decreases and the amount of wire-reinforcement in the membrane increases. Despite the non-uniform stress-strain distribution, they concluded that cyclic simple shear test is still acceptable in practical engineering analyses.

Effects of specimen height to diameter ratio were also investigated by Vucetic and Lacasse (1982) and Amer et al. (1987). Vucetic and Lacasse (1982) found that specimen height to diameter ratio ranging from 0.14 to 0.33 has no significant effect on the static stress-strain behaviour and further concluded that direct simple shear test gives reasonably useful results to define stress-strain behaviour of soil in engineering practice. However, Amer et al. (1987) reported that except for specimens with height to diameter ratio of 0.125 or less, other specimen sizes seem to have a definite effect on the shear modulus and damping ratio of the dry sand tested.

Large-scale simple shear tests on a shake table were performed by De Alba et al. (1976) to investigate non-uniform stress condition of small-scale cyclic simple shear test specimen with the assumption that stress non-uniformities in shake table tests are minimal. The long thin soil specimen in the large-scale simple shear table was 2.3 m x 1.1 m x 0.1 m (i.e., length x width x height). In contrast, the specimen height to diameter ratios used for the small-scale cyclic simple shear test ranges from 0.25 to 0.33. It was found that the data obtained from the carefully conducted small-scale simple shear test approximate closely to those obtained from the shake table tests. Base on the consistency of soil response from these two specimen configurations (i.e., large-scale and small-scale simple shear tests), Seed (1976) pointed out that the errors due to stress concentrations in the small-scale cyclic simple shear tests were negligibly small or they were counter-balanced by some other features of the test and therefore it was concluded that the cyclic simple shear test can give reasonably useful results in engineering practice.

Despite the uncertainties and shortcomings of the cyclic simple shear test, several researchers suggested that for most practical engineering analyses, simple shear test is able to provide reasonable approximation of in-situ deformations of soils subjected to vertical propagating shear waves (De Alba et al. 1976; Shen et al. 1978; Vucetic and Lacasse 1982).

2.7 Summary

This chapter provides an overview of laboratory cyclic testing to determine shear modulus and damping ratio of soil. The tests include resonant column test, cyclic simple shear test, cyclic triaxial test and cyclic torsional shear test. The major factors affecting the shear modulus and damping ratio have been discussed. This chapter also describes the characteristics of unsaturated soil and its dynamic properties. The distinct characteristics of residual soil and its dynamic properties have been discussed in detail. Extensive literature review on cyclic simple shear test has also been presented in this chapter.

Chapter 3

Evaluation of Cyclic Simple Shear Apparatuses

3.1 General

There are many cyclic simple shear apparatuses available commercially: GCTS, GDS, Geocomp, Geonor, Geotest, Seiken, and Wykeham Farrance. In Nanyang Technological University (NTU), cyclic simple shear apparatus manufactured by GCTS was commissioned in 2002 and primarily used in this study. To evaluate the performance of GCTS cyclic simple shear apparatus, GDS cyclic simple shear apparatus owned by a commercial soil testing laboratory, Geolab Services, was used for comparison. The instrumentations and characteristics of the two cyclic simple shear apparatuses are described. The cyclic simple shear properties of a standard sand were “calibrated” against the results from published literature.

3.2 Cyclic Simple Shear Apparatus

Figures 3.1 and 3.2 show the schematic drawing of the GCTS and GDS cyclic simple shear apparatuses, respectively. Both the cyclic simple shear apparatuses follow the NGI-type simple shear apparatus design and uses cylindrical specimens. For GCTS simple shear apparatus, lateral support is provided via confining water pressure whereas GDS simple shear apparatus uses a stack of Teflon-coated rings (each Teflon-coated ring has a thickness of 10 mm). As plain rubber membrane is used in the GCTS cyclic simple shear apparatus, back-pressure saturation is possible for fully saturated soils. The use of plain rubber membrane in a constant height and constant volume cyclic simple shear test has been justified by a number of researchers (Tatsuoka and Silver 1981; Boulanger et al. 1993; Sitar and Salgado 1994; Rau and Sitar 1998; Mao and Fahey 2003; and Bray and Sancio 2006). For GDS cyclic simple shear apparatus, back-pressure saturation is not possible because the plain rubber membrane is enclosed within a stack of Teflon-coated rings. To prevent slippage between the specimen and the platens, GCTS cyclic simple shear apparatus has 2.5 mm high and 1 mm thick blades at 4.5 mm spacing along the platens (Figure 3.3) while GDS cyclic simple shear apparatus has 2.5 mm width

grooves along the platens (Figure 3.4). The platens for both cyclic simple shear apparatuses are 70 mm in diameter.

Both cyclic simple shear apparatuses are equipped with normal and shear load actuators and computer interface unit with servo control, data acquisition system with signal conditioning and personal computer. For GCTS cyclic simple shear apparatus, the system is operated by a DOS-based program (GCTS Version 7.01) whereas the system for GDS cyclic simple shear apparatus is operated by a WINDOW-based program (GDSLAB). The normal and shear actuators are controlled hydraulically for GCTS cyclic simple shear apparatus, and mechanically for GDS cyclic simple shear apparatus. The GCTS cyclic simple shear apparatus is equipped with a chamber where cell pressure can be supplied and controlled pneumatically whereas GDS cyclic simple shear apparatus does not have a chamber and thus cell pressure application is not possible.

For GCTS cyclic simple shear apparatus, additional instrumentations are added. These include linear variable differential transformer (LVDT) for internal vertical displacement measurement, proximity transducers for horizontal displacements of top platen and middle height of soil specimen. The instrumentations of the two cyclic simple shear apparatuses are summarized in Table 3.1.

3.3 Test Materials

Tests were performed on dry Ottawa 20-30 sand. Ottawa 20-30 sand is a commercially available “standard” uniform silica sand. Basic soil properties such as grain-size distribution and specific gravity were obtained in accordance with ASTM D 422-63 (2007) and ASTM D 854-06 (2006), respectively. The maximum and minimum density of the tested sand were determined in accordance with ASTM D 4253-00 (2006) and ASTM D 4254-00 (2006), respectively. Grain size distribution curve of Ottawa 20-30 sand is shown in Figure 3.5. Table 3.2 summarizes the basic soil properties of Ottawa 20-30 sand.

Figure 3.1 Overview of GCTS cyclic simple shear apparatus.

Figure 3.2 Overview of GDS cyclic simple shear apparatus.

Figure 3.3 Top and bottom platens of GCTS cyclic simple shear apparatus.

Figure 3.4 Top and bottom platens of GDS cyclic simple shear apparatus.

Chapter 3 Evaluation of Cyclic Simple Shear Apparatuses

Table 3.1 Instrumentation comparison between GCTS and GDS cyclic simple shear apparatuses

Parameters		GCTS	GDS
Power unit of servo actuator		Electro-hydraulic	Electro-mechanical
Lateral support		Confining water	Teflon-coated rings
Axial load (kN)		±22.24 (SW10-5K(100611A))	±5
Axial displacement (mm)	High-range	±25 (PR-750-1000)	±20
	Low-range	±2.5 (RDP-D5/100W)	±5
Shear load (kN)	Thrust	-	±5
	Low-range	±2.24 (SW10-500(106859A))	±5
Shear displacement (mm)	High-range	±25 (PR-750-1000)	±20
	Low-range	±1.25 (PR-750-050)	±1.25
Top platen horizontal displacement (mm)		±4 (AEC-55MS-L-PU-20)	-
Horizontal displacement of middle height of specimen (mm)		±4 (AEC-55MS-L-PU-20)	-
Pore-water pressure (kPa)		1000 (WF 17060)	-

Figure 3.5 Grain size distribution of Ottawa 20-30 sand.

Table 3.2 Basic soil properties of the Ottawa 20-30 sand

Soil Properties	Ottawa sand 20-30
USCS	SP
Specific gravity, G_s	2.65
Grain size analysis	
D_{60} (mm)	0.83
D_{30} (mm)	0.78
D_{10} (mm)	0.69
Coefficient of uniformity, C_u	1.20
Coefficient of curvature, C_c	1.06
Density test analyses	
e_{max}	0.69
e_{min}	0.48

USCS: Unified Soil Classification System

3.4 Specimen Preparation Method

Air pluviation technique was used in this study to prepare the dry sand specimens. This technique was adopted for both the GCTS and GDS cyclic simple shear tests. Sands were prepared directly in both the cyclic simple shear apparatuses. A plain rubber membrane was sealed to the bottom platen by O-rings and held rigidly in a split mould for GCTS cyclic simple shear apparatus and a stack of rings for GDS cyclic simple shear apparatus that was placed on the specimen bottom platen, thus forming a mould for the sand. A pre-weighed quantity of sand was poured into a funnel. Air pluviation was done by raising the tip of the funnel in a swirling motion from the bottom of the specimen platen. To avoid significant segregation of the sand grains, great care was taken to ensure that the sand was pluviated continuously. Sand pluviation was done in three layers. Each layer was lightly tamped to a predetermined height with a small tamper to achieve the required void ratio. The amount of energy used in tamping process was carefully controlled at 15 blows per layer. All sand specimens were prepared at a void ratio of approximately 0.60 (relative density of 45%).

After completion of sand specimen preparation, the top platen was positioned on the specimen surface. The membrane was carefully pulled over the top platen and secured with O-rings, before the split mould for GCTS cyclic simple shear apparatus was dismantled. A small normal seating stress of 5 kPa was applied onto

the specimen, following the suggestion by ASTM D6528-07 (2007). The dimensions of the specimen were then measured and the initial void ratio was calculated. In this study, all the specimens have a diameter of 70 mm and a height of about 20 mm. The height-to-diameter ratio, h/d is 0.29, conforming to ASTM D6528-07 (2007) for monotonic direct simple shear testing which suggests that the ratio shall not exceed 0.40. In past researches, cyclic simple shear tests were conducted with h/d ranging from 0.19 to 0.30 (Silver and Seed 1971a; Moussa 1975; Ōhara and Matsuda 1988; Doroudian and Vucetic 1995; Miller et al. 1996; Whang et al. 2004).

3.5 Testing Procedure

The test procedure for dry sands consisted of two stages: consolidation and shearing. For GCTS cyclic simple shear apparatus, after the cell chamber was fully filled with distilled water, isotropic consolidation was performed by ramping the cell pressure and vertical load up linearly to the desired consolidation pressure. For GDS cyclic simple shear apparatus, vertical load is ramped up linearly to the desired consolidation pressure while lateral strain is prevented by the stack of Teflon-coated rings.

Shearing was performed using strain-controlled mode. Drained constant vertical load was chosen in this study, because in many field conditions, the pore-air pressure is drained for dry sandy materials. Multi-stage test on a single specimen was adopted in this study to eliminate the variation in multi-specimen tests. A sinusoidal loading at a frequency of 0.5 Hz was applied at each shear strain level, γ_c . In each loading step, γ_c was increased with vertical deformation permitted (i.e., in each step the specimen has different initial void ratio, e , from the original void ratio, e_0). Ten cycles of controlled cyclic shear strain were applied to the soil specimen at each loading step following the recommendation of Das (1993) that dynamic properties of soils determined at the fifth cycle are likely to provide fairly reasonable value for practical use. Furthermore, Das (1993) reported that the number of significant strain cycles is likely to be less than 20. The shear strain levels range from about 0.01% to 1% for both the GCTS and GDS cyclic simple

shear apparatuses. A sampling rate of 100 points/cycle was logged in the data acquisition system during cyclic loading. This sampling rate conforms to ASTM D3999-91 (2003) for cyclic triaxial testing which suggests that the minimum sampling points per cycle is 40.

3.6 Comparison of Anisotropic Cyclic Simple Shear Test with Other Isotropic Tests

Finn et al. (1971) pointed out that effective vertical stress applied in the simple shear test is not a good index of the confinement of the soil. If the vertical stress on a natural field deposit is σ'_v , the horizontal stress, σ'_h , can be equated as follows:

$$\sigma'_h = K_0 \sigma'_v \tag{3.1}$$

where K_0 is the coefficient of earth pressure at rest. To enable comparison to be drawn meaningfully between cyclic simple shear test and other isotropic tests (Park and Silver 1975; Cavallaro et al. 2003), mean effective stress, σ'_m , for the simple shear test can be used and computed as follows:

$$\sigma'_m = \frac{(\sigma'_v + 2K_0 \sigma'_v)}{3} \tag{3.2}$$

To estimate K_0 , the empirical correlation, $K_0 = 1 - \sin \phi'$ proposed by Jaky (1948) is employed. The ϕ' value was estimated by monotonic shearing of the sand at the end of the cyclic loading. Table 3.3 shows σ'_m at the end of consolidation stage for both cyclic simple shear apparatuses.

Table 3.3 Mean effective stress at the end of consolidation stage

σ'_v (kPa)	σ'_m (kPa)	
	GCTS - isotropic	GDS - anisotropic
25	25 (GCTS 25 kPa)	17 (GDS 17 kPa)
100	100 (GCT 100 kPa)	70 (GDS 70 kPa)
200	200 (GCTS 200 kPa)	140 (GDS 140 kPa)

Note: characters in the parenthesis indicate test name.

3.7 Equipment Testing

Equipment testing is of crucial importance in soil dynamic testing. Khan et al. (2011) reported that the measurement of low-strain properties (wave velocity and damping ratio) of geomaterials is affected by apparatus-generated delays, coupling of transducers, and wave reflections. Due to apparatus-generated damping, D_{app} , Stokoe et al. (1995) suggested that the damping ratio of soil obtained using resonant column should be corrected by subtracting D_{app} . In cyclic triaxial testing, Gookin et al. (1996) pointed out that besides calibrating each transducer individually, the overall testing apparatus' signal conditioning and acquisition system need to be assessed. By assessing the load-deformation characteristics of aluminium, they found that a time delay was introduced by the load cell signal processing conditioner in the cyclic triaxial apparatus. In addition, even though internal measurements were used in the cyclic triaxial test, Gookin (1998) reported that friction and damping of the apparatus were still recorded (Figure 3.6).

Figure 3.6 Equipment testing for cyclic triaxial apparatus (modified from Gookin 1998).

Wu (2002) found that the cyclic simple shear apparatus experienced a friction load of 2.5N (Figure 3.7). However, apparatus damping was not determined even though a hysteretic loop was clearly seen in Figure 3.7. Seo (2008) performed cyclic simple shear test on sand at 0.005% shear strain and noted that in the cyclic loop, the change in load at the tip of hysteresis loop upon stress reversal occurs with no change in strain with the result that shear modulus and damping ratio were lower and higher, respectively, as compared to values in the literature (Figure 3.8). Seo (2008) attributed the differences to apparatus compliance and attempted to develop a correction procedure. However, the correction procedure was not elaborated in detail. Seo (2008) further reported that damping ratio below shear strain of 0.5% was still unreliable even though correction procedure was applied, due to excessive apparatus compliance.

Figure 3.7 Equipment testing for cyclic simple shear apparatus (Wu 2002).

Figure 3.8 Cyclic loop of Ottawa sand at 0.005% shear strain
(modified from Seo 2008).

In this study, the displacement control of both GCTS and GDS cyclic simple shear apparatuses was evaluated by comparing the displacement-time histories of a similar test on Ottawa 20-30 sand under a sinusoidal shear displacement loading given as follows:

$$\delta_{hdt} = A \sin(2\pi ft) \quad (3.3)$$

where δ_{hdt} is the applied horizontal displacement, A is the magnitude of the applied horizontal displacement, π is a mathematical constant (i.e. 3.1415...), f is the frequency of the test, and t is the time.

Figures 3.9 and 3.10 show a typical displacement-time history for tests GCTS 25 kPa and GDS 17 kPa, respectively, under a sinusoidal shear displacement loading with $A = 0.04$ mm and $f = 0.5$ Hz as given by Equation 3.3. It can be seen that both GCTS and GDS cyclic simple shear apparatuses have excellent displacement

control, as the controlled displacement-time curves are almost identical to the input signal (i.e. sinusoidal wave).

The shear load-time histories of GCTS and GDS cyclic simple shear tests are compared in Figure 3.11 where shear load is normalized by mean effective force, F_m . It can be seen that the normalized shear load curve for GCTS cyclic simple shear apparatus has a data acquisition time delay of approximate 0.04s as compared to GDS cyclic simple shear apparatus. This is probably because there are more connecting parts in between the soil specimen and load cell in GCTS cyclic simple shear apparatus, resulting in a delay of load recording. Another possible reason is that the GDS cyclic simple shear apparatus has a dedicated data acquisition board separate from the actuator control board. If the time delay for shear load is left uncorrected, the damping ratio of the soil in the GCTS cyclic simple shear test would appear to be smaller than it should be compared to the GDS cyclic simple shear test. As such, for all GCTS cyclic simple shear tests, a time shift of 0.04s was applied to the shear load to make it comparable with GDS cyclic simple shear tests. Figure 3.12 shows a typical normalized shear load-time history after correction for time shift of 0.04s was applied for GCTS 25 kPa test. It should be noted that shear modulus remains the same with the time shift in GCTS 25 kPa test.

3.7.1 Friction Test

In cyclic simple shear apparatus equipped with triaxial cell chamber, the shear load cell can be mounted either inside (Boulanger et al. 1993; Biscontin 2001) or outside (Chang and Hong 2008) the cell chamber. Biscontin (2001) reported that even though the shear load cell was placed inside the cell chamber, frictional and inertial loads that were not applied to the soil specimen were also recorded. A maximum track friction of 4.5N was measured in her tests. However, owing to the difficulty in defining the measurement for the cyclic loads at reversal points during cyclic tests, no correction for friction was applied to the measured horizontal load (Biscontin 2001).

a) 10 loading cycles

b) Enlarged first 1.5 cycles.

Figure 3.9 Applied horizontal displacement–time history for test GCTS 25 kPa with a perfect sine curve.

Figure 3.10 Applied horizontal displacement–time history for test GDS 17 kPa with a perfect sine curve.

Figure 3.12 Normalized shear load–time history after time shift for tests GCTS 25 kPa and GDS 17 kPa: a) 10 loading cycles; b) Enlarged first 1.5 cycles.

Figure 3.11 Normalized shear load–time history for tests GCTS 25 kPa and GDS 17 kPa: a) 10 loading cycles; b) Enlarged first 1.5 cycles.

ASTM D6528-07 (2007) defines friction as the addition of two friction components: slide table friction and lateral confinement device (i.e., wire-reinforced membrane and Teflon-coated rings) resistance which is a function of applied horizontal displacement. Accounting for friction components that come from the roller bearing, and the contact between seal and reciprocating horizontal piston, the shear stress measured is theoretically $\tau_{total} = \tau_{soil} + \tau_{friction}$ (Doroudian and Vucetic 1995). Except for double specimen direct simple shear (DSDSS) apparatus, other cyclic simple shear apparatuses appear to suffer from friction problems (Doroudian and Vucetic 1995). Table 3.4 summarizes the friction value reported by various researchers for cyclic simple shear apparatuses. The friction load in cyclic simple shear apparatus ranges from 2.2N to 14N.

Table 3.4 Friction of cyclic simple shear apparatuses

References	Friction (N)
Boulanger et al. (1993)	6.7
Biscontin (2001)	4.5
Whang (2001)	2.3
Wu (2002)	2.5
Duku et al. (2007)	2.2
Chang and Hong (2008)	14

To determine friction in the GCTS and GDS cyclic simple shear apparatuses, a series of displacement-controlled cyclic test were performed with no soil specimen at a frequency of 0.5 Hz for 10 loading cycles. For GCTS cyclic simple shear apparatus, the friction test was performed without cell pressure in the chamber. The main source of friction of the GCTS cyclic simple shear apparatus is the contact between seal and reciprocating horizontal piston (Figure 3.13). For GDS cyclic simple shear apparatus, the Teflon-coated rings were stacked up to the top platen with no soil specimen. The friction test measured the slide table friction and the resistance of the lateral confinement device as shown in Figure 3.14. Idealized friction measurement and its definition are presented in Figure 3.15 while typical friction measurement and its properties for GCTS and GDS cyclic simple shear apparatuses are shown in Figures 3.16 and 3.17, respectively. Details of all the friction tests are shown in Appendix A.

Figure 3.13 Friction components of GCTS cyclic simple shear apparatus.

Figure 3.14 Friction components of GDS cyclic simple shear apparatus.

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Figure 3.15 Idealized friction measurement and its definition.

Figure 3.16 Typical friction test for GCTS cyclic simple shear apparatus.

Figure 3.17 Typical friction test for GDS cyclic simple shear apparatus.

Figure 3.18 shows that friction load was slightly affected by number of loading cycles for GCTS cyclic simple shear apparatus. Friction decreases with number of loading cycles until it reaches a steady-state condition. For GDS cyclic simple shear apparatus, the friction load appears to be almost independent of number of loading cycles. The average friction load for ten loading cycles as a function of applied horizontal displacement is shown in Figure 3.19. Friction load is not significantly affected by the applied horizontal displacement for GCTS cyclic simple shear apparatus as compared to GDS cyclic simple shear apparatus. The slight increment of friction load with applied horizontal displacement for GCTS cyclic simple shear apparatus is likely due to the stiffness of the plain rubber membrane. For GDS cyclic simple shear apparatus, the friction load increases more significantly with increasing applied horizontal displacement. This is because GDS cyclic simple shear apparatus has lateral confinement rings (i.e, Teflon-coated rings, but not GCTS cyclic simple shear apparatus). ASTM D6528-07 (2007) suggests that lateral confinement device's resistance is a function of applied horizontal displacement. The friction loads for both GCTS and GDS cyclic simple shear apparatuses can be approximated by:

for GCTS: $F_f = 0.204 \ln \left(\frac{\delta_{lvd\text{t}}}{1 \text{ mm}} \right) + 5.468 \text{ (N)}$ (3.4a)

for GDS: $F_f = 0.545 \ln \left(\frac{\delta_{lvd\text{t}}}{1 \text{ mm}} \right) + 3.536 \text{ (N)}$ (3.4b)

Figure 3.18 Effect of number of loading cycles on friction for GCTS and GDS cyclic simple shear apparatuses.

Figure 3.19 Average friction load of 10 loading cycles for both GCTS and GDS cyclic simple shear apparatuses.

Friction energy is divided into two components: damping energy, W_{fd} , and strain energy, W_{fs} . The effect of number of loading cycles on W_{fd} and W_{fs} appears to be negligible for both GCTS (Figure 3.20) and GDS (Figure 3.21) cyclic simple shear apparatuses. However, both W_{fd} and W_{fs} increase with increasing applied horizontal displacement for both GCTS and GDS cyclic simple shear apparatuses as shown in Figure 3.22. The trends can be approximated by power functions which are given as follows:

$$\text{for GCTS: } W_{fd} = 19.614 \left(\frac{\delta_{lvd}}{1 \text{ mm}} \right)^{1.0455} \quad (3.5a)$$

$$W_{fs} = 11.072 \left(\frac{\delta_{lvd}}{1 \text{ mm}} \right)^{1.0432} \quad (3.5b)$$

$$\text{for GDS: } W_{fd} = 1.6064 \left(\frac{\delta_{lvd}}{1 \text{ mm}} \right)^{1.0337} \quad (3.5c)$$

$$W_{fs} = 9.3483 \left(\frac{\delta_{lvd}}{1 \text{ mm}} \right)^{1.3203} \quad (3.5d)$$

This friction energy will be used in the later section for damping ratio correction.

Figure 3.20 Effect of number of loading cycles on friction energy for GCTS cyclic simple shear apparatus.

Figure 3.21 Effect of number of loading cycles on friction energy for GDS cyclic simple shear apparatus.

Figure 3.22 Average friction energy dissipated in 10 loading cycles for both GCTS and GDS cyclic simple shear apparatuses.

3.7.2 Shear Strain Uniformity

A soil element in pure simple shear does not experience volume change, but this is difficult to achieve in a simple shear apparatus and therefore simple shear apparatus also allows vertical strain under constant vertical stress application (Reyno et al. 2005). For dry or unsaturated soil, drained cyclic simple shear tests with constant load application were widely used (Silver and Seed 1971a; Chu and Vucetic 1992; Hsu and Vucetic 2004; and Whang et al. 2004). The test permits volume change to occur in which seismic compression and earthquake-induced settlement can be investigated (Silver and Seed 1971; Hsu and Vucetic 2004; and Whang et al. 2004). In constant vertical load tests, the soil specimen is subject to constant vertical load while the vertical displacement is allowed to change. This is achieved by maintaining the vertical load constant at a value at the end of consolidation stage using the feedback system. As volume change is permitted in constant load test, the lateral flexible side of soil specimen can therefore, expand during the cyclic test. In GDS cyclic simple shear apparatus, the lateral expansion of soil specimen is prevented by a stack of Teflon-coated rings while in GCTS cyclic simple shear apparatus, the unreinforced rubber membrane has a lesser restraint to the vertical faces of the soil specimen which is provided by the cell pressure (Reyno et al. 2005; Joer et al. 2010; and Doherty and Mahey 2011).

As the use of unreinforced membrane in the GCTS cyclic simple shear apparatus inevitably leads to departure of the ideal simple shear condition (Reyno et al. 2005), a water-proofed proximity transducer (AEC-55MS-L-PU-20) was installed to measure the horizontal displacement of the soil specimen at mid height for shear strain uniformity monitoring.

As shown in Figure 3.23, the shear displacement, δ_{lvd} was applied at the bottom, while the horizontal displacement at the top and middle-height of the soil specimen were measured. Horizontal displacement of the top platen was observed in Figure 3.23, indicating the cyclic simple shear apparatus suffered from rocking problem. As there was a rotation at the top platen, the theoretical mid-height displacement, $\delta_{mid,theo}$ can be computed as:

$$\delta_{mid,theo} = \frac{(\delta_{lvd} + \delta_{top})}{2} \tag{3.6}$$

Figure 3.23 shows the discrepancy between measured and theoretical mid-height displacements, indicating the presence of non-uniform shear strain in the test. The discrepancy between measured and theoretical mid-height displacements varied from 14% to 28% for drained constant vertical stress tests on dry Ottawa 20-30 sand specimens (Table 3.5), 4% to 28% for drained constant vertical stress tests on unsaturated residual soil specimens (See Table 6.2), and 3% to 28% for undrained constant height tests on saturated residual soil specimens (See Table 6.1). This observation shows that the departure from ideal simple shear condition was comparable to cyclic simple shear apparatus using reinforced membrane as demonstrated by Lucks et al. (1972) who observed that only approximately 70% of the specimen had uniform stress condition in an NGI type simple shear apparatus from three-dimensional finite element analyses. Given that no cyclic simple shear apparatus is able to impose ideal simple shear condition, the test results from shear strain monitoring suggested that since the specimen was thin, then away from the vertical boundary, the conditions must approach “simple shear” because of the horizontal boundary displacement.

Figure 3.23 Comparison of theoretical and measured mid-height horizontal displacement at $\gamma_c = 0.453\%$ for test GCTS 25 kPa.

Table 3.5 Discrepancy between theoretical and measured mid-height horizontal displacement of Ottawa 20-30 sand specimens under constant vertical stress application

Specimen no.	Shear strain, γ_c (%)	Discrepancy between theoretical and measured mid-height horizontal displacement of soil specimen (%)
GCTS 25 kPa	0.036	24
	0.079	25
	0.169	20
	0.453	14
	0.934	28
GCTS 100 kPa	0.033	28
	0.070	28
	0.147	27
	0.403	26
	0.870	20
GCTS 200 kPa	0.033	27
	0.070	28
	0.142	27
	0.382	25
	0.824	26

In this study, the departure of the specimen from the simple shear condition was attributed to the rocking of the top platen which will be discussed in detail in Section 3.8.

3.7.3 Loading System

Compliance of loading system in cyclic simple shear apparatus affecting test results has been reported by a number of researchers (Boulanger et al. 1993; Miller et al. 1996; Duku et al. 2007). An extremely rigid loading system is needed to test stiff geomaterial like sand. To stiffen the loading system, the GCTS cyclic simple shear apparatus has a cross brace secured to the internal posts, while allowing vertical load to be applied. In GDS cyclic simple shear apparatus, there was no cross brace to stiffen the loading system. However, the placement of shear thrust at the top platen for shear load measurement was believed to compensate the compliance of the loading system.

To investigate the effect of loading system compliance on cyclic simple shear test, the movable cross brace in GCTS cyclic simple shear apparatus was adjusted to two locations to create two contrasting loading systems: “flexible” and rigid”. For “flexible” loading system, the cross brace was secured to the internal post, 130 mm from the top platen. For “rigid” loading system, the cross brace was secured to the internal post, 2 mm from the top platen. The margin of 2 mm was provided to allow unexpected vertical piston movement.

Figures 3.24 and 3.25 show the ten cyclic loops of Ottawa sand 20-30 sand under “flexible” and “rigid” loading systems, respectively. The dry Ottawa 20-30 sand had an initial void ratio of 0.60, corresponding to 45% relative density, and tested under a mean effective stress of 100 kPa with constant load application at shear strain about 1%. Figures 3.24 and 3.25 show the change in load upon stress reversal was more gradual under “flexible” loading system than under “rigid” loading system. Consequently, the size of the cyclic loop under “flexible” loading system was “slimmer” than under “rigid” loading system. The shear modulus obtained under “flexible” loading system was found to be about 2.5 times lower that that obtained under “rigid” loading system. From the finding, it can be concluded that a rigid loading system is needed to prevent the soil specimen from departing the intended simple shear condition.

Figure 3.24 Cyclic loop under “flexible” loading system for GCTS cyclic simple shear apparatus.

Figure 3.25 Cyclic loop under “rigid” loading system for GCTS cyclic simple shear apparatus.

3.8 Shear Modulus Determination

False deformation occurs when loading system is not stiff enough. This results in the soil specimen to appear to be softer than it should be. Over the years, improvement has been made to stiffen the loading system of cyclic simple shear apparatus to prevent the top platen from rocking (Boulanger et al. 1993; Miller et al. 1996). However, rocking still persists. The GCTS cyclic simple shear apparatus used in this study has a cross brace secured to the internal posts to minimize rocking, while allowing vertical load to be applied. To investigate the effect of rocking caused by the lack of complementary shear stress, the horizontal displacement of the top platen in GCTS cyclic simple shear apparatus was measured using a proximity transducer. Figure 3.26 shows that the horizontal displacement of top platen did occur, resulting in false deformation of the soil. As such, the actual peak horizontal displacement, δ_{act} , of the soil was corrected by subtracting the average peak-to-peak applied horizontal displacement, δ_{lvd} with average peak-to-peak measured top platen horizontal displacement, δ_{top} , i.e.,

$$\delta_{act} = \delta_{lvd} - \delta_{top} \quad (3.7)$$

Therefore, the shear strain, γ_c of the soil specimen is given as:

$$\gamma_c = \frac{\delta_{act}}{h_i} \quad (3.8)$$

where h_i is the height of the soil specimen at the beginning of i^{th} stage test. Typical applied horizontal displacement-, measured top platen horizontal displacement-, and actual horizontal displacement-time histories plot are shown in Figure 3.26. Typical cyclic loops showing the applied and actual horizontal displacements are shown in Figure 3.27.

For GDS cyclic simple shear apparatus, the top platen horizontal displacement was not measured because there was insufficient channel in the data acquisition system. However, the top platen horizontal displacement was estimated based on the deflection of the shear thrust. The deflection of the shear thrust is about 0.005 mm at full range (GDS 2010). Considering the full range of the shear load used is 1 kN, the deflection of the thrust, δ_{thrust} , resulting from a shear load, x , can be estimated as follows:

$$\delta_{thrust} = \frac{x \text{ kN}}{1 \text{ kN}} (0.005 \text{ mm}) \quad (3.9)$$

Assuming the top of the shear thrust is rigid, while the maximum deflection is at the bottom of the shear thrust, the top platen horizontal displacement can be estimated using a linear relationship as shown in Figure 3.28. As the top platen has no cross brace which is used to minimise the horizontal movement, the GDS cyclic simple shear apparatus top platen is suspected to move in a non-linear fashion. A factor of 4.2 was applied based on back-calculation on the proposed correlation between shear modulus correction factor $c(G)$ and R_δ which will be described later in this chapter:

$$\delta_{top(GDS)} = 11.36(\delta_{thrust}) \text{ (mm)} \quad (3.10)$$

Typical cyclic loops showing the applied and estimated actual horizontal displacements (from Equations 3.9 and 3.10) for a GDS test (e.g. GDS 17 kPa) are shown in Figure 3.29. It can be seen that the correction made is not large and almost similar to those in GCTS cyclic simple shear test.

Figure 3.26 Typical applied horizontal displacement-, measured top platen horizontal displacement-, and actual horizontal displacement-time histories plot for test GCTS 25 kPa.

Figure 3.27 Typical cyclic loops for the applied and actual horizontal displacement for test GCTS 25 kPa for the 1st loading cycle.

Figure 3.28 Illustration of top platen horizontal displacement of GDS cyclic simple shear apparatus.

Figure 3.29 Typical cyclic loops for the applied and estimated actual horizontal displacement for test GDS 17 kPa for the 1st loading cycle.

For friction correction, the procedures suggested in ASTM D6528-07 (2007) were followed. The calculated actual shear load, F_{act} , is obtained by subtracting the average shear load, F_t , measured in a soil test with average friction load, F_f , measured in the friction test. The equation is given as follows:

$$F_{act} = F_t - F_f \quad (3.11)$$

In this study, F_f is approximated by Eqs. 3.4(a) and 3.3(b) for GCTS and GDS cyclic simple shear apparatuses, respectively. Shear stress of soil, τ , is obtained by dividing F_{act} by area of soil specimen, A_{soil} , i.e.,

$$\tau = \frac{F_{act}}{A_{soil}} \quad (3.12)$$

The shear modulus after correction for apparent deformation and friction can be written as:

$$G = \frac{\tau}{\gamma_c} \quad (3.13)$$

3.8.1 Comparison of Shear Modulus with Other Studies

Using resonant column and torsional shear apparatuses, Alarcon-Guzman et al. (1989) tested dry Ottawa 20-30 sand at four mean effective stresses σ'_m : 29, 68, 103, and 202 kPa. The tested Ottawa 20-30 sand had an initial void ratio of 0.64, corresponding to relative density of 40%. Comparison was made between this study and Alarcon-Guzman et al. (1989), as the material used and initial test condition in Alarcon-Guzman et al. (1989) were comparable to this study.

Figure 3.30 shows the shear moduli for tests GCTS 25 kPa and GDS 17 kPa on Ottawa 20-30 sand, respectively, after the above correction. From Figure 3.30, it

can be seen that the shear moduli are comparable to those obtained by Alarcon-Guzman et al. (1989) for Ottawa 20-30 sand tested at mean effective stress of 29 kPa using resonant column and torsional shear apparatuses for shear strains from about 0.03% to 1%, considering the difference in mean effective stress. However, below shear strain of 0.03%, the shear moduli obtained using both the GCTS and GDS cyclic simple shear apparatuses were considerably lower than those of Alarcon-Guzman et al. (1989). Therefore, it is suggested that the cyclic simple shear apparatuses used are only suitable for tests for shear strain $\geq 0.03\%$ at low vertical stress, of the order of 25 kPa (i.e., less compliance/rocking problem).

Figure 3.30 Comparison of shear modulus degradation curves for GCTS 25 kPa and GDS 17 kPa tests with Alarcon-Guzman et al. (1989) test.

Figure 3.31 shows the shear moduli of the tested Ottawa 20-30 sand at various σ'_m obtained using GCTS and GDS cyclic simple shear apparatus for shear strain from about 0.01% to 1%. From Figure 3.31, it can be seen that the shear moduli in the range of shear strains from about 0.03% to 0.2% for σ'_m above 25 kPa obtained by both GCTS and GDS cyclic simple shear apparatuses are considerably lower than those from resonant column and torsional shear apparatuses by Alarcon-Guzman et

al. (1989). As σ'_m increases, the difference of the shear moduli becomes more pronounced as compared with Alarcon-Guzman et al. (1989). For GDS cyclic simple shear apparatus, although the shear modulus increases with increase in σ'_m , the values are still much lower than those of Alarcon-Guzman et al. (1989), especially for shear strains from 0.03% to 0.2%. For GCTS cyclic simple shear apparatus, the increment of shear moduli reduces with increasing mean effective stress. In Figure 3.31, it can be seen that the shear modulus for the mean effective stress of 200 kPa is lower than the shear modulus for the mean effective stress of 140 kPa conducted using GDS cyclic simple shear apparatus. This implies that the GCTS cyclic simple shear apparatus suffers more compliance problems than GDS cyclic simple shear apparatus.

Figure 3.31 Comparison of shear modulus degradation curves for GCTS and GDS cyclic tests with Alarcon-Guzman et al. (1989) tests.

Low values of shear modulus obtained using cyclic simple shear apparatus are often associated with compliance of the apparatus (Miller et al. 1996; Seo 2008). One major issue is rocking as discussed earlier. In this study, horizontal displacement of

top platen, δ_{top} , was observed and the low shear modulus values were suggested to be a function of R_δ defined as follows:

$$R_\delta = \frac{(\delta_{lvd} - \delta_{top})}{\delta_{lvd}} \tag{3.14}$$

where $R_\delta = 1$ when $\delta_{top} = 0$, indicating perfect simple shear condition; and $R_\delta = 0$ when $\delta_{top} = \delta_{lvd}$, indicating that condition is no longer in simple shear.

Figure 3.32 shows the variation of R_δ with shear strain levels. It can be seen that R_δ increases with increasing shear strain. At small shear strain (i.e., $< 0.5\%$), R_δ is low and therefore the shear modulus conducted using cyclic simple shear apparatus is considerably lower than Alarcon-Guzman et al. (1989) study. At large shear strain (i.e., $\geq 0.5\%$), when R_δ approaches unity, the shear modulus is almost the same with those obtained by Alarcon-Guzman et al. (1989). For a given shear strain level, as σ'_m increases, R_δ decreases, indicating the tests tend to depart from the simple shear condition. Consequently, the increment of shear modulus reduces with increasing of σ'_m .

Figure 3.32 Variation of R_δ with shear strain.

In order to “bring up” the shear modulus of Ottawa 20-30 sand from cyclic simple shear test to that of resonant column and torsional shear tests, a correction factor, $c(G)$, was proposed to be applied to the shear modulus from cyclic simple shear test:

$$c(G) = \frac{G_{\text{expected}}}{G_{\text{measured}}} \quad (3.15)$$

where G_{expected} is shear modulus from Alarcon-Guzman et al. (1989) and G_{measured} is shear modulus obtained from GCTS and GDS cyclic simple shear apparatuses. The correction factor, $c(G)$ was then correlated with R_δ as shown in Figure 3.33. Figure 3.33 shows that $1/c(G)$ is linearly related to R_δ for R_δ values between 0.7 and 0.82 and the relationship is given by:

$$\frac{1}{c(G)} = 6.991(R_\delta) - 4.756 \quad (3.16)$$

As R_δ increases, $1/c(G)$ increases. For $R_\delta < 0.7$, no correlation was found and it can be construed that simple shear condition no longer prevails and cannot be corrected. For $R_\delta > 0.82$, it is suggested that simple shear condition is approached and no correction is required.

Using the proposed correction factor [Eqs. 3.14 through 3.16], the shear moduli obtained by GCTS and GDS cyclic simple shear apparatus were re-plotted and compared with Alarcon-Guzman et al. (1989) in Figure 3.34. Figure 3.34 shows that the “corrected” shear moduli compared well with those of Alarcon-Guzman et al. (1989) for shear strains from about 0.03% to 1%. The shear modulus curves increase with increasing σ'_m .

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Figure 3.33 Correlation between correction factor, $c(G)$ with R_δ

Figure 3.34 Shear modulus degradation curves for GCTS and GDS cyclic simple shear apparatuses tests after R_δ correction.

3.9 Damping Ratio Determination

Damping ratio represents the energy dissipated by the soil and can be calculated as:

$$D = \frac{1}{\pi} \frac{W_d}{W_s} \quad (3.17)$$

where W_d is damping energy dissipated in one cycle of loading, and W_s is maximum stored strain energy in the cycle (Zhang et al. 2005). Figure 3.35 illustrates the cyclic loop and definition of damping ratio of the soil.

Figure 3.36 shows the comparison of damping ratio of Ottawa 20-30 sand obtained using GCTS and GDS cyclic simple shear apparatuses. For GDS cyclic simple shear apparatus, it can be seen that the damping ratio increases with increasing shear strain levels, but decreases with increasing σ'_m . For GCTS cyclic simple shear apparatus, it can be seen that although damping ratio decreases with increasing σ'_m , no defined trend was observed for damping ratio with shear strain levels. This appears to suggest that correction is also needed for damping ratio determination.

Figure 3.35 Definition of damping ratio.

Figure 3.36 Comparison of damping ratio of Ottawa 20-30 sand.

Finn et al. (1982) suggested that the energy loss due to friction, which is represented by an almost rectangular hysteresis loop, need to be corrected. However, no detailed correction procedure was presented. After comparing the “standard” sand tested in NGI-type cyclic simple shear apparatus which is believed to have negligible friction, Matasović and Kavazanjian (1998) and Matasović et al. (1998) suggested that damping ratio be corrected by subtracting 4% from the experimental damping ratio values in their large-scale cyclic simple shear apparatus.

In this study, correction of damping ratio due to friction energy loss proposed by Finn et al. (1982) was adopted. Since both GCTS and GDS cyclic simple shear apparatuses were subjected to friction, corrections were made to test results from both apparatuses. To obtain damping energy solely due to soil, W'_d , damping energy loss due to friction, W_{fd} for a given applied horizontal displacement (i.e., Eqs. 3.5a and 3.5c for GCTS and GDS cyclic simple shear apparatuses, respectively) was subtracted from damping energy obtained in a soil test, W_d as given by:

$$W'_d = W_d - W_{fd} \quad (3.18)$$

To obtain strain energy stored solely due to soil, W'_s , strain energy induced solely by apparatus, W_{fs} for a given applied horizontal displacement (i.e., Eqs. 3.5b and 3.5d for GCTS and GDS cyclic simple shear apparatus, respectively) was subtracted from strain energy obtained in a soil test, W_s as given by:

$$W'_s = W_s - W_{fs} \quad (3.19)$$

Finally, the damping ratio of soil after correcting for equipment friction can be written as:

$$D = \frac{1}{\pi} \frac{W'_d}{W'_s} \quad (3.20)$$

Using the correction procedure [Eqs. 3.18 through 3.20], the damping ratio of Ottawa 20-30 sand obtained by GCTS and GDS cyclic simple shear apparatuses were re-plotted and compared with Seed and Idriss (1970) envelope and other studies (Amer et al. 1986; and Alarcon-Guzman 1986) in Figure 3.37. From Figure 3.37, it can be seen that all the damping ratios increase with increasing of shear strain levels for the shear strains from about 0.03% to 1%. The damping ratios of Ottawa 20-30 sand for both GCTS and GDS cyclic simple shear apparatuses follow a typical defined trend in which the damping ratio decreases with increasing σ'_m . This trend is similar to Alarcon-Guzman (1986) study. No effort was made to “bring up” the damping ratios obtained using cyclic simple shear apparatus to resonant column and torsional shear apparatus (i.e. Alarcon-Guzman 1986). The difference in damping ratios is due to inherent variation of different testing apparatuses as pointed out by Alarcon-Guzman (1986).

Figure 3.37 Damping ratio of Ottawa 20-30 sand after friction correction.

3.10 Repeatability Test with GCTS Cyclic Simple Shear Apparatus

Repeatability test is important in an experimental investigation, especially for specialised testing such as performed in this study. Repeatability test was conducted on the Ottawa 20-30 sand using the GCTS cyclic simple shear apparatus. Two “identical” sand specimens were prepared at a void ratio of approximately 0.60 (relative density of 45%) and tested under the same loading conditions (i.e. $f = 0.5$ Hz, sinusoidal waveform, 10 loading cycles, and $\sigma'_m = 100$ kPa). The resulting shear modulus and damping ratio of both sand specimens are compared in Figure 3.38.

Figure 3.38 shows the shear modulus and damping ratio of the tested sand specimens after making the corrections described in Sections 3.8 and 3.9. It can be seen that the shear moduli and damping ratios obtained in Test #1 are almost identical to the values obtained in Test #2, thus suggesting that test is repeatable in the GCTS cyclic simple shear apparatus.

Figure 3.38 Shear modulus and damping ratio of Tests #1 and #2 on Ottawa 20-30 sand.

3.11 Summary

This chapter examines the capability of the GCTS and GDS cyclic simple shear apparatuses. It was found that the cyclic simple shear apparatuses suffer from compliance problem and experience rocking to the top platen. In current practice of cyclic simple shear tests, the top platen of the cyclic simple shear apparatus is assumed to be rigid and has negligible rocking. However, this is not the case and the amount of rocking depends on the design of the cyclic simple shear apparatus. As a result of rocking, the shear modulus at small shear strains tend to be much lower than it should be. A cut-off shear strain for GCTS and GDS cyclic simple shear apparatuses was identified to be about 0.03%. Despite the limitation, cyclic simple shear tests conducted from shear strain of 0.03% can still give reasonably useful results for a number of the geotechnical applications: retaining walls, foundations, and tunnel (see Figure 1.1).

Due to compliance of the apparatus and rocking of the top platen, calibration of a cyclic simple shear apparatus is essential before it can be used to characterize the

cyclic properties of soils, especially in the small-strain zone. In this study, the cyclic simple shear apparatus was calibrated by comparing the test results on Ottawa sand 20-30 sand with those using resonant column and torsional shear tests which have a proven track record on measuring small-strain soil properties.

To ensure simple shear boundary condition is met, measurement of horizontal displacement of top platen is suggested. This measurement is particularly important in the small shear strain range, as in some cases, δ_{top} could even be as high as 40% of the applied shear displacement, δ_{lvd} . Consequently, the shear modulus of soil will appear to be much lower than it actually is. With the measurement of δ_{top} , a correlation between R_δ (i.e, a function of δ_{top}) and shear modulus correction factor, $c(G)$, could be established in order to “bring up” the shear modulus of soil to the shear modulus determined in resonant column and torsional shear apparatus.

Researchers have opted not to apply friction correction (Biscontin 2001) or apply friction correction to either only shear modulus (Miller et al. 1996), or damping ratio (Matasović and Kavazanjian 1998; and Matasović et al. 1998). In this study it is suggested that for cyclic simple shear apparatuses subjected to friction load that is not negligible, friction correction is essential for both shear modulus and damping ratio determination. If left uncorrected, the shear modulus of soil will appear stiffer than it actually is, and damping ratio will not follow an orderly trend.

Chapter 4

Materials and Unsaturated Soil Specimen Preparation

4.1 General

This chapter presents the properties of the soils used in this study. Consolidating an unsaturated soil specimen to the desired stresses can be very time-consuming. To reduce the consolidation time of unsaturated residual soil specimens in the cyclic simple shear apparatus, a pressure plate apparatus was used to apply the desired matric suction to the soil specimens before transfer to the cyclic simple shear apparatus.

4.2 Materials

Both undisturbed and reconstituted residual soils were used in this study. For reconstituted soils, a sand-kaolin mixture was also used for comparative study. Commercially available kaolin was mixed with sand to produce the sand-kaolin mixture. Basic soil properties test such as grain-size distribution, soil classifications, specific gravity, and Atterberg Limits were performed in accordance with ASTM D422-63 (Reapproved 2002), ASTM D2487-06 (2006), ASTM D854-06 (2006), and ASTM D4318-05 (2005), respectively. Figure 4.1 and Table 4.1 show the grain-size distributions, and basic index properties of the soils used in this study, respectively.

4.2.1 Undisturbed Residual Soils

The undisturbed residual soils were used for a series of cyclic simple shear tests under both saturated and unsaturated conditions. As discussed in Chapter 2, there are two main types of residual soils in Singapore: Jurong Formation and Bukit Timah Granite. To compare the characteristics of residual soils from the two formations, residual soils derived from both Jurong Formation and Bukit Timah Granite were used. Jurong Formation residual soils were obtained from a site in Jalan Minyak, denoted as JM, southern central part of Singapore. Bukit Timah

Granite residual soils were obtained from a site in Ang Mo Kio, denoted as AMK, central part of Singapore. Both residual soil samples obtained were high quality undisturbed soil samples obtained using continuous foam drilling with a Mazier sampler. Figure 4.2 shows the sampling location of the residual soils.

As the degree of weathering decreases with depth, the residual soils used in this study were from shallow depths. For JM soil, only the sample from shallow depths 2.5m to 3.0m was used. The groundwater table is about 15m below the ground surface. Hence, the undisturbed soil sample was unsaturated in nature. The soil has a natural water content of about 12.1% and a void ratio of 0.34. It consisted of 15% sand and 85% fines (silt and clay), with a liquid limit of 34%, plastic limit of 20%, and plasticity index of 14%. According to USCS, it is classified as low-plasticity clay, CL.

For AMK soil, the sample recovered from shallow depths of 3.0m to 3.5m was used. The groundwater table is about 2.5m below the ground surface. The soil has a natural water content of about 41.8% and a void ratio of 1.09. It consisted of 37% sand and 63% fines (silt and clay), with a liquid limit of 58%, plastic limit of 32%, and plasticity index of 26%. According to USCS, it is classified as high-plasticity silt, MH.

Figure 4.1 Grain-size distributions of the soils used in this study.

Table 4.1 Basic index properties of soil used in this study

Properties	Undisturbed		Reconstituted	
	Residual soil		Residual soil	Sand-kaolin mixture
	Bukit Timah Granite	Jurong Formation	Jurong Formation	Mixture
	AMK	JM	NTU	SKM
Specific gravity, G_s	2.66	2.70	2.69	2.67
Liquid Limit, LL (%)	58	34	37	31
Plastic Limit, PL (%)	32	20	19	15
Plasticity Index, PI (%)	26	14	18	16
Grain-size distribution (%)				
- Gravel	0	0	5	0
- Sand	37	15	41	63
- Silt and clay	63	85	54	37
USCS	MH	CL	CL	SC
Sampling depth (m)	3.0 – 3.5	2.5 – 3.0	0 - 2	NA
Method of sampling	Mazier sampler	Mazier sampler	Excavation	NA
Groundwater table (m)	2.5	15	NA	NA

NTU = Nanyang Technological University

JM = Jalan Minyak

AMK = Ang Mo Kio

Figure 4.2 Location of sampled residual soils in Singapore (modified from PWD 1976).

4.2.2 Reconstituted Residual Soils

Due to heterogeneity of residual soils as a result of variable degree of weathering, residual soil obtained from a construction site in Nanyang Technological University, denoted as NTU, western part of Singapore (Figure 4.2), was used to prepare reconstituted soil specimens. The reconstituted residual soil provides “homogeneous” soil specimens, such that systematic investigation can be carried out. Reconstituted residual soil was used for investigations of equilibrium time in pressure plate test and factors affecting the cyclic simple shear properties of residual soil (i.e., shear strain, mean effective stress, matric suction, and number of loading cycles).

The origin of the soil is Jurong Formation, and it was obtained from shallow depths of 0m to 2m using an excavator. The soil consisted of 5% gravel, 41% sand, and 54% fines, with a liquid limit of 37%, plastic limit of 19%, and plasticity index of 18%. According to USCS, it is classified as high-plasticity clay, CL. The reconstituted residual soil was mixed with water, approximately 1.5 times the liquid limit of the soil, to produce a slurry-like soil. The slurry was then consolidated in a 30-cm diameter consolidation tank under a consolidation pressure of 100 kPa. The final water content of the reconstituted residual soil was 21% and the bulk density was 2.08 Mg/m³.

4.2.3 Sand-Kaolin Mixture

Sand and kaolin mixture was used for investigation of equilibration time in pressure plate test. The reason for using sand and kaolin is because these materials can be easily reconstituted into reproducible and homogeneous soil specimens and thus enabling systematic investigation to be carried out. Sand from a reclamation site was mixed with commercially available kaolin. The soil mixture is denoted as SKM. It consisted of 63% sand, and 37% fines (kaolin), with a liquid limit of 31%, plastic limit of 15%, and plasticity index of 16%. According to USCS, it is classified as clayey sand, SC. The soil mixture was mixed with water, approximately 1.5 times the liquid limit of kaolin, to produce a slurry-like soil. The slurry was then subjected to a consolidation pressure of 100 kPa in a 30-cm diameter consolidation tank. The

final water content of the reconstituted sand-kaolin mixture was 19% and the bulk density was 2.10 Mg/m^3 .

4.3 Preparation of Unsaturated Soil Specimen Using a Pressure Plate Apparatus

A number of apparatuses can be used to apply matric suction to a soil specimen. Typically, Tempe cell is used for low matric suction up to 100 kPa while pressure plate is used for matric suction up to 1500 kPa depending on the capacity of ceramic plates. For matric suction greater than 1500 kPa, equilibrium in a fixed relative humidity environment method (i.e., vacuum desiccators) is employed (Agus et al. 2001, Fredlund 2006). In this study, pressure plate was used to prepare unsaturated soil specimen for cyclic simple shear tests due to its relatively broad range capacity of matric suction application (i.e. 0 – 1500 kPa). The soil specimens were subjected to the desired matric suction in the pressure plate apparatus before being transferred to cyclic simple shear apparatus. It is crucial that the soil specimen attained the applied matric suction within a reasonable time to facilitate testing in the cyclic simple shear apparatus. Therefore, equilibrium time of soil specimen was investigated.

4.3.1 Equilibrium Time of Soil Specimen in Pressure Plate Apparatus

Despite the extensive use of pressure plate test for determining SWCC, there appears to be little guidance on the equilibrium time at an applied matric suction. For example the former ASTM D2325-68 (Reapproved 2000) suggested that equilibrium is attained within two days. ASTM D6836-02 (2002) Methods B and C which replaces ASTM D2325-68 (Reapproved 2000) suggested that at least two days are required to attain equilibrium. Table 4.2 summarizes the equilibrium time suggested by both standards.

To date, different equilibrium times for pressure plate tests have been suggested. Richards (1965) and Klute (1986) suggested that equilibrium time was about 3 days. However, Madsen et al. (1986) suggested that true equilibrium was reached after more than 34 days at a matric suction of 500 kPa for six Danish soil horizons

containing 2 to 27% clay content (clay defined as $< 2 \mu\text{m}$ diameter). Tinjum et al. (1997) and Vanapalli et al. (1999) suggested that equilibrium was reached around seven days for compacted clays and till, respectively. Longer equilibrium time of up to 40 days was reported by Picornell and Nazarian (1998) for fine-grained soils and 45 days was reported by Oliveira and Marinho (2006) for residual soil specimens tested at a matric suction of 400 kPa. Table 4.3 summarizes the equilibrium time suggested in the literature.

ASTM D6836-02 (2002) established the equilibrium criteria based on matric suction level. Equilibrium is said to be achieved if there is no outflow for at least 24 hours, 48 hours, and 96 hours, for matric suction less than 500 kPa, between 500 kPa and 1000 kPa, and greater than 1000 kPa, respectively. Equilibrium is assumed to be attained when outflow of water has practically decreased to a “small” volume (Ball and Hunter 1988; Gee et al. 2002; Creswell et al. 2008). However, Oliveira and Marinho (2006) doubted if the aforementioned criterion is practical, as they observed that outflow never stopped for 62 days for residual soils tested under a matric suction of 400 kPa. The continuous outflow was probably due to condensation inside the pressure chamber. Oliveira and Marinho (2006) recommended that the equilibrium should instead be monitored using the mass change of the specimen.

Table 4.2 Recommended equilibrium time by ASTM

References	Specimen size			Equilibrium criteria
	Height (mm)	Diameter (mm)	Volume (cm ³)	
*ASTM D2325-68 (Reapproved 2000) 10 kPa $\leq (u_a - u_w) \leq 101$ kPa	10	50	19.63	No water flows out of the outlet tubes during a 30-minute period. Suggested equilibrium time = 18 to 48 hours.
ASTM D6836-02 (2002) Methods B and C 0 kPa $\leq (u_a - u_w) \leq 1500$ kPa	≥ 5	≥ 25	≥ 2.45	Allow specimen to equilibrate at least 48 hour, until movement of air-water interface in the capillary tube ceases. For $(u_a - u_w) < 500$ kPa, the air-water interface has not moved for at least 24 hours. For $500 \text{ kPa} \leq (u_a - u_w) \leq 1000$ kPa, the air-water interface has not moved for at least 48 hours. For $(u_a - u_w) > 1000$ kPa, the air-water interface has not moved for at least 96 hours.

* Withdrawn and replaced by ASTM D6836-02 (2002)

Chapter 4 Materials and Unsaturated Soil Specimen Preparation

Table 4.3 Summary of equilibrium time suggested by different researchers

Reference	Soil type	Specimen size		Equilibrium time
		Height (mm)	Diameter (mm)	
Richards (1965)	Soils passing 2-mm sieve	10	60	About 2 days
Klute (1986)	Silty clay loam and sand	20 – 30	50 - 80	2- 3 days
Burke et al. (1986)	-	-	-	2 – 14 days
Madsen et al. (1986)	Danish soils from 2 to 27% clay content*	10	60	> 34 days for $(u_a - u_w) \geq 500$ kPa
Ball and Hunter (1988)	Fine-textured soils	50	75	14 days for $(u_a - u_w) = 50$ kPa; 21 days for $(u_a - u_w) = 100$ kPa
	Fine-textured soils	20	28	16 days for $(u_a - u_w) = 1500$ kPa
Topp et al. (1993)	Clayey soils	76	76	12 - 15 days for $(u_a - u_w) = 100$ kPa;
				15 – 20 days for $(u_a - u_w) = 400$ kPa;
				20 - 25 days for $(u_a - u_w) = 1500$ kPa
Tinjum et al. (1997)	Compacted clays	20	89	5 – 8 days
Picornell and Nazarian (1998)	Fine-grained soils	50	25	15 – 40 days
Vanapalli et al. (1999)	Compacted till	21	63.5	6 – 7 days
Gee et al. (2002)	Sands and clays	15 – 30	44 - 47	> 10 days
Oliveira and Marinho (2006)	Residual soils	38 - 89	38	3 days for $(u_a - u_w) = 200$ kPa, $\Delta(u_a - u_w) = 150$ kPa;
				$(u_a - u_w) = 250$ kPa, $\Delta(u_a - u_w) = 50$ kPa;
				$(u_a - u_w) = 300$ kPa, $\Delta(u_a - u_w) = 50$ kPa;
				$(u_a - u_w) = 400$ kPa, $\Delta(u_a - u_w) = 100$ kPa;
				About 45 days for $(u_a - u_w) = 400$ kPa, $\Delta(u_a - u_w) = 380$ kPa
Creswell et al. (2008)	Australian soils from 1 < to 78% clay content*	10	19	5 – 6 days
Bittelli and Flury (2009)	Silt loam	30	53.5	7 days for $(u_a - u_w) = 1$ kPa, 10 kPa, and 50 kPa
				13 days for $(u_a - u_w) = 100$ kPa;
				27 days for $(u_a - u_w) = 300$ kPa;
				26 days for $(u_a - u_w) = 500$ kPa
				41 days for $(u_a - u_w) = 1000$ kPa 75 days for $(u_a - u_w) = 1500$ kPa

Notes: *clay content is defined as soil particle < 2 μ m diameter.

#Equilibrium is attained after outflow has ceased for at least two days.

Equilibrium time in pressure plate test has been suggested to be dependent on the height of soil specimen (Topp et al. 1993; Townend et al. 2001), in which the time to reach equilibrium is proportional to the square of the height of the specimen (Klute 1986). Khoury and Miller (2008) tested cylindrical specimens of ground silica sand with diameter of 63.5 mm and heights of 6.35 mm and 25.4 mm using a custom made test cell. They found that reducing specimen height by 75%, the time required to determine SWCC was reduced by about 50%. Equilibrium was assumed to be attained when there was less than 1% change of water volume over a period of 4 hours for both drying and wetting processes. Generally, soil specimen should be large enough to be a representative volume of soil in field condition (Ball and Hunter 1988; Topp et al. 1993). Specimen of 10 mm height tends to be an ideal thickness in pressure plate test whereas specimen height greater than 50 mm is undesirable (Townend et al. 2001). Klute (1986) suggested that a practical height is about 20 to 30 mm, as specimen with height less than 10 mm is difficult to handle. Due to gravity flow, non-uniform moisture equilibrium tends to occur for a tall specimen (i.e., 76 mm) in pressure plate tests (Topp et al. 1993). However, Oliveira and Marinho (2006) suggested that equilibrium time in pressure plate test is independent of volume and shape of the specimen.

The contact between soil specimen and ceramic disk has been identified as one of the possible factors that led to long equilibrium time of pressure plate test (Topp et al. 1993; Marinho et al. 2008). Poor contact of soil specimen on the ceramic disk impedes water flow and subsequently results in apparent non-equilibrium condition. Poor contact is not only due to human operators, but may in part due to soil shrinkage on drying. Apart from good contact between soil specimen and ceramic disk, water drainage is also closely associated with the ceramic disk conductance. For soil specimens subjected to long test duration, clogging of ceramic plates by soil particles or biological growth after repeated use may also greatly reduce the water flow and consequently result in non-equilibrium of matric suction in the pressure plate apparatus (Campbell 1985; Townend et al. 2001; Gee et al. 2002; Creswell et al. 2008). The equilibrium matric suction may as a result be lower than that assumed from the applied matric suction and the soil specimens are wetter.

Equilibrium is harder to achieve in pressure plate test at high matric suctions as the rate of water loss is very slow and practically zero (Topp et al. 1993). This is particularly true for coarse-grained soils due to its low hydraulic conductivity at high matric suction. Consequently, the applied matric suction cannot be achieved in a reasonable time. More often, the equilibrium matric suction is lower than the applied matric suction (Campbell 1985; Madsen et al. 1986; Gee et al. 2002; Creswell et al. 2008).

Air leakage from the pressure plate may also lead to inaccurate application of matric suction (Gee et al. 2002; Perez et al. 2008). Chahal and Yong (1965) reported that even small air leaks can result in a continuous loss of water vapour and the effect becomes more pronounced for tests performed over a longer period. Generally, air leakage results in desiccation of the soil specimen. In this case, after a prolonged period, the soil specimen becomes drier than a soil specimen that is properly equilibrated.

From the above-mentioned issues, determining equilibrium time for pressure plate test still remains a challenge (Marinho et al. 2008). From the literature, it appears that equilibrium of a soil-water system for pressure plate test may take days to weeks or longer in some circumstances. Recognising the long duration needed to prepare unsaturated soil specimens and to avoid inaccurate application of matric suction to the soil specimens, the equilibrium time of soil specimens in pressure plate test using independent matric suction measurement by high suction tensiometer was investigated as part of this study.

4.3.1.1 Material used

Two soil types were used: sand-kaolin mixture (SKM) and reconstituted Jurong Formation residual soil (NTU). The basic soil properties and soil preparation methods were described in Section 4.2. The consolidated sample was trimmed to specimens with a diameter of 50 mm, using a soil lathe. To investigate the effect of specimen height on the equilibrium time in pressure plate test, different heights of

20, 40, and 80 mm were used. Table 4.4 summarizes the initial conditions of soil specimens used in this study.

Table 4.4 Initial conditions of soil specimens used in this study

Soils	Specimen no.	Bulk density, ρ_b (Mg/m ³)	Void ratio, e	Initial water content, w_i (%)
Residual Soil (NTU)	NTU20	2.08	0.57	21.2
	NTU40	2.06	0.59	21.9
	NTU80	2.06	0.59	21.9
Sand-kaolin mixture (SKM)	SKM20-1	2.10	0.52	19.4
	SKM40-1	2.10	0.52	19.5
	SKM80-1	2.10	0.52	19.6
	SKM20-2	2.10	0.52	19.5
	SKM40-2	2.10	0.51	19.2
	SKM80-2	2.10	0.52	19.4

4.3.1.2 Pressure Plate Apparatus

NTU modified pressure plate apparatus (Figure 4.3) described by Leong et al. (2004) was used in this study. The consolidated soil specimens were placed on the saturated high air entry ceramic disk. Good contact between the soil specimens and the saturated ceramic disks was ensured by placing a 1 kg dead weight on the soil specimens. The pressure chamber was then sealed with the top lid by bolts and nuts. The air pressure in the pressure chamber was varied to give different matric suctions, ranging from 5 to 400 kPa. Since the pressure of the water in the water compartment was maintained at atmospheric pressure (i.e., the pore-water pressure, $u_w = 0$), the applied air pressure, u_a , gives the matric suction. The air pressure, u_a , was regulated at the required matric suction. Pressure plate tests were carried out in the laboratory where the environment was relatively stable (i.e, temperature was about 26 °C ±2 °C while the relative humidity (RH) was about 60% ±2%). The variation of temperature in the laboratory conforms to the requirements of ASTM D 6836-02 (2002) (±3 °C).

To enhance hydraulic conductance of soil-ceramic disk system, three ceramic disks with different air-entry values were used in this study. For low matric suctions of 5 and 200 kPa, all the specimens were tested in the pressure plate with a 3-bar ceramic disk. The specimens were then transferred to a 5-bar ceramic disk for

suctions of above 200 kPa. For wetting process, all the specimens were tested with a 2-bar ceramic disk. To monitor the evaporation rate in the pressure plate apparatus and the laboratory environment, beakers of water with surface area of 43 cm² were placed in and outside the pressure plate apparatus. The weights of the beakers of water were determined periodically.

During testing, the soil specimens' weights and dimensions were measured periodically. The specimens were removed from the pressure chamber and weighed with a weighing balance to an accuracy of 0.01g. At the same time, the water compartment was flushed by using the flushing pots to ensure that the water compartment is fully saturated (Leong et al. 2004). Air can diffuse through the ceramic plates and reappear in the water compartment during long test duration. To ensure hydraulic conductivity between soil specimen and water compartment, the surface of the ceramic plate was wetted before the specimens were placed onto the ceramic plate. At the end of the test, the specimens were oven-dried to determine the final water content. The gravimetric water contents, w , of the soil specimens at the end of each applied matric suction were back-calculated from the final water content and dry mass of the soil.

Figure 4.3 Modified pressure plate apparatus (after Leong et al. 2004).

4.3.1.3 High Suction Tensiometer (HST)

In order to independently measure the matric suction of the soil specimen in the pressure plate test with time, an NTU developed high suction tensiometer as shown in Figure 4.4 was employed. The working principle of the device is given in He et al. (2006). The main feature of the high suction tensiometer is its ability to achieve suction equilibrium in several minutes (typically 2-5 minutes). This is important as the soil specimens taken out from the pressure plate are susceptible to evaporation. In the laboratory, the evaporation rate was estimated to be about 46.5 g/h/mm^2 for a free water surface area of 43 cm^2 outside the pressure plate apparatus.

Figure 4.4 High suction tensiometer (after He et al. 2006).

4.3.1.4 Drying process

Figure 4.5 shows the typical drying behaviour of a soil specimen in pressure plate test. From Figure 4.5(a), the decrement of gravimetric water content, w with time was generally observed. The corresponding matric suction measurement is shown in Figure 4.5(b). The non-equilibrium condition of the soil specimen was attributed to evaporation effects during weighing, suction measurement, and during the test in the pressure plate apparatus. From Figure 4.5(c), it can be seen that the rate of gravimetric water content change ($\Delta w/\Delta t$) is high at the beginning of the test, and drops almost exponentially over time. Figure 4.5(d) shows the enlarged scale of the rate of gravimetric water content change ($\Delta w/\Delta t$) with time. It can be seen that the

drop of the rate of gravimetric water content change with time is more significant for longer measurement interval of the pressure plate specimens. The significant drop of $(\Delta w/\Delta t)$ is related to the longer measurement interval. A correction factor, CF , is needed to correct the difference in measurement interval.

Water loss from water beaker in the pressure plate apparatus was observed and the evaporation rate, H_e , was determined to be 11.6 g/day/mm², 7.0 g/day/mm², and 4.2 g/day/mm² for daily, 2-day-, and 13-day-measurement intervals, respectively, as shown in Figure 4.6. As water loss of the beaker was related to the measurement interval (i.e., frequency of opening the pressure chamber), a correction factor was obtained using the evaporation rate of the water beaker. The evaporation rate for different applied air pressures with respect to measurement interval is shown in Figure 4.7. From Figure 4.7, it can be seen that the evaporation rate decreases with longer measurement interval. Furthermore, the evaporation rate is not affected by the applied air pressure. To obtain an equivalent daily evaporation rate, a correction factor, CF is applied to the evaporation rate for measurement interval Δt . The correction factor, CF is given by:

$$CF = \frac{H_{e,1\text{ day}}}{H_{e,\Delta t}} \quad (4.1)$$

For $\Delta t \leq 1$ day, $CF = 1$, i.e. no correction is applied. For $\Delta t > 1$ day, CF increases with Δt . Note that CF in Equation 4.1 is dependent on the air volume of the pressure plate used and the humidity of the supplied air. As each laboratory has different pressure plate apparatus and compressed air supply, it is suggested that CF be calibrated independently using the procedures described above. The correction factor with different reading time intervals is illustrated in Figure 4.8 and given by:

$$CF = 0.6974 \ln \left(\frac{\Delta t}{1 \text{ day}} \right) + 1 \quad (4.2)$$

Figure 4.5 Variations of water content, measured matrix suction and, rate of gravimetric water content change with time for NTU40 during drying.

Figure 4.6 Evaporation loss from water beaker.

Figure 4.7 Evaporation rate at various measurement intervals.

Figure 4.8 Correction factor for different measurement intervals.

To obtain a corrected rate of gravimetric water content change, the correction factor in Eq. 4.1 was applied to the rate of gravimetric water content change as given by:

$$\text{Corrected rate of gravimetric water content change} = CF \left(\frac{\Delta w}{\Delta t} \right) \quad (4.3)$$

The corrected rate of gravimetric water content change for Figures 4.5(c) and 4.5(d) are replotted in Figures 4.9(c) and 4.9(d), respectively. The drop of the corrected rate of gravimetric water content change (Figure 4.9d) for longer reading measurement interval was reduced as compared to the Figure 4.5(d). Consequently, the corrected rate of gravimetric water content change appears to follow a decreasing trend with time more closely. Using the matric suction measurement (Figure 4.9b), the equilibrium times of the soil specimens for applied matric suctions 20, 50, 100, 200, 350 kPa were determined to be 7, 5, 3, 8, 5.5 days, respectively. To establish a relationship between equilibrium time and corrected rate of gravimetric water content change, the corresponding corrected rate of gravimetric water content change values at the equilibrium times 7, 5, 5, 8, 5.5 days were determined to be 0.11, 0.10, 0.08, 0.11 and 0.09 %/day, respectively in Figure

4.9(d). In order to have a larger database, the plots (gravimetric water content – time, measured matric suction – time, corrected rate of gravimetric water content change - time, enlarged scale of corrected rate of gravimetric water content change - time) were redrawn and the above procedure was repeated for other residual soil specimens and sand-kaolin mixture soil specimens with different heights (See Appendix B).

Table 4.5 summarizes the equilibrium time and the corresponding corrected rate of gravimetric water content change for the tested soil specimens. It can be seen in Table 4.5, the soil specimens in the pressure plate apparatus attained the applied matric suction within 1 – 8 days, irrespective of the specimen height tested. This finding is in agreement with previous works (Richards 1965; Klute 1986; Burke et al. 1986; Tinjum et al. 1997; Vanapalli et al. 1999; Leong et al. 2004; and Creswell et al. 2008). The equilibrium time of pressure plate specimens was also found to be independent of matric suction increment ranging from 5 to 302 kPa. However, this finding differs from Oliveira and Marinho (2006) who found that equilibrium of pressure plate specimen was attained around 3 days for matric suction increments of 50 and 100 kPa, and 45 days for matric suction increment of 380 kPa.

At matric suction equilibrium, the corrected rate of gravimetric water content change of the soil specimens varied from 0.06 to 0.33 %/day with an average value of 0.15 %/day. In this study, the average value of 0.15 %/day was suggested to be used as the criterion for the soil specimen to attain the applied matric suction in the pressure plate apparatus for drying process, irrespective of the soil specimen height. The performance of the criterion and the associated error will be presented later in this chapter.

(a) Gravimetric water content – time plot

(c) Corrected rate of gravimetric water content change – time plot

(b) Measured matrix suction – time plot

(d) Corrected rate of gravimetric water content change – time plot (Enlarged)

Figure 4.9 Variations of water content, measured matrix suction and, corrected rate of gravimetric water content change with time for NTU40 during drying.

Table 4.5 Equilibrium conditions of the tested soil specimens for drying process

Soil Specimen	Matric suction (kPa)	Time to reach applied matric suction as measured by HST (day)	Corresponding CF.Δw/Δt (%/day)
NTU20	20	7	0.10
	50	5	0.09
	100	2.5	0.15
	200	7	0.13
	350	8	0.06
NTU40	20	7	0.11
	50	5	0.10
	100	3	0.08
	200	8	0.11
	350	5.5	0.09
NTU80	20	8	0.09
	50	3.5	0.15
	100	2	0.10
	200	8	0.08
	350	6	0.07
SKM20-1	5	2	0.27
	20	2	0.27
	50	2	0.21
	100	1	0.26
	200	2	0.20
SKM40-1	400	6	0.10
	5	3	0.16
	20	3	0.22
	50	3	0.15
	100	2	0.29
SKM80-1	200	2.5	0.16
	400	7	0.13
	5	3	0.22
	20	3	0.20
	50	3	0.13
SKM20-2	100	3	0.09
	200	1	0.19
	400	6	0.11
	5	3	0.14
	20	3	0.24
SKM40-2	50	2	0.27
	100	2	0.13
	200	1	0.33
	400	8	0.13
	5	4	0.13
SKM80-2	20	3	0.20
	50	3	0.12
	100	3	0.10
	200	2	0.14
	400	8	0.14
Average			0.15

4.3.1.5 Wetting Process

Similar to drying process, test results of soil specimens in the wetting process are presented in four plots (gravimetric water content – time, measured matric suction – time, corrected rate of gravimetric water content change - time, enlarged scale of corrected rate of gravimetric water content change - time). Figure 4.10 shows the typical wetting behaviour of a soil specimen in pressure plate test. From Figure 4.10(a), it can be seen that the increment of gravimetric water content, w with time was generally observed. The corresponding matric suction measurement is shown in Figure 4.10(b). Figures 4.10(c) and 4.10(d) show the corrected rate of gravimetric water content change and enlarged scale of corrected rate of gravimetric water content change, respectively. Similar to drying process, the corrected rate of gravimetric water content change is high at the beginning of the test, and decreases exponentially with time.

Using the matric suction measurement (Figure 4.10b), the equilibrium times of the soil specimens for applied matric suctions 100 and 25 kPa were determined to be 5 and 3 days, respectively. To establish a relationship between equilibrium time and corrected rate of gravimetric water content change, the corresponding corrected rate of gravimetric water content change values at the equilibrium times 5 and 3 days were determined to be 0.08 and 0.06 %/day, respectively in Figure 4.10(d). In order to have a larger database, the above procedure was repeated for other residual soil specimens and sand-kaolin mixture soil specimens with different heights (See Appendix C).

At matric suction equilibrium, the corrected rate of gravimetric water content change of the soil specimens varied from 0.02 to 0.16 %/day with an average value of 0.08 %/day. In this study, the average value of 0.08 %/day was suggested to be used as the criterion for the soil specimen to attain the applied matric suction in the pressure plate apparatus for wetting process, irrespective of the soil specimen height. This value is about half of that of the drying process. The performance of the criterion will be presented later in this chapter.

Table 4.6 Equilibrium conditions of the tested soil specimens for wetting process

Soil Specimen	Matric suction (kPa)	Time to reach applied matric suction as measured by HST (day)	Corresponding $CF.\Delta w/\Delta t$ (%/day)
NTU20	100	6	0.02
	25	3	0.08
NTU40	100	5	0.08
	25	3	0.06
NTU80	100	6	0.03
	25	3	0.14
SKM20-1	200	4	0.12
	50	4	0.10
SKM40-1	200	6	0.16
	50	4	0.03
SKM80-1	200	6	0.16
	50	4	0.02
SKM20-2	200	4	0.15
	50	7	0.08
SKM40-2	200	6	0.02
	50	8	0.02
SKM80-2	200	6	0.02
	50	5.5	0.09
Average			0.08

4.3.1.6 Effect of Specimen Height on Soil-Water Characteristic Curve

Figure 4.11 shows the effect of specimen height on soil-water characteristic curve for the residual soil specimens. Both the drying and wetting curves tend to be higher for thicker specimens (i.e. higher water content was observed for thicker specimen for a given applied matric suction). An attempt was made to investigate the water distribution over the specimen. Three specimens with a diameter of 50 mm and heights of 20, 40, and 80 mm were tested under an applied matric suction of 200 kPa from saturated condition. The specimens were weighed, followed by matric suction measurement at their base (i.e. contact area between soil and ceramic disk in pressure plate). Suction measurement was also performed on the top surface of the soil specimens. The matric suctions measured at both the top and base of the soil specimens are shown in Figure 4.12. Figure 4.12 shows that the measured matric suction at the top surface is about 1 to 6% higher than that measured at the bottom surface of the specimen, not related to specimen height. This suggests that the specimens have a rather uniform matric suction distribution. After six days, all the specimens were cut into thin layers (Table 4.7) before they were oven-dried. The

water contents of the thin layers are summarized in Table 4.7. From Table 4.7, it can be seen that the difference in water content over the specimens was negligible, indicating that the specimens have a near uniform water distribution, regardless of the specimen height.

Figure 4.11 Effect of specimen height on SWCC of reconstituted NTU residual soil.

Figure 4.12 Comparison of matric suction measured at the top and bottom surfaces of the soil specimens

Table 4.7 Division of specimen for water content determination

20 mm (w = 16.0 %)	Top: 10 mm (w = 16.0 %)	Outer (w = 15.9 %)
		Inner: (30 x 30 mm) (w = 16.0 %)
	Bottom: 10 mm (w = 16.1 %)	Outer (w = 16.0 %)
		Inner (30 x 30 mm) (w = 16.1 %)
40 mm (w = 16.2 %)	Top: 10 mm (w = 16.1 %)	Outer (w = 16.0 %)
		Inner: (30 x 30 mm) (w = 16.1 %)
	Middle: 20 mm (w = 16.2 %)	Outer (w = 16.1 %)
		Inner: (30 x 30 mm) (w = 16.3 %)
	Bottom: 10 mm (w = 16.2 %)	Outer (w = 16.1 %)
		Inner: (30 x 30 mm) (w = 16.2 %)
80 mm (w = 16.7 %)	Top: 10 mm (w = 16.6 %)	Outer (w = 16.5 %)
		Inner: (30 x 30 mm) (w = 16.7 %)
	Middle: 60 mm (w = 16.7 %)	Outer (w = 16.6 %)
		Inner: (30 x 30 mm) (w = 16.8 %)
	Bottom: 10 mm (w = 16.6 %)	Outer (w = 16.5 %)
		Inner: (30 x 30 mm) (w = 16.7 %)

Figure 4.13(a) shows the water content with time for the soil specimens. Figure 4.13(b) shows that all the specimens reached the applied matric suction within 2.5 to 3.5 days. The difference in water content observed in the three soil specimens is attributed to errors in the determination of the water content. In order to achieve a water content determination accuracy of $\pm 0.1\%$, for a soil with maximum particle size passing 9.5 mm sieve (i.e., this sample), the minimum moist specimen mass shall be 500g (ASTM D2216-10 2010). In this study, the specimens of a height of 20, 40, and 80 mm have moist mass of only about 80, and 160 and 320g, respectively. As such, the water content determination accuracy for all the three specimens is only up to $\pm 1\%$ (ASTM D2216-10 2010). The error in water content determination becomes smaller for thicker specimen as the moist mass approaches the minimum required soil mass for an accuracy of $\pm 0.1\%$ water content determination as suggested by ASTM D2216-10 (2010). At equilibrium, all the soil

specimens would have a similar water content within the water content determination error of $\pm 1\%$. For the three soil specimens, the maximum difference in water content is only about 0.54%, which is within the accuracy in water content determination i.e. $\pm 1\%$.

(a) Gravimetric water content – time plot

(b) Measured matric suction – time plot

Figure 4.13 Variations of water content and measured matric suction for specimens subjected to matric suction of 200 kPa from saturated condition.

Figure 4.14 shows the effect of specimen height on soil-water characteristic curve for the sand-kaolin mixture. It can be seen that the data is less scattered for the sand-kaolin mixture than the residual soil. This can be attributed to less error in water content determination. The tested soil has a maximum particle size passing 4.75 mm sieve (i.e., sand-kaolin mixture). Therefore, the minimum moist specimen mass needed to achieve an accuracy of $\pm 0.1\%$ is 100g (ASTM D2216-10 2010). In this study, the specimens of a height of 20, 40, and 80 mm have moist mass of about 80, and 160 and 320g, respectively. As such, the water content determination accuracy for specimens of a height of 20 mm is $\pm 1\%$ and a height of 40 and 80 mm is $\pm 0.1\%$ (ASTM D2216-10 2010). In Figure 4.14, it can be seen that the specimen size has negligible effect on SWCC determination and the subtle variation of water content for a given matric suction is due to error in water content determination.

Figure 4.14 Effect of specimen height on SWCC of sand-kaolin mixture

4.3.1.7 Evaluation of the Proposed Criteria

The performance of the proposed criteria was evaluated using data from published literature. There is very limited data (water content - time and water content -

matric suction plots under drying or wetting process) availability on the pressure plate tests. Data from Leong et al. (2004) were used for evaluation. A total of 12 data point for drying curve and 10 data points for wetting curve were used in the evaluation stage. The soil used is a compacted residual specimen containing 60% of fines and plasticity index of 14%. The increment matric suction ranges from 20 to 100 kPa for drying process whereas the decrement matric suction ranges from 20 to 300 kPa for wetting process.

Both drying and wetting tests from Leong et al. (2004) are presented in three plots (gravimetric water content – time, measured matric suction – time, corrected rate of gravimetric water content change - time, enlarged scale of corrected rate of gravimetric water content change - time). Figure 4.15 shows the variations of water content and corrected rate of gravimetric water content change with time for S1 soil specimen during drying. Using the proposed criterion (corrected rate of gravimetric water content change of 0.15 %/day) in Figure 4.15(c), the “equilibrium” times for S1 soil specimen equilibrated at applied matric suctions 50, 100, 200, 300, 400, and 500 kPa were determined to be 1.5, 2.6, 3.1, 7, 4.5, and 5 days, respectively. The corresponding gravimetric water contents at the “equilibrium” times were determined to be 21.0, 20.0, 19.1, 17.1, 15.4, and 13.0%, respectively, in Figure 4.15(a). The above procedure was repeated for S2 soil specimen in Figure 4.16. Figure 4.17 shows the variations of water content and corrected rate of gravimetric water content change with time for S1 soil specimen during wetting. Using the proposed criterion (corrected rate of gravimetric water content change of 0.08 %/day) in Figure 4.17(c), the “equilibrium” times for S1 soil specimen equilibrated at applied matric suctions 400, 300, 200, 100, 50, and 30 kPa were determined to be 4, 4.3, 4, 5, 5.5, and 4.8 days, respectively. The corresponding gravimetric water contents at the “equilibrium” times were determined to be 13.5, 14.3, 14.7, 16.4, 18.0, and 19.2%, respectively, in Figure 4.17(a). The above procedure was repeated for S2 soil specimen in Figure 4.18.

(a) Gravimetric water content – time plot

(b) Corrected rate of gravimetric water content change – time plot

(c) Corrected rate of gravimetric water content change – time plot (Enlarged)

Figure 4.15 Variations of water content and corrected rate of gravimetric water content change with time for S1 during drying.

(a) Gravimetric water content – time plot

(b) Corrected rate of gravimetric water content change – time plot

(c) Corrected rate of gravimetric water content change – time plot (Enlarged)

Figure 4.16 Variations of water content and corrected rate of gravimetric water content change with time for S2 during drying.

(a) Gravimetric water content – time plot

(b) Corrected rate of gravimetric water content change – time plot

(c) Corrected rate of gravimetric water content change – time plot (Enlarged)

Figure 4.17 Variations of water content and corrected rate of gravimetric water content change with time for S1 during wetting.

(a) Gravimetric water content – time plot

(b) Corrected rate of gravimetric water content change – time plot

(c) Corrected rate of gravimetric water content change – time plot (Enlarged)

Figure 4.18 Variations of water content and corrected rate of gravimetric water content change with time for S2 during wetting.

Table 4.8 summarizes the “equilibrium” time for pressure plate specimens subjected to drying and wetting processes, using the proposed criteria. For drying test, it took about 1.5 to 7 days to attain “equilibrium” whereas for wetting test, it took about 3.2 to 5.5 days to attain “equilibrium”. Using the proposed criteria, the corresponding gravimetric water contents at the “equilibrium” time were re-plotted and compared to the published SWCC in Figure 4.19. Figure 4.19 shows that there was negligible difference between the SWCC constructed using the proposed criteria and the published SWCC. This suggests that the proposed criteria provide a useful guide for an operator to stop a test and proceed to the next matric suction.

Table 4.8 Summary of “equilibrium” time of published data by Leong et al. (2004) using the proposed criteria

Process	$(u_a - u_w)$, (kPa)	$\Delta(u_a - u_w)$, (kPa)	S1		S2	
			“Equilibrium” time using proposed criterion	Corresponding water content (%)	“Equilibrium” time using proposed criterion	Corresponding water content (%)
Drying	50	20	1.5	21.0	1.7	21.3
	100	50	2.6	20.0	3	20.2
	200	100	3.1	19.1	3	19.2
	300	100	7	17.1	6	17.5
	400	100	4.5	15.4	5	15.6
	500	100	5	13.0	4	12.8
Wetting	400	100	4	13.5	-	-
	300	100	4.3	14.3	-	-
	200	100/300*	4	14.7	4	14.8
	100	100	5	16.4	4	16.7
	50	50	5.5	18.0	3.2	18.1
	30	20	4.8	19.2	5.2	19.5

Notes: * $\Delta(u_a - u_w) = 300$ kPa for S2, wetting from matric suction of 500 kPa to 200 kPa.

Figure 4.19 Comparison of SWCC obtained using proposed criterion and published data.

4.3.2 Preparation of Unsaturated Soil Specimen for Cyclic Simple Shear Test

4.3.2.1 Material used

To investigate the cyclic simple shear properties of residual soil specimens under a known matric suction condition, nine residual soil specimens were used. The soil specimens were prepared from undisturbed AMK and JM residual soils and reconstituted NTU residual soils. The basic soil properties and soil sampling have been described in Section 4.2. The soil samples were trimmed to specimens with a diameter of 70 mm and a height of 20 mm, using a soil lathe. Table 4.9 summarizes the initial condition of the residual soil specimens used.

Table 4.9 Initial condition of residual soil specimen used

Residual Soils	Specimen no.	w_l (%)	Void ratio, e	ρ_b (Mg/m ³)
Undisturbed AMK	AMK50-100	41.8	1.09	1.80
	AMK50-200	41.8	1.09	1.80
	AMK50-400	41.8	1.09	1.80
Undisturbed JM	JM50-400	12.1	0.34	2.26
	JM50-400	12.1	0.34	2.26
	JM50-400	12.1	0.34	2.26
Reconstituted NTU	NTU100-100	20.3	0.54	2.10
	NTU100-200	20.3	0.54	2.10
	NTU100-400	20.3	0.54	2.10

4.3.2.2 Procedures of Unsaturated Soil Specimen Preparation

A series of increasing matric suctions (i.e., 100, 200 and 400 kPa) were applied to each soil specimen. The modified pressure plate as described in Section 4.3.1.3 was used to apply the desired matric suction to the residual soil specimens. Pressure plate test was conducted following the procedures described in Section 4.3.1.3. The NTU-developed high suction tensiometer as described in Section 4.3.1.4 was also used to independently measure the matric suction of the soil specimen equilibrated in pressure plate apparatus. After the soil specimens reached the matric suction equilibrium, they were transferred to cyclic simple shear apparatus for tests. To evaluate the performance of the proposed equilibrium criterion (corrected rate of gravimetric water content change of 0.15 %/day), the data obtained in this section were used.

4.3.2.3 Equilibrium of Residual Soil Specimens

All the soil specimens equilibrated in pressure plate apparatus are presented in three plots: gravimetric water content - time, measured matric suction - time, and corrected rate of gravimetric water content change - time plots. Figures 4.20, 4.21, and 4.22 show the three plots for the undisturbed AMK, undisturbed JM, and reconstituted NTU residual soil specimens, respectively. Using the proposed criterion (corrected rate of gravimetric water content change of 0.15 %/day) in Figure 4.20(c), the “equilibrium” times for AMK soil specimens equilibrated at applied matric suctions of 100, 200, and 400 kPa were determined to be 4.3, 5.5, and 5 days, respectively. The corresponding matric suctions at the “equilibrium” times were determined to be 96, 192, and 400 kPa, respectively, in Figure 4.20(b). For JM soil specimens, the “equilibrium” times equilibrated at applied matric suctions of 100, 200, and 400 kPa were determined to be 2, 1.8, and 3 days, respectively, using the proposed equilibrium criterion in Figure 4.21(c). The corresponding matric suctions at the “equilibrium” times were determined to be 98, 195, and 360 kPa, respectively, in Figure 4.21(b). For NTU soil specimens, the “equilibrium” times equilibrated at applied matric suctions of 100, 200, and 400 kPa were determined to be 2.9, 4, and 6 days, respectively, using the proposed equilibrium criterion in Figure 4.22(c). The corresponding matric suctions at the “equilibrium” times were determined to be 98, 188, and 375 kPa, respectively, in Figure 4.22(b).

Table 4.10 summarizes the “equilibrium” condition of the residual soil specimens using the proposed criterion while Figure 4.23 shows the plot of measured matric suction at equilibrium time determined from the proposed criterion against applied matric suction. It can be seen that using the proposed criterion, the soil specimens would have attained a matric suction that is about $\pm 10\%$ of the applied matric suction. This suggests that the proposed equilibrium criterion can be used with reasonable confidence, in the event where high suction tensiometer is not available to measure the matric suction.

Figure 4.20 Variations of water content, measured matric suction, and corrected rate of gravimetric water content change with time for AMK soil specimens.

Figure 4.21 Variations of water content, measured matric suction, and corrected rate of gravimetric water content change with time for JM soil specimens.

(a) Gravimetric water content – time plot

(b) Measured matric suction – time plot

(c) Corrected rate of gravimetric water content change – time plot

Figure 4.22 Variations of water content, measured matric suction, and corrected rate of gravimetric water content change with time for NTU soil specimens.

Table 4.10 “Equilibrium” condition of the residual soil specimens using the proposed criterion

Residual soils	Specimen no.	Applied matric suction, $(u_a - u_w)_a$ (kPa)	“Equilibrium” time using proposed criterion (day)	Measured matric suction, $(u_a - u_w)_m$ (kPa)
Undisturbed AMK	AMK50-100	100	4.3	96
	AMK50-200	200	5.5	192
	AMK50-400	400	5.0	400
Undisturbed JM	JM50-400	100	2.0	98
	JM50-400	200	1.8	195
	JM50-400	400	3.0	360
Reconstituted NTU	NTU100-100	100	2.9	98
	NTU100-200	200	4.0	188
	NTU100-400	400	6.0	375

Figure 4.23 Performance of the proposed criterion.

4.4 Summary

This chapter presented the soil materials used in this study. A method of using pressure plate apparatus to apply the matric suction to the residual soil specimens for cyclic simple shear tests has also been described.

To eliminate uncertainty with regards to the equilibrium matric suction in pressure plate apparatus, an investigation has been carried out on fine-grained reconstituted residual soil specimens and coarse-grained sand-kaolin mixture to study equilibrium time in a pressure plate test and the effect of specimen height on the equilibrium time using a high suction tensiometer. For drying curve, matric suctions ranging from 5 kPa – 400 kPa and an increment of matric suction ranging from 5 kPa – 302 kPa were tested whereas for wetting curve, matric suctions ranging from 25 kPa – 200 kPa and a decrement of matric suction ranging from 75 kPa - 279 kPa were tested. Using matric suction measurement from high suction tensiometer, the equilibrium time for the tested soil specimens was found to be 1 to 8 days for drying process, and 3 to 8 days for wetting process, irrespective of the specimen height. Using a combination of pressure plate test and high suction tensiometer measurement, corrected rates of gravimetric water content change of 0.15 %/day and 0.08 %/day were suggested as criteria to determine “equilibrium” condition in the pressure plate test for drying and wetting processes, respectively.

Applicability of the proposed criterion for drying process was evaluated using unsaturated residual soil specimens for cyclic simple shear tests. The measured matric suctions at the “equilibrium” time determined from the proposed criterion showed a maximum difference of 10% of the applied matric suctions. This suggests that the proposed equilibrium criterion can be used with reasonable confidence, in the event where high suction tensiometer is not available to measure the matric suction.

The experiments conducted in this chapter were attempted to show the effects of soil type and specimen height on equilibrium time, however the experimental results showed that equilibrium times for pressure plate soil specimens were not

affected much by soil type, specimen height, suction range or wetting or drying path. Plausible explanations for these observations are given below:

a) Specimen height

Soil specimens with different heights of 20 mm, 40 mm, and 80 mm were used. To impose matric suction on the soil specimens in pressure plate apparatus, air pressure was applied to the soil specimens using axis-translation method. As the height of soil specimen increased, the exposed side area of the soil specimen subjected to the applied air pressure also increased. As the imposed suction gradient was not one dimensional along the specimen's height, it is plausible that the equilibrium time of pressure plate soil specimen was not much affected by the specimen height.

b) Soil type

It is well known that hydraulic conductivity decreases with soil suction (Gee et al. 2002). The reduced hydraulic conductivity is particularly significant for coarse-grained soil with low air-entry value (AEV) which fully desaturates at a relatively low matric suction value. The low hydraulic conductivity impedes the drainage of the water, thus resulting in longer equilibrium time needed in a pressure plate test. The current study investigated two soil types (residual soil having $PI=18\%$, $P_{200}=54\%$; and sand-kaolin mixture having $PI=16\%$, $P_{200}=37\%$). As the indices (e.g. PI and P_{200}) were not significantly different for both soils and furthermore the equilibrium time of pressure plate test was measured in term of days and not minutes and seconds, it can be expected that equilibrium time in pressure plate tests was not significantly different.

c) Suction range

Matric suctions ranging from 5 to 400 kPa were used in the current study. ASTM D 6836-02 (2002) established the equilibrium criteria based on three zones of matric suction level. Equilibrium is said to be achieved if there is no outflow for at least 24 hours, 48 hours, and 96 hours, for matric suction less than 500 kPa (e.g. zone 1), between 500 kPa and 1000 kPa (e.g. zone 2), and greater than 1000 kPa (e.g. zone 3), respectively.

As the matric suctions used were within zone 1 (i.e. < 500 kPa), where the equilibrium criterion of no water outflow for at least 24 hours was suggested by ASTM 6836 – 02 (2002), it is plausible that equilibrium time does not vary much.

d) Drying/wetting path

The equilibrium time for the soil specimens used ranged from 1 to 8 days for drying path and 3 to 8 days for wetting path. It appears that a longer equilibrium time was needed for the wetting path. However, the equilibrium time for the wetting path was not significantly different from the equilibrium time for the drying path as it was measured in term of day.

Chapter 5

Soil-Water Characteristic Curve using the One-Point Measurement Method

5.1 General

Research has shown that the soil-water characteristic curve (SWCC) of a soil can be used as a proxy to determine a number of unsaturated soil parameters such as shear strength (Vanapalli et al. 1996), coefficient of permeability (Millington and Quirk 1961; Mualem 1976) and coefficient of water volume change (Fredlund and Rahardjo 1993). However, it is time-consuming and laborious to measure a complete SWCC. This chapter describes a method to estimate the SWCC based on a one-point measurement method. The method is first illustrated, and the development of the method is then described. The proposed method is evaluated and compared with other methods using independent data from published literature. Finally, the proposed method is used to estimate the SWCC of the soils used for cyclic simple shear tests. The three distinct zones of SWCC are defined.

Most of the materials in this chapter have been published in Chin et al. (2010) and much of the text is taken verbatim from that paper.

5.2 Nature of Soil-Water Characteristic Curve and Its Relation to the Mechanical Behaviour of Soil

A typical SWCC for a wide range of matric suction values (i.e., 0 to 1 GPa) is illustrated in Figure 5.1. In Figure 5.1, gravimetric water content, w is plotted against matric suction. However, a similar plot can be drawn either with volumetric water content, or degree of saturation, S , against matric suction. The relationship between w and S is given as:

$$w = \frac{Se}{G_s} \quad (5.1)$$

where e is the void ratio and G_s is the specific gravity of the soil.

The relationship between w and θ_w is given as:

$$w = \frac{\theta_w}{\rho_d} \quad (5.2)$$

where ρ_d is the dry density of the soil.

SWCC was first divided into three zones by White et al. (1970) to present a justification for the interpretation of saturation and matric suction in porous material, and thus a theoretical relationship. Figure 5.1 shows the three distinct zones of a drying SWCC: boundary effect zone, transition zone, and residual zone. The air-entry value subdivides the boundary effect zone and transition zone while residual value subdivides the transition zone and residual zone. In the boundary effect zone, almost all the soil pores are filled with water. The soil is saturated and the air phase is essentially in occluded form, if it exists. The water phase is continuous in the boundary effect zone. The soil desaturates at the air-entry value as it enters the transition zone. In this zone, the flow of water is in the liquid phase and the soil desaturates rapidly with increasing matric suction. The soil continues to desaturate until it enters the residual zone. In this zone, the water phase loses its continuity and large increases in matric suction only lead to relatively small changes in water content.

Vanapalli (1994) has shown the probable increase in shear stress with matric suction for the three distinct zones in SWCC (Figure 5.2). In boundary effect zone, the shear stress increases linearly with increase in matric suction until air-entry value. Thereafter, as it enters transition zone, the increase of shear stress reduces with matric suction. In residual zone, there is no increase of shear stress and the shear stress becomes constant with matric suction. Analogous to increase in shear stress with matric suction is the increase in shear modulus (shear stress is divided by shear strain) with matric suction. Therefore, it is necessary to identify the three

zones in a SWCC of the soils when interpreting cyclic simple shear tests for unsaturated soils.

Figure 5.1 Three distinct zones of a typical SWCC (modified from Fredlund 2006).

Figure 5.2 Probable variation of shear strength in three distinct zones (modified from Vanapalli 1994).

5.3 Estimation of Soil-Water Characteristic Curve

In soil science, many attempts have been made to estimate SWCC based on soil texture and grain size distribution (Gupta and Larson 1979; Arya and Paris 1981; Rawls et al. 1982; Ahuja et al. 1985; Saxton et al. 1986). More recently, Saxton and Rawls (2006) have incorporated additional variables such as organic matter, density, gravel content and salinity into SWCC estimation equations. Most of these estimations are based on regression and statistical approaches. Table 5.1 shows the SWCC estimation equations in soil science. In geotechnical engineering, a number of SWCC estimation methods have been proposed based on grain size distribution, index properties (Fredlund et el. 1997, 2002; Zapata 1999; Perera 2003; Sung et al. 2006) and pore-size distribution (Simms and Yanful 2002).

Table 5.1 Estimation of SWCC in soil science

Reference	Equations
Gupta and Larson (1979)	$\theta_w = C_1 \times \text{sand (\%)} + C_2 \times \text{silt (\%)} + C_3 \times \text{clay (\%)} + C_4 \times \text{organic matter (\%)} + C_5 \times \text{bulk density (g/cm}^3\text{)}$
Rawls et al. (1982)	$\theta_w = C_1 + C_2 \times \text{sand (\%)} + C_3 \times \text{silt (\%)} + C_4 \times \text{clay (\%)} + C_5 \times \text{organic matter (\%)} + C_6 \times \text{bulk density (g/cm}^3\text{)}$
Saxton et al. (1986)	$\theta_w = \theta_s \text{ for } (u_a - u_w) \leq (u_a - u_w)_{AEV}$ $\theta_w = \frac{[10 - (u_a - u_w)](\theta_s - \theta_{w(10)}) + \theta_{w(10)}[10 - (u_a - u_w)_{AEV}]}{[10 - (u_a - u_w)_{AEV}]}$ $\text{for } (u_a - u_w)_{AEV} < (u_a - u_w) < 10 \text{ kPa}$ $\theta_w = \left[\frac{(u_a - u_w)}{A} \right]^{\frac{1}{B}} \text{ for } (u_a - u_w) \geq 10 \text{ kPa}$

Note : C_1, C_2, C_3, C_4, C_5 and C_6 are the multiple regression coefficients; $(u_a - u_w)_{AEV}$ is the air-entry value of soil; $\theta_{w(10)}$ is the volumetric water content at $(u_a - u_w) = 10 \text{ kPa}$; A and B are constants obtained from regression statistical analyses; and textures defined by the United States Department of Agriculture (USDA) system (Soil Survey Staff 1975).

More recent development of SWCC estimation method in geotechnical engineering has been extended from solely based on basic index properties to basic index properties coupled with one point SWCC measurement. Vanapalli and Catana (2005) presented a method to estimate the SWCC of coarse-grained soils using one point SWCC measurement and basic soil properties. Vanapalli and Catana (2005) defined coarse-grained soils in accordance with the Unified Soil Classification System (USCS). In their study, the basic soil parameters of 14 soils were correlated with the parameters (a , n , m) of Fredlund and Xing (1994) equation coupled with one point SWCC measurement in the suction range of 0.1 to 10 kPa were used to estimate SWCC. The correlated parameters are:

$$a = \frac{1.33}{(d_e)^{0.86}} \quad kPa \quad (5.3)$$

$$n = \frac{7.78}{\left(\frac{D_{60}}{D_{10}} \times e\right)^{1.14}} \quad (5.4)$$

$$m = x \quad (5.5)$$

where D is the soil particle diameter in mm corresponding to the percent of particle passing given as the subscript; e is the void ratio; x is the adjustable variable for SWCC to pass through or come close to the measured SWCC point; and d_e is the dominant particle size diameter as given by Vukovic and Soro (1992):

$$\frac{1}{d_e} = \sum_{i=1}^{i=n} \Delta g_i \frac{\ln\left(\frac{d_i^g}{d_i^d}\right)}{(d_i^g - d_i^d)} \quad (5.6)$$

where Δg_i is the fraction weight in parts of the total weight; d_i^g is the maximum grain diameter of the corresponding fraction; and d_i^d is the minimum grain diameter of the corresponding fraction. Figure 5.3 illustrates the procedure of calculating the dominant particle size diameter, d_e , from Vukovic and Soro (1992).

Figure 5.3 Procedures of obtaining dominant particle size diameter (modified from Vukovic and Soro 1992).

For non-plastic soils (i.e., soils with zero plasticity index), SWCC estimation using one-point SWCC measurement coupled with basic index properties has been proposed by Houston et al. (2006). Houston et al. (2006) adopted a similar correlation approach as Vanapalli and Catana (2005) but with a larger database consisting of non-plastic soils. The correlated parameters $[a, n, m, (u_a - u_w)_r]$ for Fredlund and Xing (1994) equation are summarized as follows:

$$a = 1.14a_1 - 0.5 \quad kPa \quad (5.7)$$

$$a_1 = -2.79 - 14.1 \log(D_{20}) - 1.9 \times 10^{-6} P_{200}^{4.34} + 7 \log(D_{30}) + 0.055 D_{100} \quad (5.8)$$

$$D_{100} = 10^{\frac{4 \log(D_{90}) - \log(D_{60})}{3}} \quad (5.9)$$

$$n = 0.936n_1 - 3.8 \quad (5.10)$$

$$n_1 = \left(5.39 - 0.29 \ln \left(P_{200} \frac{D_{90}}{D_{10}} \right) + 3D_0^{0.57} + 0.021P_{200}^{1.19} \right) \left(\frac{30}{\log(D_{90}) - \log(D_{60})} \right)^{0.1} \quad (5.11)$$

$$D_0 = 10^{\frac{3 \log(D_{10}) - \log(D_{30})}{2}} \quad (5.12)$$

$$m = 0.26e^{0.758m_1} + 1.4D_{10} \quad (5.13)$$

$$m_1 = \log \left(\frac{20}{\log(D_{30}) - \log(D_{10})} \right)^{1.15} - 1 + \frac{1}{n} \quad (5.14)$$

$$(u_a - u_w)_r = 100 \quad kPa \quad (5.15)$$

where P_{200} is the percent of soil passing standard sieve #200 and PI is the plasticity index.

To the author's understanding, there are two possible ways to estimate the SWCC of non-plastic soils using Equations 5.7 – 5.15. The first method involves shifting the particle size distribution curve to the right or left, so that the estimated SWCC passes through the measured SWCC point. The second method involves shifting the SWCC such that it passes through the measured point. However, the suitable suction range for the one point SWCC measurement was not reported.

For compacted fine-grained soils, Catana et al. (2006) proposed that the SWCC be estimated using one point SWCC measurement in the range of 50 - 500 kPa and one-point estimation of the SWCC using suction capacity, C , which is correlated with the product of liquid limit, LL and clay fraction, C_f given as:

$$C_f = 0.12\beta + 4.5 \quad (5.16)$$

where $\beta = LL \times C_f$.

Catana et al. (2006) defined fine-grained soils in accordance with the Unified Soil Classification System (USCS). Suction capacity, C , is defined as rate of desorption or amount of moisture loss per unit change of matric suction on one logarithmic scale of the drying SWCC. Once C has been determined from Equation 5.16, the desorption line with a gradient C can be drawn. Catana et al. (2006) recommended that the desorption line be located such that it intersects the matric suction axis between 300 and 500 MPa. After the desorption line has been located, one-point estimation of SWCC can be established at the intersection of a matric suction in the range of 1000 to 3000 kPa and the desorption line as shown in Figure 5.4. The one

point SWCC measurement in the range 50-500 kPa coupled with one point estimation of SWCC in the range 1000-3000 kPa were fitted using the two-parameter Brutsaert (1966) equation which is given as:

$$\theta_w = \theta_r + (\theta_s - \theta_r) \frac{1}{1 + \left[\frac{(u_a - u_w)}{a_b} \right]^{n_b}} \quad (5.17)$$

where a_b and n_b are fitting parameters.

Figure 5.4 Illustration of SWCC estimation for fine-grained soils (adapted from Catana et al. 2006). Ψ_p , intercept of the desorption line on the suction axis.

For estimation of SWCC of plastic soils (i.e., soils with plasticity index greater than zero), Houston et al. (2006) used a product of PI and P_{200} coupled with one point SWCC measurement. This method works best when the product of PI and P_{200} is adjusted until the SWCC curve passes through the one-point SWCC measurement. However, the optimal location of the one-point SWCC measurement was not reported. Fredlund and Xing (1994) equation was used and the fitting parameters are given as:

$$a = 32.835 \ln(P_{200} \times PI) + 32.438 \quad kPa \quad (5.18)$$

$$n = 1.421(P_{200} \times PI)^{-0.3185} \quad (5.19)$$

$$m = -0.2154 \ln(P_{200} \times PI) + 0.7145 \quad (5.20)$$

$$(u_a - u_w)_r = 500 \quad kPa \quad (5.21)$$

5.3.1 Database

To date, a number of soil databases with SWCC are available commercially as well as in the public domain. Dutch and Scottish soil databases have been used by Koekkoek and Bootink (1999) to develop their ANN for SWCC estimation. The Unsaturated Hydraulic Database (UNSODA) (Nemes et al. 2001) is widely used in soil science for unsaturated soil modeling. In general, UNSODA database contains the following properties from 790 soil horizons: grain size distribution, texture, bulk density, specific gravity, porosity, organic matter, saturated volumetric water content, saturated and unsaturated coefficient of permeability. In the UNSODA database, there are a total of 730 soils with laboratory drying SWCC data.

SoilVision (2002) has a larger soil database consisting over 6000 soil horizons. Information such as grain size distribution, texture, bulk density, porosity, specific gravity, saturated volumetric water content, saturated and unsaturated coefficient of permeability in the SoilVision database are almost similar to the UNSODA database. However, both of the databases do not contain Atterberg limits (i.e., LL and PI). There are 878 soils that contain both SWCC and grain size distribution data in the SoilVision database. In this study, the SoilVision database is used to develop the proposed method.

5.3.2 Approach to Develop a Simplified Method to Estimate the SWCC

Most proposed empirical SWCC equations have two common problems (Fredlund 2006). Firstly, most empirical equations become asymptotic to a horizontal line in the low suction range which means the coefficient of water volume change with respect to a change in matric suction, m_2^w , approaches zero. Using such a SWCC equation for numerical modelling in the low suction range will lead to numerical

instability and is construed as incorrect. Secondly, most empirical equations become asymptotic to a horizontal water content line going to infinity at high suctions beyond residual condition which is unreasonable. However, these problems have been overcome by Fredlund and Xing (1994) equation. Fredlund and Xing (1994) equation gives a SWCC with a small gradient in the low suction range and with the incorporation of the correction function, $C(u_a - u_w)$, the Fredlund and Xing (1994) equation is always directed to a soil suction of 1 GPa at zero water content. In this study, the Fredlund and Xing (1994) equation was chosen to develop the SWCC estimation method.

Published literatures have shown that the SWCC estimation methods can be developed using grain size distribution and volume-mass properties. Gupta and Larson (1979) used readily available data such as grain size distribution, organic matter and bulk density to estimate the SWCC. Zapata (1999) used P_{200} and PI , and D_{60} and e , for plastic (i.e., $PI > 0$) and non-plastic (i.e., $PI = 0$) soils respectively for SWCC estimation using Fredlund and Xing (1994) equation. Perera (2003) also used P_{200} and PI but with a different correlation to estimate the SWCC of plastic (i.e., $PI > 0$) soils. For non-plastic (i.e., $PI = 0$) soils, gradation parameters such as D_{10} , D_{20} , D_{30} , D_{90} and P_{200} were used. In this study, attempts were made to develop correlations between the parameters $[a, n, m, (u_a - u_w)_r]$ of Fredlund and Xing (1994) equation and the parameters derived from the grain size distribution data and soil basic index properties. In the development of the proposed method, disturbed (compacted or reconstituted) and undisturbed soils were not differentiated and the proposed method is assumed to be applicable to both disturbed and undisturbed soils.

A total of 60 soils were extracted from SoilVision database. Since the SoilVision database does not contain Atterberg limits, categorization of soils in term of plastic soils or non-plastic soils are not possible. Hence, in this study, the soils were divided into two major categories according to the grain size distribution data. The two categories are fine-grained soils which are defined as soils with $P_{200} \geq 30\%$ and coarse-grained soils which are defined as soils with $P_{200} < 30\%$. The definitions of

“fine-grained soils” and “coarse-grained soils” are different from the definitions in the USCS, Catana et al. (2006) and Vanapalli and Catana (2005). Juang and Holtz (1986) have shown that the permeability of sand-clay mixtures is affected severely with increase in clay content. With a clay content of 30%, the permeability of the sand-clay mixture is in the range of clay soils (i.e., $\leq 10^{-6}$ m/s), thus providing some basis for the categorization of fine-grained soils and coarse-grained soils adopted in the proposed method.

Briefly, the steps adopted to develop the proposed method are as follows:

- 1) A query for soil texture of clay or sand having both grain size distribution and SWCC was made in SoilVision database.
- 2) From the SoilVision database, soils with information on grain size distribution, volume-mass properties and SWCC were initially selected. From this selection only data containing at least five data points for grain size distribution and SWCC were selected.
- 3) From the above selection, 30 data sets each for fine-grained and coarse-grained soils were randomly sampled (each soil in the above selection has an equal chance of being sampled and that every possible combination of the specified number of soils has an equal chance of selection) to develop the correlations between the parameters of Fredlund and Xing equation and basic index properties. The 30 data sets composed the “calibration” data set. In this study, 30 data sets were used as they provide a large enough sample size for statistical analyses (Montgomery and Runger 2007) and keep the development work tractable.
- 4) In order to generate the gradation parameters (i.e., D_{10} , P_{200}), the grain size distribution was digitized using the SoilVision program. These parameters together with volume mass properties (i.e., bulk density, initial void ratio) and SWCC data were exported to Microsoft Excel for further analysis.
- 5) Using Fredlund and Xing equation, four fitting parameters: a , n , m and $(u_a - u_w)_r$ were generated using a minimization algorithm for the selected data sets. The quantity to minimize is the sum of the squared normalized residuals, SSNR defined as follows:

$$SSNR = \sum_{i=1}^N \left(\frac{\theta_i - \theta_{i(est)}}{\theta_i} \right)^2 \quad (5.22)$$

where $\theta_{i(est)}$ is the estimated value of θ_i and N is the number of data.

- 6) The fitting parameters and the corresponding basic index properties for each data set are tabulated in Tables 5.2 and 5.3 for fine-grained and coarse-grained soils, respectively.
- 7) Using regression analyses, correlation was made between the Fredlund and Xing SWCC parameters obtained in step (5) and basic index properties of soils such as e , D_{10} , D_{30} , D_{50} , D_{60} , coefficient of uniformity c_u , coefficient of curvature c_c and P_{200} . Other possible combined parameters such as eD_{50} , eD_{60} , eP_{200} and $e \frac{D_{60}}{D_{10}}$ were also attempted in the correlation. The coefficient of determination, R^2 , was used as an indicator of the most significantly correlated parameters. Table 5.4 presents the R^2 for the various combined parameters attempted. A sensitivity analysis could be performed for a more accurate correlation of the parameters but is not performed in this study.
- 8) Once the correlation has been identified, the fitting parameters were expressed as a function of soil basic index properties. To match the one-point SWCC measurement, at least one fitting parameter was expressed as a function of an adjustable variable, x , instead of soil basic index properties, so that the estimated SWCC can be adjusted to pass through the measured SWCC point.
- 9) To assess the suitability of the correlation, the parameters from the correlation were plotted with the experimental data points for all the calibration data sets.
- 10) Various correlations were attempted by trial-and-error procedure until the correlated fitting parameters plot showed a good agreement with the experimental data in the calibration data sets.

Table 5.2 Fitting parameters and basic index properties for fine-grained soils ($P_{200} \geq 30\%$) in calibration stage.
Soil condition in all cases was undisturbed.

No.	Soil counter	Fitting parameters				Void ratio, e	Grain size distribution			Reference
		a (kPa)	n	m	$(u_a - u_w)_r$ (kPa)		Sand (%)	P_{200}		
								Silt (%)	Clay (%)	
1	10760	72.90	0.25	0.57	1754	0.62	27.2	31.44	41.36	Quisenberry et al. (1987)
2	10765	68.01	0.01	0.45	68	0.73	45.27	11.21	43.52	Quisenberry et al. (1987)
3	10776	0.66	1.54	0.11	1001	0.75	34.6	12.7	52.7	Bruce et al. (1983)
4	10777	0.18	0.59	0.24	5221	0.81	38.6	10.98	50.42	Bruce et al. (1983)
5	10831	100.00	0.01	1.14	363	0.80	4.13	28.83	67.04	Plagge et al. (1990)
6	10838	14.13	1.35	0.15	517	0.81	11.18	39.15	49.67	Wendroth (1990)
7	10847	100.00	0.13	1.16	500	0.65	3.4	37.97	58.63	Wendroth (1990)
8	10851	6.31	0.76	0.17	561	0.86	1.63	27.59	70.78	Wendroth (1990)
9	10980	3.55	1.53	0.12	603	0.97	15.64	31.02	53.34	Schindler et al. (1985)
10	10981	6.30	0.83	0.14	585	1.26	1.7	27.96	70.34	Schindler et al. (1985)
11	10982	99.99	0.55	0.17	500	1.26	3.55	20.11	76.34	Schindler et al. (1985)
12	11306	0.98	0.55	0.34	599	1.18	8.23	31.8	59.97	Vereecken (1988)
13	11307	8.23	0.49	0.49	521	1.39	6.61	27.44	65.95	Vereecken (1988)
14	11409	12.07	0.91	0.25	516	1.25	9.18	35.51	55.31	Becher (1970)
15	11410	100.00	0.57	0.44	500	1.37	9.55	28.29	62.16	Becher (1970)
16	12428	1077.60	0.53	0.74	1264	0.85	5.97	23.7	70.33	Russam (1958)
17	12429	9677.57	0.33	1.53	9851	0.87	3.57	30.88	65.55	Russam (1958)
18	12430	99.91	12.88	0.04	500	0.57	10.46	36.9	52.64	Russam (1958)
19	12431	1017.72	0.66	0.86	1075	1.37	6.95	29.88	63.17	Russam (1958)
20	12432	4.65	0.54	0.99	470	2.49	5.97	23.7	70.33	Russam (1958)
21	12433	4.17	1.21	0.61	500	3.00	3.57	30.88	65.55	Russam (1958)
22	12434	3.29	1.67	0.47	500	1.75	10.46	36.9	52.64	Russam (1958)
23	12435	41.47	0.40	1.31	498	2.69	6.95	29.88	63.17	Russam (1958)
24	12445	1465.80	3.18	1.42	26611	1.07	2.69	7.25	90.06	Ridley (1993)
25	10839	11.02	1.41	0.19	518	0.78	10.71	39.57	49.72	Wendroth (1990)
26	10840	1.29	0.58	0.52	589	1.16	12.95	45.18	41.87	Wendroth (1990)
27	10843	18.14	1.55	0.13	503	0.82	9.12	47.52	43.36	Wendroth (1990)
28	11148	2.45	1.59	0.37	500	0.84	2.02	45.18	52.8	Shein (1991)
29	11217	11.62	1.34	0.42	505	0.91	1.24	53.31	45.45	Römkens et al. (1986)
30	11224	5.19	0.68	0.66	515	1.22	0	56.87	43.13	Römkens et al. (1985)

Table 5.3 Fitting parameters and basic index properties for coarse-grained soils ($P_{200} < 30\%$) in calibration stage.
In all cases, $(u_\sigma - u_w)_r = 100$ kPa and soil condition was undisturbed.

No.	Soil counter	Fitting parameters			Basic index properties						Grain-size distribution			Reference
		a (kPa)	n	m	void ratio, e	D_{10} (mm)	D_{30} (mm)	D_{50} (mm)	D_{60} (mm)	Sand (%)	P_{200}			
											Silt (%)	Clay (%)		
1	10709	0.67	1.19	1.40	0.58	0.1360	0.5757	0.7559	0.8459	92.83	4.03	3.14	Dane et al. (1983)	
2	10710	0.59	1.63	1.09	0.57	0.1220	0.5212	0.7050	0.7974	93.25	3.9	2.85	Dane et al. (1983)	
3	10715	2.15	3.36	0.91	0.75	0.0640	0.1411	0.1880	0.2121	85.83	10.75	3.42	Dane et al. (1983)	
4	10716	2.63	4.35	0.89	0.65	0.0710	0.1475	0.1924	0.2149	87.8	8.35	3.85	Dane et al. (1983)	
5	10717	2.84	4.01	1.07	0.64	0.0645	0.1421	0.1878	0.2107	86.14	9.72	4.14	Dane et al. (1983)	
6	10718	1.55	5.37	0.66	0.74	0.0923	0.3090	0.4494	0.5356	90.47	5.02	4.51	Dane et al. (1983)	
7	10720	0.91	6.85	0.63	0.71	0.0807	0.3641	0.5551	0.6631	89.93	2.83	7.24	Dane et al. (1983)	
8	10721	0.91	7.73	0.77	0.72	0.2367	0.4368	0.6288	0.7250	94.7	1.26	4.04	Dane et al. (1983)	
9	10722	0.94	10.53	0.79	0.72	0.2869	0.4979	0.6777	0.7675	97.02	0.39	2.59	Dane et al. (1983)	
10	10723	1.35	3.09	0.83	0.71	0.1036	0.2281	0.3540	0.4188	91.14	5.44	3.42	Dane et al. (1983)	
11	10724	1.66	8.15	0.67	0.66	0.0697	0.2264	0.3593	0.4276	89.32	5.26	5.42	Dane et al. (1983)	
12	10726	1.62	8.84	0.70	0.70	0.1262	0.2458	0.3919	0.4654	91.76	3.39	4.85	Dane et al. (1983)	
13	10727	1.25	3.36	0.76	0.91	0.0809	0.2159	0.3964	0.5018	89.79	6.59	3.62	Dane et al. (1983)	
14	10728	0.79	2.05	0.96	0.64	0.0865	0.2532	0.4678	0.6129	90.16	4.14	5.7	Dane et al. (1983)	
15	10729	0.74	2.21	0.99	0.66	0.1051	0.2676	0.5260	0.6997	91.00	3.07	5.93	Dane et al. (1983)	
16	10730	0.87	3.03	1.09	0.65	0.1670	0.3964	0.6858	0.8406	95.51	1.32	3.17	Dane et al. (1983)	
17	10731	0.81	2.90	1.19	0.70	0.1824	0.4159	0.6832	0.8253	96.33	1.11	2.56	Dane et al. (1983)	
18	10732	0.77	3.78	0.80	0.68	0.1549	0.3713	0.5934	0.7116	93.48	1.05	5.47	Dane et al. (1983)	
19	10749	1.86	2.46	0.64	0.57	0.0427	0.1608	0.2259	0.2714	85.13	10.78	4.09	Quisenberry et al. (1987)	
20	10767	2.75	6.35	0.64	0.70	0.0642	0.1612	0.2246	0.2680	87.31	8.98	3.71	Quisenberry et al. (1987)	
21	10768	1.93	1.78	0.94	0.56	0.0500	0.1541	0.2180	0.2500	85.18	10.54	4.28	Quisenberry et al. (1987)	
22	10857	3.41	7.14	0.74	0.57	0.1013	0.1522	0.2101	0.2946	95.60	1.8	2.6	Neumann et al. (1985)	
23	11166	1.87	1.69	0.90	0.70	0.0600	0.1427	0.2054	0.2368	86.92	8.47	4.61	Schuh et al. (1991)	
24	11167	2.03	3.38	0.70	0.70	0.0629	0.1404	0.2006	0.2307	87.09	9.29	3.62	Schuh et al. (1991)	
25	11168	1.66	3.25	0.86	0.78	0.0718	0.1454	0.2037	0.2328	88.84	7.04	4.12	Schuh et al. (1991)	
26	11169	2.04	4.19	0.70	0.69	0.0864	0.1533	0.2119	0.2412	90.94	6.89	2.17	Schuh et al. (1991)	
27	11175	2.13	5.36	0.67	0.73	0.0651	0.1516	0.2209	0.2605	87.93	7.6	4.47	Schuh et al. (1991)	
28	11186	3.81	2.87	0.50	0.65	0.0572	0.1411	0.2242	0.2753	84.8	13.57	1.63	Schuh et al. (1991)	
29	11188	2.82	10.56	0.44	0.71	0.0885	0.1479	0.1974	0.2221	90.48	5.84	3.68	Schuh et al. (1991)	
30	112	2.79	12.98	0.61	0.81	0.0953	0.1610	0.2240	0.2554	93.03	6.97	0	Mualern and Klute (1984)	

Table 5.4 Coefficient of determination, R^2 for various correlations between basic properties and fitting parameters

Parameter	R^2 for fine-grained soils				R^2 for coarse-grained soils*		
	a	n	m	$(u_a-u_w)_r$	a	n	m
a	1	0.0349	0.3587	0.5034	1	0.1816	0.3418
n	0.0349	1	0.2606	0.0956	0.1816	1	0.4409
m	0.3587	0.2606	1	0.1788	0.3418	0.4409	1
$(u_a-u_w)_r$	0.5034	0.0956	0.1788	1	-	-	-
D_{10}	-	-	-	-	0.3982	0.1109	0.1703
D_{30}	-	-	-	-	0.7771	0.0221	0.3383
D_{50}	-	-	-	-	0.8306	0.0405	0.4500
D_{60}	-	-	-	-	0.8300	0.0610	0.4994
P_{200}	0.0776	0.0947	0.1320	0.0058	0.3075	0.1484	0.0885
$\frac{D_{30}^2}{D_{60} \times D_{10}}$	-	-	-	-	0.1350	0.1780	0.1854
$\frac{D_{60}}{D_{10}}$	-	-	-	-	0.3126	0.4425	0.1166
e	0.0221	0.0694	0.1328	0.0170	0.0064	0.1910	0.1433
eD_{50}	-	-	-	-	0.7871	0.1862	0.2351
eD_{60}	-	-	-	-	0.7796	0.1408	0.2522
eP_{200}	0.0064	0.0279	0.1440	0.0128	0.3139	0.0993	0.1523
$e \frac{D_{60}}{D_{10}}$	-	-	-	-	0.3057	0.3174	0.1186

* $(u_a-u_w)_r$ is fixed at 100 kPa

5.3.3 Fine-Grained Soils ($P_{200} \geq 30\%$)

In general, there are two rules to be followed in the attempt to develop the proposed method. The fitting parameters a , n , m and $(u_a-u_w)_r$ must not be negative and $(u_a-u_w)_r$ must be larger than a as described by Leong and Rahardjo (1997), so that the definition of $(u_a-u_w)_r$ is not violated. As the fine-grained soils have been defined as soils with $P_{200} \geq 30\%$ which cover soils ranging from silty to clayey soils, the range of parameter a also becomes broader. Furthermore, the new SWCC estimation method shall also allow the estimated SWCC could pass through the measured SWCC point. From Table 5.4, it can be seen that parameter m and eP_{200} has the highest R^2 value among the correlation of the fitting parameters and the basic soil properties. For correlation between the fitting parameters, parameters a and n show strong correlation with m while $(u_a-u_w)_r$ is strongly correlated with a . By examining the various relationships [m vs eP_{200} , a vs n , n vs m and $(u_a-u_w)_r$ vs a], it was found that all the four fitting parameters can be correlated to a single parameter eP_{200} . To allow the SWCC to pass through or come close to the measured SWCC point, eP_{200}

is replaced by the adjustable variable, x . These correlations in the proposed SWCC estimation method for fine-grained soils are given below:

$$a = -2.4(x) + 722 \quad kPa \quad (5.23)$$

$$n = 0.07(x)^{0.4} \quad (5.24)$$

$$m = 0.015(x)^{0.7} \quad (5.25)$$

$$(u_a - u_w)_r = 914 \exp[-0.002(x)] \quad kPa \quad (5.26)$$

From the calibration data set, the range of values for variable x is from 0 to 300.8 and the corresponding ranges for a , n , m and $(u_a - u_w)_r$ are from 0 to 722 kPa, 0 to 0.68, 0 to 0.81 and 500 to 914 kPa, respectively, as shown in Figure 5.5. The proposed family of drying SWCC is shown in Figure 5.6.

Figure 5.5 The working range of fitting parameters a , n , m , $(u_a - u_w)_r$ corresponding to the working range of variable x for fine-grained soils ($P_{200} \geq 30\%$).

Figure 5.6 Family of drying SWCC for fine-grained soils ($P_{200} \geq 30\%$).

5.3.4 Coarse-Grained Soils ($P_{200} < 30\%$)

From Table 5.4, it can be seen that the highest coefficient of determination, R^2 , for fitting parameters and basic soil properties is for the correlation between parameter a and D_{50} ($R^2 = 0.8306$). The relationship between parameter a and D_{50} is shown in Figure 5.7. Table 5.4 also shows that parameter n has the highest R^2 with $\frac{D_{60}}{D_{10}}$ ($R^2 = 0.4425$) and parameter m has the highest R^2 with D_{60} ($R^2 = 0.4994$). This suggests that parameters n and m are closely related as confirmed by the correlation among the fitting parameters. The relationship between n and m is presented in Figure 5.8. As parameter n increases, parameter m decreases. This trend agrees with the findings by Smettem and Gregory (1996). To reduce the number of soil parameters used in the proposed method, parameters n and m were correlated to a single parameter D_{60} . The fitting parameter $(u_a - u_w)_r$ shows very poor correlation with the other fitting parameters and basic soil properties. Fixing the value of $(u_a - u_w)_r$ at 100 kPa as recommended by Houston et al. (2006) provides good SWCC estimation for the calibration data set. To allow the SWCC to pass through or come close to the measured SWCC point, D_{60} is replaced with the adjustable variable, x . The correlations are as follow:

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$$a = 0.53(D_{50})^{-0.96} \quad kPa \tag{5.27}$$

$$n = x \tag{5.28}$$

$$m = -0.23 \ln(x) + 1.13 \tag{5.29}$$

$$(u_a - u_w)_r = 100 \quad kPa \tag{5.30}$$

Figure 5.7 Correlation of a and D_{50} for coarse-grained soils ($P_{200} < 30\%$).

Figure 5.8 Relationship between m and n for coarse-grained soils ($P_{200} < 30\%$).

5.3.5 Evaluation of the Proposed Method

The validity of the SWCC estimation for both fine-grained and coarse-grained soils using the proposed equation was examined using independent data sets from published literatures. A total of 62 independent data sets with 31 soils each for fine-grained and coarse-grained soils as tabulated in Tables 5.5 and 5.6, respectively, were used for evaluation. This data set is deemed as the “evaluation” data set.

After the SWCC measurement point has been selected, Equations 5.23 – 5.26 or 5.27 – 5.30 were employed, so that the fitted curve passes through or comes close to the selected SWCC point. Curve fitting was performed by using the solver routine provided in Microsoft Excel 2003. The variable x in Equations 5.23 – 5.26 or 5.27 – 5.30 was adjusted in the solver routine until the estimated SWCC passes through the measured point. In a few cases, the estimated SWCC could not pass through the measured point and therefore the final estimated SWCC curve was the one which gives the least SSNR.

Selection of the one point SWCC measurement (i.e., volumetric water content with the associated matric suction value) is important in the SWCC estimation method. Fredlund (2006) suggested that there are three zones describing the SWCC which are boundary effect, transition and residual zones (Figure 5.1). Sensitivity analyses have been performed to examine the location of the one point SWCC measurement which would give the most reliable SWCC for the proposed method. For fine-grained soils, analyses were performed at different matric suction values of 100, 500, and 1000 kPa which covered all the three zones as described by Fredlund (2006). For coarse-grained soils, sensitivity analyses were performed at matric suction values of 5, 10 and 50 kPa. The estimation errors at each matric suction were quantified by sum of squares errors, SSE which is given as follows:

$$SSE = \sum_{i=1}^N (\theta_i - \theta_{i(est)})^2 \quad (5.31)$$

Table 5.5 Properties of fine-grained soils ($P_{200} \geq 30\%$) for evaluation

Reference	Code	Volume-mass properties		Liquid limit	Plastic limit	Plasticity index	Grain-size distribution				USCS	Soil condition
		Bulk density, ρ_b (Mg/m ³)	Void ratio, e				Gravel (%)	Sand (%)	P_{200}			
									Silt (%)	Clay (%)		
Tinjum et al. (1997) ^a	B _{sp} (dry)	1.999	0.645	49	23	26	0	6	54	40	CL	Compacted
	B _{sp} (opt)	2.272	0.538	49	23	26	0	6	54	40	CL	Compacted
	B _{sp} (wet)	2.270	0.616	49	23	26	0	6	54	40	CL	Compacted
	B _{mp} (dry)	2.274	0.416	49	23	26	0	6	54	40	CL	Compacted
	B _{mp} (opt)	2.466	0.379	49	23	26	0	6	54	40	CL	Compacted
	B _{mp} (wet)	2.404	0.464	49	23	26	0	6	54	40	CL	Compacted
	C _{sp} (dry)	2.073	0.550	27	12	15	0	24	48	28	CL	Compacted
	C _{sp} (opt)	2.439	0.427	27	12	15	0	24	48	28	CL	Compacted
	C _{sp} (wet)	2.368	0.550	27	12	15	0	24	48	28	CL	Compacted
	C _{mp} (dry)	2.190	0.377	27	12	15	0	24	48	28	CL	Compacted
	C _{mp} (opt)	2.532	0.309	27	12	15	0	24	48	28	CL	Compacted
	C _{mp} (wet)	2.516	0.370	27	12	15	0	24	48	28	CL	Compacted
	F _{sp} (dry)	1.934	0.783	67	21	46	0	6	41	53	CH	Compacted
	F _{sp} (opt)	2.238	0.686	67	21	46	0	6	41	53	CH	Compacted
	F _{sp} (wet)	2.239	0.739	67	21	46	0	6	41	53	CH	Compacted
	F _{mp} (dry)	2.194	0.527	67	21	46	0	6	41	53	CH	Compacted
	F _{mp} (opt)	2.465	0.462	67	21	46	0	6	41	53	CH	Compacted
	F _{mp} (wet)	2.455	0.527	67	21	46	0	6	41	53	CH	Compacted
	M _{sp} (dry)	2.087	0.608	32	18	14	1	14	41	44	CL	Compacted
	Samingan et al. (2003)	M _{sp} (opt)	2.341	0.508	32	18	14	1	14	41	44	CL
M _{sp} (wet)		2.353	0.608	32	18	14	1	14	41	44	CL	Compacted
M _{mp} (dry)		2.422	0.387	32	18	14	1	14	41	44	CL	Compacted
M _{mp} (opt)		2.532	0.366	32	18	14	1	14	41	44	CL	Compacted
M _{mp} (wet)		2.536	0.408	32	18	14	1	14	41	44	CL	Compacted
UP1		2.029	0.800	36	26	10	0	50	35	15	ML	Undisturbed
UP2		1.937	1.050	54	23	31	0	43	45	12	MH	Undisturbed
UP3		2.345	0.610	34	21	13	0	16	58	26	CL	Undisturbed
UP4		1.937	0.670	48	37	11	0	66	10	24	SC	Undisturbed
DOP		2.110	0.580	36	17	19	0	28	42	30	CL	Compacted
Vanapalli (1994)	OP	2.093	0.520	36	17	19	0	28	42	30	CL	Compacted
	WOP	1.955	0.545	36	17	19	0	28	42	30	CL	Compacted

Note: CH, high-plasticity clay; CL, low-plasticity clay; MH, high-plasticity silt; ML, low-plasticity silt; SC, clayey sand.
^a Specific gravity, G_s , is assumed as 2.7.

Table 5.6 Properties of coarse-grained soils ($P_{200} < 30\%$) for evaluation

Reference	Code	Void ratio, e	Grain-size distribution				
			D_{10} (mm)	D_{30} (mm)	D_{50} (mm)	D_{60} (mm)	P_{200} (%)
Smettem and Gregory (1996)	EB10	0.563	0.0420	0.1649	0.3122	0.4105	16.28
	EB20	0.563	0.0424	0.1639	0.3168	0.4210	16.28
	D10	0.639	0.1024	0.3233	0.4224	0.4689	9.02
	D20	0.493	0.1139	0.3032	0.4006	0.4491	8.28
	D30	0.493	0.0906	0.2884	0.3840	0.4358	9.46
	D50	0.471	0.0682	0.2746	0.3660	0.4209	11.00
	D90	0.515	0.0558	0.2678	0.3546	0.4102	11.90
	WH10	0.613	0.0599	0.2128	0.3082	0.3524	13.40
	WH20	0.515	0.0001	0.2190	0.3341	0.3920	17.61
	WH30	0.538	0.0001	0.1874	0.3178	0.3768	20.99
	WH50	0.515	0.0001	0.1748	0.3139	0.3749	22.15
Jauhainen (2004)	11CTA	1.101	0.0508	0.2774	0.4221	0.4944	12.62
	11CTB1	0.946	0.0514	0.2644	0.4142	0.4891	13.25
	11CTB2	0.786	0.0502	0.2680	0.4300	0.5109	13.37
	11CTC	0.661	0.1831	0.2982	0.4021	0.4540	3.69
	21VTB1	1.105	0.0001	0.0886	0.1471	0.1763	29.08
	21VTB2	0.779	0.0118	0.0906	0.1413	0.1666	28.15
	21VTC	0.650	0.0548	0.1186	0.1785	0.2265	19.07
	22VTC	0.730	0.0170	0.1001	0.1587	0.1880	25.16
	111MTA	1.242	0.0697	0.2694	0.4187	0.4933	11.33
	111MTB1	0.976	0.0754	0.2704	0.4185	0.4926	10.89
	111MTB2	0.733	0.0993	0.2822	0.4276	0.5004	8.84
	111MTC	0.698	0.1140	0.2856	0.4260	0.4961	7.45
	121MTA	1.415	0.0287	0.0999	0.1752	0.2318	26.29
	121MTB1	1.033	0.0392	0.1131	0.1907	0.2644	22.99
	211CTA	1.564	0.0627	0.1539	0.2763	0.3534	15.09
	211CTB1	0.866	0.0768	0.1533	0.2622	0.3418	12.38
	211CTB2	0.764	0.0818	0.1596	0.2743	0.3516	11.07
	211CTC	0.718	0.0876	0.1209	0.2488	0.3276	9.5
	231MTA	1.632	0.0124	0.0883	0.2603	0.3915	29.68
Yang et al. (2004)	GS	0.617	2.7300	3.6800	4.7300	5.1500	0

From the analysis, it was found that the one-point measurement at matric suctions 100 kPa and 500 kPa gave equally good estimation SWCC for fine-grained soils. However, it is recommended that matric suction of 500 kPa to be used in the proposed method. This finding also agreed with Catana et al. (2006) suggestion that the one-point suction measurement should be in the range of 50 to 500 kPa. For coarse-grained soils, it is recommended that matric suction of 10 kPa which gives the least total SSE to be used in the proposed method. This finding agreed with Vanapalli and Catana (2005) suggestion that the one-point suction measurement should be in the range of 0.1 to 10 kPa. It is possible that coarse-grained soils fully desaturate at matric suction of 10 kPa. However, the proposed method is still applicable as illustrated in Figure 5.9 for a coarse-grained soil that fully desaturated

at a matric suction of 10 kPa,. Table 5.7 shows the total SSE for each of the matric suction examined for both fine-grained and coarse-grained soils. In Table 5.7, the evaluation was performed at the one-point SWCC measurement at 500 and 10 kPa for fine-grained and coarse-grained soils, respectively.

Figure 5.9 SWCC estimation for coarse-grained soil-code GS using one-point measurement at matric suction 10 kPa.

Table 5.7 Total SSE of SWCC estimation at various one-point measurements

Soil group	Matric suction (kPa)					
	5	10	50	100	500	1000
Fine-grained soils	-	-	-	0.146	0.142	0.236
Coarse-grained soils	0.472	0.262	0.322	-	-	-

The mean of the squared errors, MSE, root mean squared error, RMSE and coefficient of determination, R^2 as shown in Equations 5.32, 5.33 and 5.34, respectively, are among the most common criteria used to evaluate the reliability of the SWCC estimation (Wösten et al. 2001; Schaap et al. 2001; and Nemes et al. 2006). For all the criteria, the objective is to minimize the estimation errors for the

experimental data sets at the population level, which is quantified by MSE and RMSE. MSE reports the systematic errors between the measurements and estimated. RMSE provides the accuracy of the estimation in terms of standard deviation. The correlation between the measured and estimated SWCC is evaluated by R^2 . Therefore, in this study, error analysis was performed to evaluate the proposed method based on MSE, RMSE and R^2 . In addition, error analysis was also performed on Vanapalli and Catana (2005), Catana et al. (2006) and Houston et al. (2006) one-point methods for comparison purposes.

$$\text{MSE} = \frac{1}{N} \sum_{i=1}^N (\theta_i - \theta_{i(est)})^2 \quad (5.32)$$

$$\text{RMSE} = \sqrt{\frac{1}{N} \sum_{i=1}^N (\theta_i - \theta_{i(est)})^2} \quad (5.33)$$

$$R^2 = 1 - \frac{\text{SSE}}{\text{SST}} \quad (5.34)$$

where

$$\text{SST} = \sum_{i=1}^N (\theta_i - \bar{\theta})^2 \quad (5.35)$$

where $\bar{\theta}$ is the average value of θ and N is the number of data.

The results for the SWCC estimation using the proposed method for fine-grained and coarse-grained soils are presented in Tables 5.8 and 5.9, respectively. From Table 5.8, it can be seen that the adjustable variable x varies from a value of 0.1 to 298 for fine-grained soils. For coarse-grained soils, the adjustable variable x ranges from a value of 0.85 to 10.3 as shown in Table 5.9. Figures 5.10 and 5.11 show the variability of the estimated volumetric water content versus measured volumetric water content for fine-grained and coarse-grained soils, respectively.

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Table 5.8 Parameters describing the SWCC of fine-grained ($P_{200} \geq 30\%$) soils using the proposed method

Code	Adjustable variable x	Parameter			
		a (kPa)	n	m	$(u_a - u_w)_r$ (kPa)
B _{sp} (dry)	259.85	98.4	0.65	0.74	544
B _{sp} (opt)	152.11	356.9	0.52	0.51	674
B _{sp} (wet)	147.79	367.3	0.52	0.50	680
B _{mp} (dry)	239.41	147.4	0.63	0.69	566
B _{mp} (opt)	249.73	122.7	0.64	0.72	555
B _{mp} (wet)	82.16	524.8	0.41	0.33	775
C _{sp} (dry)	235.76	156.2	0.62	0.69	570
C _{sp} (opt)	48.84	604.8	0.33	0.23	829
C _{sp} (wet)	39.50	627.2	0.30	0.20	845
C _{mp} (dry)	228.68	173.2	0.61	0.67	579
C _{mp} (opt)	244.59	135.0	0.63	0.70	560
C _{mp} (wet)	18.62	677.3	0.23	0.12	881
F _{sp} (dry)	252.08	117.0	0.64	0.72	552
F _{sp} (opt)	123.09	426.6	0.48	0.44	715
F _{sp} (wet)	69.54	555.1	0.38	0.29	795
F _{mp} (dry)	278.25	54.2	0.67	0.77	524
F _{mp} (opt)	217.10	201.0	0.60	0.65	592
F _{mp} (wet)	0.10	721.8	0.03	0.00	914
M _{sp} (dry)	224.32	183.6	0.61	0.66	584
M _{sp} (opt)	237.45	152.1	0.62	0.69	568
M _{sp} (wet)	104.61	470.9	0.45	0.39	741
M _{mp} (dry)	271.25	71.0	0.66	0.76	531
M _{mp} (opt)	174.82	302.4	0.55	0.56	644
M _{mp} (wet)	84.65	518.8	0.41	0.34	772
UP1	280.00	50.0	0.67	0.77	522
UP2	298.00	6.8	0.68	0.81	504
UP3	249.96	122.1	0.64	0.72	554
UP4	297.96	6.9	0.68	0.81	504
DOP	293.00	18.8	0.68	0.80	509
OP	273.55	65.5	0.66	0.76	529
WOP	262.37	92.3	0.65	0.74	541

Table 5.9 Parameters describing the SWCC of coarse-grained soils ($P_{200} < 30\%$) using the proposed method. $(u_a - u_w)_r = 100$ kPa for all cases.

Code	Adjustable variable x	Parameter		
		a (kPa)	n	m
EB10	1.42	1.62	1.42	1.05
EB20	2.06	1.60	2.06	0.96
D10	1.78	1.21	1.78	1.00
D20	2.44	1.28	2.44	0.93
D30	2.47	1.33	2.47	0.92
D50	2.09	1.39	2.09	0.96
D90	1.84	1.43	1.84	0.99
WH10	1.77	1.64	1.77	1.00
WH20	1.25	1.52	1.25	1.08
WH30	1.08	1.59	1.08	1.11
WH50	0.85	1.61	0.85	1.17
11CTA	1.34	1.21	1.34	1.06
11CTB1	0.95	1.24	0.95	1.14
11CTB2	1.76	1.19	1.76	1.00
11CTC	8.12	1.27	8.12	0.65
21VTB1	2.21	3.34	2.21	0.95
21VTB2	1.78	3.47	1.78	1.00
21VTC	10.30	2.77	10.30	0.59
22VTC	2.98	3.10	2.98	0.88
111MTA	1.04	1.22	1.04	1.12
111MTB1	1.29	1.22	1.29	1.07
111MTB2	1.34	1.20	1.34	1.06
111MTC	8.01	1.20	8.01	0.65
121MTA	1.55	2.82	1.55	1.03
121MTB1	1.05	2.60	1.05	1.12
211CTA	1.49	1.82	1.49	1.04
211CTB1	1.56	1.92	1.56	1.03
211CTB2	2.23	1.83	2.23	0.95
211CTC	9.22	2.01	9.22	0.62
231MTA	0.85	1.93	0.85	1.17
GS	5.54	0.12	5.54	0.74

Figure 5.10 Estimated versus measured θ_w for fine-grained soils ($P_{200} \geq 30\%$) in this study ($R^2=0.929$).

Figure 5.11 Estimated versus measured θ_w for coarse-grained ($P_{200} < 30\%$) soils in this study ($R^2=0.975$).

5.3.6 Comparison with Other Methods

Table 5.10 summarizes the comparison of the proposed method with other existing one-point SWCC estimation methods. It can be seen that for fine-grained soils, the proposed method only uses one independent variable, compared to two independent variables used by Catana et al. (2006) and Houston et al. (2006). For coarse-grained soils, the proposed method only uses two independent variables, compared to five and six independent variables used by Vanapalli and Catana (2005) and Houston et al. (2006), respectively.

The performance of the proposed method was compared with other one-point methods. Different researchers have categorized soils differently. Houston et al. (2006) proposed a one-point method for two groups, namely non-plastic soils and plastic soils. Vanapalli and Catana (2005) and Catana et al. (2006) suggested the one-point method for coarse-grained and compacted fine-grained soils, respectively. To compare the various one-point methods, only soils which satisfy the definition for all the three researchers' criteria were used. For Catana et al. (2006) method, the SWCC was estimated using one-point estimation of the SWCC at the intersection of a matric suction of 2000 kPa and the desorption line of gradient C which intersects the matric suction axis of 400 MPa.

Table 5.11 presents the comparisons between the proposed method and other one-point methods in term of SSE for each of the soils estimated. From Table 5.11, it can be seen that the variations of SSE are 0 to 0.0579 (proposed method), 0 to 0.2614 (Catana et al. 2006) and 0 to 0.0775 (Houston et al. 2006), for fine-grained soils, and 0.0009 to 0.0315 (proposed method), 0.0025 to 0.2834 (Vanapalli and Catana 2005) and 0.0004 to 0.1380 (Houston et al. 2006), for coarse-grained soils. It is highlighted that Catana et al. (2006) method was developed specifically for compacted fine-grained soils. The data set (Table 5.5) used for evaluation consists of mostly compacted soils and therefore Catana et al. (2006) method is not prejudiced in the evaluation. The comparisons are illustrated in Figures 5.12, 5.13 and 5.14, for fine-grained soils and Figures 5.15, 5.16 and 5.17, for coarse-grained soils as stated in Table 5.11. These figures represent the best (Figures 5.12 and

5.15), intermediate (Figures 5.13 and 5.16), and worst (Figures 5.14 and 5.17) estimation cases, using the proposed method for the evaluation data set. The overall performance of the proposed method with the other one-point methods in term of MSE, RMSE and R^2 is summarized in Table 5.12. It can be seen that the proposed method has the lowest MSE and RMSE and the highest R^2 values compared to the other existing one-point methods. This suggests that the proposed method performed better than Vanapalli and Catana (2005), Catana et al. (2006), and Houston et al. (2006) methods.

Table 5.10 Comparison of proposed method with other one-point methods

Description	Fine-grained soils			Coarse-grained soils		
	Proposed Method	Catana et al. (2006)	Houston et al. (2006)	Proposed Method	Vanapalli and Catana (2005)	Houston et al. (2006)
Definition	$P_{200} \geq 30\%$	USCS	$PI > 0$	$P_{200} < 30\%$	USCS	$PI = 0$
SWCC equation	Fredlund and Xing (1994)	Brutsaert (1966)	Fredlund and Xing (1994)	Fredlund and Xing (1994)	Fredlund and Xing (1994)	Fredlund and Xing (1994)
Number of independent variables	1 (x)	2 (LL, C_f)	2 (PI, P_{200})	2 (D_{50}, x)	5 ($d_e, D_{10}, D_{60}, e, x$)	6 ($D_{10}, D_{20}, D_{30}, D_{60}, D_{90}, P_{200}$)
Number of fitting parameters	4	2	4	4	3	4
One-point SWCC measurement at matric suction (kPa)	500	50 - 500	Not reported	10	0.1 - 10	Not reported

Table 5.11 Comparison of proposed method with other one-point methods in term of SSE

Fine-grained soils				Coarse-grained soils			
Code	Proposed Method	Catana et al. (2006)	Houston et al. (2006)	Code	Proposed Method	Vanapalli and Catana (2005)	Houston et al. (2006)
B _{sp} (dry)	0.0052	0.0315	0.0104	EB10	0.0087	0.0044	0.0067
B _{sp} (opt)	0.0002	0.0018	0.0003	EB20	0.0033	0.0133	0.0349
B _{sp} (wet)	0.0003	0.0007	0.0001	D10	0.0081	0.0276	0.0004
B _{mp} (dry)	0.0002	0.0149	0.0005	D20	0.0028	0.0491	0.0008
B _{mp} (opt)	0.0002	0.0167	0.0001	D30	0.0031	0.0395	0.0006
B _{mp} (wet)	0.0001	0.0005	0.0003	D50	0.0037	0.0283	0.0014
C _{sp} (dry)	0.0005	0.0052	0.0012	D90	0.0040	0.0255	0.0049
C _{sp} (opt)	0.0001	0.0000	0.0001	WH10	0.0064	0.0200	0.0090
C _{sp} (wet)	0.0002	0.0008	0.0001	WH20	0.0084	0.0285	0.0106
C _{mp} (dry)	0.0003	0.0165	0.0006	WH30	0.0097	0.0228	0.0150
C _{mp} (opt)	0.0001	0.0098	0.0003	WH50	0.0153	0.0194	0.0089
C _{mp} (wet)	0.0001	0.0000	0.0001	11CTA	0.0012	0.0200	0.0377
F _{sp} (dry)	0.0005	0.0006	0.0003	11CTB1	0.0009	0.0065	0.0462
F _{sp} (opt)	0.0002	0.0018	0.0004	11CTB2	0.0087	0.0270	0.0022
F _{sp} (wet)	0.0006	0.0001	0.0001	11CTC	0.0135	0.0763	0.0158
F _{mp} (dry)	0.0007	0.0684	0.0006	21VTB1	0.0016	0.0158	0.0260
F _{mp} (opt)	0.0006	0.0128	0.0002	21VTB2	0.0054	0.0645	0.0078
F _{mp} (wet)	0.0000	0.0006	0.0000	21VTC	0.0056	0.0038	0.0366
M _{sp} (dry)	0.0034	0.0030	0.0052	22VTC	0.0082	0.0746	0.0152
M _{sp} (opt)	0.0024	0.0061	0.0042	111MTA	0.0094	0.0143	0.1079
M _{sp} (wet)	0.0004	0.0014	0.0008	111MTB1	0.0046	0.0161	0.0697
M _{mp} (dry)	0.0002	0.0325	0.0012	111MTB2	0.0030	0.0425	0.0507
M _{mp} (opt)	0.0001	0.0046	0.0001	111MTC	0.0228	0.0653	0.0156
M _{mp} (wet)	0.0001	0.0006	0.0001	121MTA	0.0048	0.0937	0.0110
UP1	0.0116	0.0126	0.0126	121MTB1	0.0315	0.0322	0.0024
UP2	0.0579	0.2614	0.0096	211CTA	0.0022	0.0312	0.0786
UP3	0.0255	0.0818	0.0298	211CTB1	0.0102	0.0124	0.0188
UP4	0.0038	0.1068	0.0775	211CTB2	0.0089	0.0037	0.0068
DOP	0.0173	0.0845	0.0075	211CTC	0.0198	0.0025	0.0027
OP	0.0029	0.0570	0.0004	231MTA	0.0149	0.2834	0.0802
WOP	0.0061	0.0425	0.0038	GS	0.0115	0.0699	0.1380

Table 5.12 Comparison of overall performance of proposed method with other one-point methods

Error Criteria	Fine-Grained Soils			Coarse-Grained Soils		
	Proposed Method	Catana et al. (2006)	Houston et al. (2006)	Proposed Method	Vanapalli and Catana (2005)	Houston et al. (2006)
MSE	0.0004	0.0027	0.0005	0.0007	0.0031	0.0023
RMSE	0.020	0.052	0.022	0.026	0.056	0.048
R ²	0.930	0.503	0.928	0.975	0.883	0.929

Note: A value of 0 indicates no error for MSE and RMSE; R² of 1 indicates a perfect fit.

Figure 5.12 SWCC estimation for fine-grained soil-code $F_{mp(wet)}$.

Figure 5.13 SWCC estimation for fine-grained soil-code WOP.

Figure 5.14 SWCC estimation for fine-grained soil-code UP2.

Figure 5.15 SWCC estimation for coarse-grained soil-code 11CTB1.

Figure 5.16 SWCC estimation for coarse-grained soil-code 231MTA.

Figure 5.17 SWCC estimation for coarse-grained soil-code 121MTB1.

5.4 Soil-Water Characteristic Curve of Soils used for Cyclic Simple Shear Test

The soils used for cyclic simple shear tests were AMK, JM, and NTU residual soils. The AMK, JM, and NTU residual soils contained 63%, 85%, and 54% fine contents, respectively. As such, they were categorized as fine-grained soils in the proposed method. The volumetric water contents equilibrated at matric suctions 100, 200, 400 kPa in pressure plate apparatus during unsaturated specimen preparation in Chapter 4 were used for SWCC estimation. Table 5.13 presents the volumetric water contents equilibrated at the different matric suctions in pressure plate apparatus. The one-point SWCC measurement at matric suction of 100 kPa was used.

Table 5.13 Volumetric water content of soil specimens used at different matric suctions

Residual soils	Specimen no.	Matric suction, $(u_a - u_w)$ (kPa)	Volumetric water content, θ_w
Undisturbed AMK	AMK50-100	100	0.457
	AMK50-200	200	0.414
	AMK50-400	400	0.354
Undisturbed JM	JM50-400	100	0.231
	JM50-400	200	0.222
	JM50-400	400	0.208
Reconstituted NTU	NTU100-100	100	0.310
	NTU100-200	200	0.282
	NTU100-400	400	0.268

The value of x in Equations 5.23 to 5.26 was adjusted, so that the fitted curve passed through the SWCC point at matric suction of 100 kPa. Table 5.14 summarizes the variable x and parameters obtained using the proposed method. From Table 5.14, it can be seen that x is 191.5 for NTU, 134.8 for JM, and 221.2 for AML residual soils.

The soil-water characteristic curves estimated from the proposed method are shown in Figures 5.18 for NTU, 5.19 for JM, and 5.20 for AMK residual soils. From Figures 5.18 to 5.20, it can be seen that the estimated SWCCs are very close to the experimental data. Also shown in Figure 5.18 to 5.20 are the volumetric water contents of the tested soil specimens under a net confining stress of 50 kPa for JM

and AMK soils and 100 kPa for NTU soil in the cyclic simples shear apparatus. The differences in volumetric water content for these specimens and those equilibrated in pressure plate apparatus with zero net confining stress are negligible. The air-entry and residual values were identified in the three soils, and thus three distinct zones were distinguished. Table 5.14 summarizes the air-entry and residual values for the soils used. The air-entry value was 80 kPa for NTU, 150 kPa for JM, and 45 kPa for AMK residual soils while the residual values was 35 MPa for NTU, 65 MPa for JM, and 35 MPa for AMK residual soils. The NTU and AMK residual soil specimens prepared for cyclic simple shear tests were from transition zone, while the JM residual soil specimens were from boundary effect and transition zones.

Table 5.14 Parameters describing the SWCC of soils used from the proposed
SWCC estimation method

Soils	Adjustable variable x	Parameter				AEV (kPa)	Residual value (kPa)
		a (kPa)	n	m	$(u_a - u_w)_r$ (kPa)		
NTU	191.5	262.3	0.57	0.59	623.1	80	35000
JM	134.8	398.5	0.50	0.46	698.0	150	65000
AMK	221.2	191.1	0.61	0.66	587.2	45	35000

Figure 5.18 Soil-water characteristic curve of NTU soil.

Figure 5.19 Soil-water characteristic curve of JM soil.

Figure 5.20 Soil-water characteristic curve of AMK soil.

Chapter 6

Cyclic Simple Shear Tests

6.1 General

This chapter presents the results of 95 strain-controlled cyclic simple shear tests on 12 undisturbed and 7 reconstituted residual soil specimens under both saturated and unsaturated conditions. The effects of mean effective stress, matric suction, and number of loading cycles on shear modulus and damping ratio are investigated.

6.2 Material Tested

Three soil samples: undisturbed Bukit Timah Granite residual soil (AMK), undisturbed Jurong Formation residual soil (JM), and reconstituted Jurong Formation residual soil (NTU) were used. The method of sampling and preparation of soil specimens have been discussed in Section 4.1. Both the undisturbed and reconstituted soil samples were trimmed using a soil lathe to a soil specimen of 70 mm diameter and 20 mm height, i.e., a height-to-diameter ratio (h/d) of 0.29. This ratio conforms to the h/d limit of 0.4 as suggested in ASTM D6528-07 (2007) for direct simple shear test of cohesive soils.

6.3 Test Procedures

6.3.1 Saturated Soils

The soil specimen was first placed onto the bottom platen. A 0.6 mm-thick plain rubber membrane was placed in a split membrane stretcher that was held together with O-rings. The stretched membrane was placed over the specimen and the top platen was gently brought in contact with the soil specimen. The rubber membrane was then released and O-rings were used to secure the rubber membrane to the platens. A small normal seating stress of 5 kPa was applied onto the specimen using

stress-control, following the suggestion by ASTM D6528-07 (2007) for direct simple shear testing of cohesive soils.

To measure the horizontal displacements of the top platen and middle-height of the soil specimen, two proximity transducers were used. Aluminium foil (20 mm x 20 mm square) targets were secured to the top platen and middle-height of the soil specimen using vacuum grease. Proximity transducers were clamped at the two internal tie rods and carefully positioned towards the aluminium foil targets. The set-up for the proximity transducers was shown in Figure 3.1.

The cell chamber was then sealed and water was supplied to provide cell pressure. Once the cell chamber was filled up, the soil specimen underwent the processes of saturation, consolidation and shearing which are described below. The saturation and consolidation procedures as given by Head (1986) were adopted. Shearing was carried out using strain-control.

6.3.1.1 Saturation

The soil specimen was saturated using back-pressure saturation as suggested by Head (1986). During saturation stage, the specimen was saturated by applying a cell pressure, σ_3 , and a back pressure, u_b , under an effective confining pressure of 10 kPa. The pore-water pressure parameter, B , was monitored. Full saturation was assumed to have been achieved when the B value was greater than 0.95 (Head 1986). The final B value, cell pressure, and back pressure, of all the saturated specimens are tabulated in Table 6.1.

Table 6.1 Summary of test programme and resulting parameters for saturated soil specimens

Soil formation	Test no.	Specimen no.	Void ratio, e	Pore-water pressure parameter, B	Cell pressure, σ_3 (kPa)	Back pressure, u_b (kPa)	Loading step no.	Shear strain, γ_c (%)	Ratio of actual displacement, R_δ	Shear modulus correction factor, $c(G)$	Degradation parameter, t_δ	Discrepancy between theoretical and measured horizontal displacement of soil specimen (%)	Volumetric threshold shear strain, γ_{r_c} (%)	Estimated G_{max} (MPa)
Jurong Formation	1	NTU25	0.54	0.99	400	375	1	0.041	0.892	1	0.010	3	0.02 - 0.04	28.06
							2	0.089	0.935	1	0.024	9		
							3	0.186	0.966	1	0.047	8		
							4	0.481	0.984	1	0.097	14		
							5	0.978	0.990	1	0.141	10		
	2	NTU50	0.54	0.98	400	350	1	0.042	0.870	1	0.008	7	0.02 - 0.04	38.87
							2	0.087	0.917	1	0.032	10		
							3	0.183	0.948	1	0.044	7		
							4	0.477	0.974	1	0.097	3		
							5	0.974	0.984	1	0.141	4		
	3	NTU100	0.54	0.97	400	300	1	0.038	0.820	1.03	0.005	17	0.02 - 0.04	53.83
							2	0.083	0.874	1	0.014	13		
							3	0.177	0.913	1	0.029	10		
							4	0.464	0.949	1	0.066	3		
							5	0.953	0.968	1	0.117	3		
	4	NTU200	0.54	0.97	500	300	1	0.036	0.794	1.26	0	28	0.02 - 0.04	74.57
							2	0.079	0.838	1	0.005	28		
							3	0.168	0.865	1	0.011	25		
							4	0.443	0.907	1	0.035	17		
							5	0.924	0.940	1	0.083	12		
Jurong Formation	5	JM50	0.34	0.98	400	350	1	0.039	0.847	1	0.005	20	0.04 - 0.06	32.94
							2	0.084	0.898	1	0.025	19		
							3	0.179	0.929	1	0.027	17		
							4	0.469	0.961	1	0.083	16		
							5	0.964	0.979	1	0.211	18		
	6	JM100	0.34	0.97	400	300	1	0.038	0.843	1	0	9	0.04 - 0.05	45.63
							2	0.083	0.885	1	0.015	7		
							3	0.177	0.920	1	0.021	20		
							4	0.462	0.949	1	0.052	17		
							5	0.951	0.967	1	0.107	20		
	7	JM200	0.34	0.96	500	300	1	0.037	0.812	1.08	0	23	0.05 - 0.07	63.20
							2	0.080	0.855	1	0.015	23		
							3	0.170	0.885	1	0.016	23		
							4	0.447	0.919	1	0.023	19		
							5	0.923	0.940	1	0.057	20		
Bukit Timah Granite	8	AMK50	1.09	0.97	400	350	1	0.041	0.880	1	0.004	14	0.02 - 0.04	34.60
							2	0.088	0.925	1	0.026	13		
							3	0.183	0.955	1	0.032	13		
							4	0.477	0.977	1	0.060	10		
							5	0.974	0.987	1	0.100	9		
	9	AMK100	1.09	0.97	400	300	1	0.038	0.835	1	0.002	15	0.02 - 0.04	47.93
							2	0.082	0.880	1	0.009	14		
							3	0.176	0.915	1	0.018	10		
							4	0.440	0.950	1	0.043	5		
							5	0.954	0.969	1	0.088	3		
	10	AMK200	1.09	0.96	500	300	1	0.039	0.823	1	0	23	0.04 - 0.06	66.38
							2	0.082	0.866	1	0.001	23		
							3	0.173	0.896	1	0.008	20		
							4	0.454	0.930	1	0.013	15		
							5	0.940	0.954	1	0.043	11		

6.3.1.2 Consolidation

After saturation stage, the soil specimen was isotropically consolidated to the desired mean effective stress, σ'_m . During consolidation, the cell pressure was raised to a higher value with respect to the back pressure to reach the desired σ'_m . The valve of the back pressure connected to a GDS pressure-volume controller was kept opened and the back pressure was controlled at the desired values. The amount of water draining out from the soil specimen was recorded by the GDS pressure-volume controller and monitored. Consolidation was assumed to be completed when the water volume change levelled off and excess pore-water pressure fully dissipated.

6.3.1.3 Shearing

Following consolidation to the desired σ'_m , cyclic simple shear tests were conducted at various shear strain levels. The tests were performed under constant height and undrained condition with pore-water pressure measurement, similar to the procedures of Boulanger et al. (1993). The soil specimen was subject to five cyclic strain-controlled loading steps from approximately 0.036% to 0.978% shear strain, which are typical mobilized soil strain ranges for construction of retaining walls, foundations and tunnels under working load condition (Mair 1993; Ng et al. 2000). Most practical problems involving cyclic loading are in the frequency range of 0.001 to 0.1 Hz (wave loading), 0.1 to 10 Hz (seismic loading) and 10 to 100 Hz (traffic loading). According to Shibuya et al. (1995), cyclic stress-strain behaviour of soil is practically independent of the frequency of seismic loading (0.1 to 10 Hz). For cyclic simple shear test, it is typically conducted in the frequency range of 0.1 to 2 Hz, with 0.5 and 1 Hz being the most common frequencies used (Table 2.5). A lower frequency will enable equilibration of pore-water pressure throughout the specimen. Therefore, a loading frequency of 0.5 Hz was used in this study. Ten cycles of controlled cyclic shear strain were applied to the soil specimen at each

loading step. Das (1993) has recommended that dynamic properties of soils determined at the fifth cycle provide fairly reasonable value for practical use. Furthermore, Das (1993) suggested that the number of significant strain cycles is likely to be less than 20. At the end of ten loading cycles at each shear strain, the soil specimen was re-consolidated to its mean effective stress, before starting the next ten loading cycles at the next shear strain level, similar to the procedures of other researchers (Ōhara and Matsuda 1988; Kagawa 1992). The cyclic shear strain was applied in a sinusoidal manner, as the shape of cyclic loading or deformation is typically sinusoidal for seismic event (Dobry and Vucetic 1987; Kim et al. 1991). A sampling rate of 100 points per cycle was used in the data acquisition system during cyclic loading. This sampling rate conforms to ASTM D3999-91 (2003) for cyclic triaxial testing which suggests that the minimum sampling points per cycle is 40.

6.3.2 Unsaturated Soils

To test unsaturated soils, the GCTS cyclic simple shear apparatus was modified to incorporate suction-controlled testing system (Chin et al. 2009). The bottom platen was replaced with one that has a spiral-grooved water compartment and a 5-bar ceramic disk was epoxied onto it as shown in Figure 6.1. The use of the high air-entry ceramic disk is to facilitate a separate control for the pore-air and pore-water pressures, enabling axis-translation technique for matric suction control. In this study, the maximum matric suction applied to the soil specimen was 500 kPa as limited by the 5-bar ceramic disk.

The testing procedure for unsaturated soil is almost the same as saturated soil, except that the soil specimen was pre-equilibrated in the pressure plate apparatus at the desired matric suction before being transferred onto the bottom platen of the cyclic simple shear apparatus. The details of preparing the unsaturated soil specimen using the pressure plate apparatus were presented in Section 4.3.2.

To measure the horizontal displacements of the top platen and middle-height of the soil specimen internally, two proximity transducers were used. Aluminium foil (20 mm x 20 mm square) targets were secured to the top platen and middle-height of the soil specimen using vacuum grease. The proximity transducers were clamped on two different internal tie rods and carefully positioned towards the aluminium foil targets. To measure the vertical displacement internally, a ± 2.5 mm capacity LVDT was clamped on the vertical piston.

The cell chamber was then sealed and water was supplied as a means to provide cell pressure. Once the cell chamber was filled up, the soil specimen underwent the processes of consolidation and shearing as described below.

Figure 6.1 Bottom platen (a) without ceramic disk; (b) with ceramic disk.

6.3.2.1 Consolidation

The soil specimen was consolidated under an isotropic cell pressure, σ_3 , while the pore-air and pore-water pressures were controlled at a value of u_a and u_w , respectively, in which σ_3 was always higher than u_a and u_a was always higher than u_w . Using axis-translation technique, initial matric suction was re-applied to the soil specimen in the cyclic simple shear apparatus. Once there was negligible movement of water in or out of the soil specimen under the applied net confining stress and matric suction, consolidation was deemed to be completed.

6.3.2.2 Shearing

Following consolidation to the desired net confining stress and matric suction, cyclic simple shear tests were conducted at various shear strain levels. Prior to shearing, the water compartment at the bottom platen was flushed to ensure that no air bubble was trapped inside the water compartment. Air can diffuse through the ceramic disk and re-appear in the water compartment during the consolidation stage, and affect the continuity between the pore water and the water in the measuring system, thus resulting in inaccurate pore-water pressure measurement.

Constant water content (CW) condition was chosen for the unsaturated soil tests, because in many field conditions, the pore-air pressure is drained, but pore-water pressure is undrained (Thu et al. 2006). The tests were performed under constant load with drained pore-air and undrained pore-water pressure conditions. During shear, the drainage valve for the pore-air was open and maintained at the u_a at the end of consolidation. However, the drainage valve for the pore water was closed. Therefore, the pore-water pressure, u_w , changed during shear under undrained loading condition and was measured at the base of specimen. Throughout the shearing, the net confining stress ($\sigma_3 - u_a$) remained constant while the matric suction changes as a result of pore-water pressure changes. Furthermore, the soil

volume also changes (i.e., settlement) as pore-air can flow in or out of the soil specimen during shear whereas water content remains constant due to undrained condition of pore water.

The cyclic simple shear loading procedures were similar to those for the saturated soil specimens. The unsaturated soil specimen was subject to five cyclic strain-controlled loading steps from approximately 0.037% to 0.948% shear strain. The frequency of loading was 0.5 Hz for all the tests. Ten cycles of controlled cyclic shear strain were applied to the soil specimen at each loading step. At the end of ten loading cycles at each shear strain, the soil specimen was re-consolidated to its initial net confining stress and matric suction, before starting the ten loading cycles at the next shear strain level. The cyclic shear strain was applied in a sinusoidal manner. A sampling rate of 100 points per cycle was used in the data acquisition system during cyclic loading.

6.4 Testing Programme

6.4.1 Saturated Soils

In undrained simple shear loading, the boundary conditions are generally assumed to be described by a constant vertical load, zero lateral strains, and zero vertical strains. However, it is very difficult to achieve these boundary conditions using typical laboratory apparatuses (Boulanger et al. 1993). An alternative procedure to duplicate the above mentioned boundary condition is constant volume or constant height testing (Ishihara and Yamazaki 1980; Tatsuoka et al. 1982, 1989; Dyvik et al. 1987; Boulanger et al. 1993). Constant height testing was used in this study for saturated soil specimens. In constant height testing, the specimen height was kept constant while the vertical load was allowed to change. This was achieved by locking the vertical loading piston using the feedback system. The volume of the soil specimen was kept constant as drainage of water from the soil specimen was

prevented. As height and volume were kept constant for undrained test, the cross-sectional area of the soil specimen was assumed to remain constant during shearing (Boulanger et al. 1993; Mao and Fahey 2003).

Ten fully saturated soil specimens were tested in this study. The parameters investigated for the saturated soil specimens are the number of loading cycles, shear strain level, and mean effective stress. For undisturbed soils, specimens were tested at three mean effective stresses σ'_m of 50, 100, and 200 kPa. These mean effective stresses corresponded to 1, 2, and 4 times the in-situ effective overburden stress. The range of mean effective stresses used in the testing programme was selected to reflect the state of stress near and above the in-situ stress state. For reconstituted soils, specimens were tested at four mean effective stresses σ'_m of 25, 50, 100, and 200 kPa. These stresses are consistent with the stresses tested for the undisturbed soils, enabling systematic investigations to be carried out. The shear modulus and damping ratio at each shear strain level were determined from the stress-strain curve.

For saturated soil specimens, the specimens were labelled using letters and numbers where the letters refer to the sampling location of the soil sample, and the numbers refer to the mean effective stress σ'_m at which the test was conducted. For example, NTU100 refers to the test where residual soil specimen was sampled at Nanyang Technological University and tested at σ'_m of 100 kPa. Table 6.1 summarizes the test programme and resulting parameters for saturated soil specimens.

Figure 6.2 shows typical test results for constant specimen height cyclic strain-controlled test for a saturated soil specimen JM100. The soil specimen was subjected to a mean effective stress σ'_m of 100 kPa. A peak-to-peak shear displacement of about ± 0.0091 mm was applied to the soil specimen for ten loading cycles (Figure 6.2a). The applied shear displacement was uniform throughout the

ten loading cycles and the resulting shear load is shown in Figure 6.2(b). It can be seen that the shear load over time was constant, and no degradation was observed. This is mainly because there was no excess pore-water pressure build-up (Figure 6.2g) at this shear strain level (i.e., volumetric threshold shear strain γ_{tv} was not exceeded). For shear strain above γ_{tv} test, deterioration of shear load with time was observed (e.g. test result given in Figure B29 of Appendix B). Figures 6.2(c) and 6.2(d) show the top platen horizontal displacement, and the specimen mid-height shear displacement, respectively, measured by the internal proximity transducers. The rocking movement of the top platen was obvious even at the small shear displacement. The measured average top platen horizontal displacement was about 0.0014 mm. The measured average specimen mid-height shear displacement was about 0.0048 mm. This value deviates slightly from the average value of the applied shear displacement at the bottom of the specimen and the horizontal displacement of the top platen. This implies that the soil specimen was not in simple shear condition due to the rocking of the top platen. Depending on the severity of the rocking effect, a correction factor as suggested in Chapter 3 was needed for the shear modulus determination. The responses of normal stress and displacement from the vertical actuator with time are shown in Figures 6.2(e) and 6.2(f), respectively. Constant height of the specimen was essentially achieved by maintaining the position of the top platen constant (i.e., zero vertical strain) whereas the normal stress was allowed to fluctuate over time. The first, second, fifth, and tenth cycle stress-strain curves are shown in Figure 6.2(h) and the shear modulus and damping ratio can be evaluated. Detailed test results of all the saturated soil specimens are given in Appendix D.

Figure 6.2 Typical test results for constant height cyclic test of JM100 at $\gamma_c = 0.038\%$.

6.4.2 Unsaturated Soils

For dry or unsaturated soil, drained cyclic simple shear tests with constant load application were widely used (Silver and Seed 1971a; Chu and Vucetic 1992; Hsu and Vucetic 2004; and Whang et al. 2004). The test permits volume change to occur in which seismic compression and earthquake-induced settlement can be investigated (Silver and Seed 1971a; Hsu and Vucetic 2004; and Whang et al. 2004). In constant vertical load testing, the soil specimen is subjected to constant vertical load while the vertical displacement is allowed to fluctuate. This is achieved by maintaining the vertical load constant at the end of consolidation stage using the feedback system.

Nine unsaturated soil specimens were tested at constant vertical load and constant water content conditions. The parameters investigated for the unsaturated soils are the number of loading cycles, shear strain level, and initial matric suction. For undisturbed soils, specimens were tested at a net confining stress of 50 kPa corresponding to the in-situ effective overburden stress. Three different initial matric suctions of 100, 200, 400 kPa were selected for testing. These values are near and above the air-entry values of the soils (i.e., boundary effect and transition zones) to capture their unsaturated behaviour in cyclic simple shear condition. For reconstituted soils, specimens were tested at a net confining stress of 100 kPa corresponding to its consolidation pressure. Similar to undisturbed soil specimens, three different initial matric suctions of 100, 200, 400 kPa were selected for testing. The shear modulus and damping ratio at each shear strain level were evaluated from the stress-strain curve.

The unsaturated soil specimens were labelled using letters followed by two numbers, separated by a dash. The letters refer to the location of the soil sample, the first number refers to the net confining stress and the second number refers to the initial

matric suction at which the test was conducted. For example, NTU100-200 refers to the test where residual soil specimen was sampled at Nanyang Technological University and tested at net confining stress of 100 kPa and an initial matric suction of 200 kPa. Table 6.2 summarizes the test programme and resulting parameters for unsaturated soil specimens.

Nine unsaturated soil specimens were tested at constant vertical load and constant water content conditions. The parameters investigated for the unsaturated soils are the number of loading cycles, shear strain level, and initial matric suction. For undisturbed soils, specimens were tested at a net confining stress of 50 kPa corresponding to the in-situ effective overburden stress. Three different initial matric suctions of 100, 200, 400 kPa were selected for testing. These values are near and above the air-entry values of the soils (i.e., boundary effect and transition zones) to capture their unsaturated behaviour in cyclic simple shear condition. For reconstituted soils, specimens were tested at a net confining stress of 100 kPa corresponding to its consolidation pressure. Similar to undisturbed soil specimens, three different initial matric suctions of 100, 200, 400 kPa were selected for testing. The shear modulus and damping ratio at each shear strain level were evaluated from the stress-strain curve.

The unsaturated soil specimens were labelled using letters followed by two numbers, separated by a dash. The letters refer to the location of the soil sample, the first number refers to the net confining stress and the second number refers to the initial matric suction at which the test was conducted. For example, NTU100-200 refers to the test where residual soil specimen was sampled at Nanyang Technological University and tested at net confining stress of 100 kPa and an initial matric suction of 200 kPa. Table 6.2 summarizes the test programme and resulting parameters for unsaturated soil specimens.

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Table 6.2 Summary of test programme and resulting parameters for unsaturated soil specimens

Soil samples	Test no.	Specimen no.	Soil condition	Cell pressure, σ_3 (kPa)	Pore-air pressure, u_a (kPa)	Pore-water pressure, u_w (kPa)	Loading step no.	Shear strain, γ_c (%)	Ratio of actual displacement, R_δ	Shear modulus correction factor, $c(G)$	Degradation parameter, t_δ	Discrepancy between theoretical and measured horizontal displacement of soil specimen (%)	Volumetric threshold shear strain, γ_v (%)	Estimated G_{max} (MPa)
Jurong Formation	1	NTU100-100	R	250	150	50	1	0.039	0.824	1	0	14	0.08 - 0.17	61.06
							2	0.082	0.865	1	0	23		
							3	0.173	0.895	1	0	23		
							4	0.455	0.930	1	0	15		
							5	0.938	0.953	1	0.030	14		
	2	NTU100-200	R	350	250	50	1	0.037	0.807	1.13	0	8	0.1 - 0.2	67.12
							2	0.079	0.831	1	0	11		
							3	0.166	0.861	1	0	21		
							4	0.434	0.893	1	0.006	13		
							5	0.911	0.923	1	0.015	14		
	3	NTU100-400	R	550	450	50	1	0.037	0.789	1.32	0	12	0.2 - 0.35	70.63
							2	0.078	0.801	1.18	0	24		
							3	0.158	0.827	1	0	24		
							4	0.416	0.856	1	0.005	20		
							5	0.879	0.894	1	0.031	16		
Jurong Formation	4	JM50-100	U	200	150	50	1	0.038	0.812	1.08	0	16	0.08 - 0.17	36.07
							2	0.080	0.846	1	0.004	16		
							3	0.169	0.872	1	0.006	15		
							4	0.440	0.896	1	0.009	9		
							5	0.917	0.926	1	0.009	5		
	5	JM50-200	U	300	250	50	1	0.037	0.807	1.13	0	17	0.1 - 0.17	37.71
							2	0.079	0.840	1	0.003	13		
							3	0.165	0.859	1	0.010	17		
							4	0.434	0.893	1	0.018	13		
							5	0.920	0.933	1	0.020	4		
	6	JM50-400	U	500	450	50	1	0.037	0.799	1.2	0	14	0.08 - 0.17	40.28
							2	0.078	0.823	1	0	11		
							3	0.161	0.842	1	0.002	12		
							4	0.425	0.875	1	0.005	11		
							5	0.904	0.915	1	0.020	4		
Bukit Timah Granite	7	AMK50-100	U	200	150	50	1	0.040	0.852	1	0	17	0.06 - 0.08	39.49
							2	0.084	0.885	1	0.007	18		
							3	0.176	0.950	1	0.009	17		
							4	0.460	0.944	1	0.010	18		
							5	0.948	0.960	1	0.016	15		
	8	AMK50-200	U	300	250	50	1	0.039	0.828	1	0	8	0.08 - 0.17	43.63
							2	0.081	0.853	1	0	15		
							3	0.173	0.899	1	0.005	15		
							4	0.452	0.926	1	0.009	17		
							5	0.936	0.949	1	0.020	18		
	9	AMK50-400	U	500	450	50	1	0.039	0.824	1	0	28	0.2 - 0.3	51.00
							2	0.081	0.860	1	0	28		
							3	0.170	0.883	1	0.011	28		
							4	0.442	0.908	1	0.011	26		
							5	0.926	0.938	1	0.011	24		

Note R: Reconstituted and U: Undisturbed

Figure 6.3 shows the typical test results for constant load cyclic strain-controlled test for an unsaturated soil specimen JM50-200 under constant water content condition. The soil specimen was subjected to a net confining stress of 50 kPa and an initial matric suction of 200 kPa. A peak-to-peak shear displacement of about ± 0.0188 mm was applied to the soil specimen for ten loading cycles (Figure 6.3a). The applied shear displacement was uniform throughout the ten loading cycles and the resulting shear load is shown in Figure 6.3(b). It can be seen that the shear load over time was approximately constant, and no deterioration was observed. This was mainly because there was no excess pore-water pressure build-up (Figure 6.3g) at this shear strain level, i.e., volumetric threshold shear strain γ_{tv} was not exceeded. For shear strain above γ_{tv} test, deterioration of shear load with time was observed (e.g. test result given in Figure C24 of Appendix C). Figure 6.3(c) and 6.3(d) show the top platen horizontal displacement, and the specimen mid-height shear displacement, respectively, measured by the internal proximity transducers. The rocking of the top platen can be observed from the top platen movement in Figure 6.3(c). The measured average top platen horizontal displacement was about 0.0030 mm. The measured average specimen mid-height shear displacement was about 0.0095 mm. This value deviates slightly from the average value of the applied shear displacement at the bottom of the specimen and the horizontal displacement of the top platen. Similar to the cyclic simple shear tests for saturated soil specimens, a correction factor as suggested in Chapter 3 is needed for the shear modulus determination. The responses of normal stress and displacement from the vertical actuator with time are shown in Figures 6.3(e) and 6.3(f), respectively. Constant load applied on the specimen was essentially achieved by maintaining the load of the top platen constant (i.e., zero vertical stress change) whereas the vertical displacement was allowed to fluctuate over the time. The first, second, fifth, and tenth cycle stress-strain curves are shown in Figure 6.3(h) and the shear modulus and damping ratio can be evaluated. Detailed test results of all the unsaturated soil specimens are given in Appendix E.

Figure 6.3 Typical test results for constant load cyclic test of JM50-200 at $\gamma_c = 0.079\%$.

6.5 Volumetric Threshold Shear Strain

6.5.1 Saturated Soils

For fully saturated soils subjected to cyclic loading, the development of cyclic pore-water pressure and the associated cyclic stiffness degradation with number of loading cycles are largely related to volumetric threshold shear strain, γ_{tv} (Dobry et al. 1982; Hsu and Vucetic 2006). Volumetric threshold shear strain is defined as the shear strain level which divides non-destructive small-strain and destructive large-strain behaviour (Dobry et al. 1982). For saturated sands and clays subjected to moderate or large cyclic shear strains (i.e., above γ_{tv}) under undrained condition, the soils experience a pore-water pressure build-up which accumulates continuously and rapidly with number of loading cycles, resulting in reduction of effective stress. Consequently, the shear modulus decreases with number of loading cycles. This is known as degradation effect (Dobry et al. 1982; Hsu and Vucetic 2006). Below γ_{tv} , pore-water pressure does not build up and effective stress remained unchanged during cyclic loading. Thus shear modulus remained unchanged with the number of loading cycles for shear strains below γ_{tv} .

Hsu and Vucetic (2006) used the variation of normalized excess pore-water pressure ($\Delta u/\sigma'_m$) with shear strain to identify the volumetric threshold shear strain. Theoretically, a unique volumetric threshold shear strain can be obtained by locating the point at which the normalized excess pore-water pressure with shear strain curves for different number of loading cycles converge at the $(\Delta u/\sigma'_m) = 0$ axis. However, rigorous determination of volumetric threshold shear strain is not possible due to the normalized excess pore-water pressure with shear strain curves for different number of loading cycles not consistently converging at exactly the same point. Consequently, volumetric threshold shear strains γ_{tv} are typically reported as a range value, in which they are identified as shear strains beyond which the normalized excess pore-water pressure starts to increase markedly.

A typical test result (JM100) is used to illustrate the determination of volumetric threshold shear strain of saturated soil specimen as shown in Figure 6.4. Figure 6.4(a) shows the strain-time histories for two shear strains, 0.038%, and 0.951%, for ten loading cycles. The two shear strains were one order of magnitude apart. The pore-water pressure responses corresponding to the two applied shear strains are shown in Figure 6.4(b). From Figure 6.4(b), it can be seen that there was essentially no excess pore-water pressure change, induced by the shear strain of 0.038%, indicating that the volumetric threshold shear strain was not exceeded. For the applied shear strain of 0.951%, the excess pore-water pressure increased with number of loading cycles, indicating that the volumetric threshold shear strain was exceeded.

The variation of normalized excess pore-water pressure ($\Delta u/\sigma'_m$) with shear strain for different number of loading cycles (discussed above) was used in this study to identify the volumetric threshold shear strain for saturated soil specimens. Figure 6.5 shows the normalized excess pore-water pressure with shear strain for different number of loading cycles, 2, 5, 10, for saturated soil specimen JM100. It can be seen that the normalized excess pore-water pressure build-up increased markedly with shear strain above the shear strain range of 0.04% – 0.05%. Therefore, the volumetric threshold shear strain, γ_{tv} for saturated soil specimen JM100 can be taken as 0.04% - 0.05%.

Using the above procedures, volumetric threshold shear strain, γ_{tv} for the ten saturated soil specimens, four reconstituted and six undisturbed, were determined. Figures 6.6, 6.7 and 6.8 show the variation of normalized excess pore-water pressure build-up with shear strain for different number of loading cycles for NTU (reconstituted), JM, and AMK (undisturbed) soil specimens, respectively. The normalized excess pore-water pressure with shear strain curves were plotted for

loading cycles, 2, 5, and 10. Overall, the three curves for saturated soil specimens tend to converge to a range of shear strain that defines the range of volumetric shear strain. Such a trend has also been observed by Hsu and Vucetic (2006). In some cases, the volumetric threshold shear strain of the saturated soil specimens can only be estimated by extrapolating the curves, due to the limitation of the cyclic simple shear apparatus as discussed in Chapter 3.

(a) Strain-time history

(b) Pore-water pressure-time history

Figure 6.4 Determination of volumetric threshold shear strain for saturated soil specimen JM100.

Figure 6.5 Identification of volumetric threshold shear strain for saturated soil specimen JM100.

The volumetric threshold shear strains γ_{tv} of the undisturbed saturated soil specimens ranged from 0.04% to 0.07% for JM specimens, and 0.02% to 0.06% for AMK specimens, with higher values of γ_{tv} being associated with the higher mean effective stress σ'_m . However, for reconstituted NTU soil specimens, γ_{tv} were estimated to be from 0.02% – 0.04%, irrespective of mean effective stresses σ'_m investigated (i.e. 25 to 200 kPa). The cyclic γ_{tv} obtained for the saturated residual soil specimens are summarized in Table 6.1.

Figure 6.6 Relationship between normalized excess pore-water pressure build-up and shear strain for different of number loading cycles for reconstituted NTU soil specimens.

Figure 6.7 Relationship between normalized excess pore-water pressure build-up and shear strain for different of number loading cycles for undisturbed JM soil specimens.

Figure 6.8 Relationship between normalized excess pore-water pressure build-up and shear strain for different of number loading cycles for undisturbed AMK soil specimens.

6.5.2 Unsaturated Soils

For dry or unsaturated soil subjected to moderate or large cyclic shear strains (i.e., above γ_{tv}) under drained condition, the soils experience a permanent volume change, i.e. settlement, which accumulates continuously and rapidly with number of loading cycles. Below γ_{tv} , volume remained unchanged during cyclic loading, i.e. no settlement (Chu and Vucetic 1992; Hsu and Vucetic 2004).

Chu and Vucetic (1992) and Hsu and Vucetic (2004) used the change of vertical strain, $\Delta\varepsilon_v$, with shear strain to determine the volumetric threshold shear strain of dry and unsaturated soils. Theoretically, a unique volumetric threshold shear strain can be obtained by locating the point at which the change of vertical strain with shear strain curves for different number of loading cycles converge at the $\Delta\varepsilon_v = 0$ axis. However, rigorous determination of volumetric threshold shear strain is not possible due to the change of vertical strain with shear strain curves for different number of loading cycles not consistently converging at exactly the same point. Consequently, volumetric threshold shear strains γ_{tv} are typically reported as a range value, in which they are identified as shear strains beyond which the change of vertical strain starts to increase markedly.

A typical test result (JM50-200) is used to illustrate the determination of volumetric threshold shear strain of unsaturated soil specimen as shown in Figure 6.9. Figure 6.9(a) shows the strain-time histories for two shear strains, 0.079%, and 0.920%, for ten loading cycles. The two shear strains are about one order of magnitude apart. The change of vertical strain responses corresponding to the two applied shear strains are shown in Figure 6.9(b). From Figure 6.9(b), it can be seen that there was essentially no change of vertical strain, induced by the shear strain of 0.079%, indicating that the volumetric threshold shear strain was not exceeded. For the applied shear strain of 0.920%, the change of vertical strain increased with number

of loading cycles, indicating that the volumetric threshold shear strain was exceeded.

Figure 6.9 Determination of volumetric threshold shear strain for unsaturated soil specimen JM50-100.

The variation of change of vertical strain with shear strain for different number of loading cycles (discussed above) was used in this study to identify the volumetric threshold shear strain for unsaturated soil specimens. Figure 6.10 shows the relationship between change of vertical strain and cyclic shear strain for loading cycles, 2, 5, and 10, for soil specimen JM50-200. It can be seen that the change of vertical strain increased markedly with shear strain above the shear strain range of about 0.1% – 0.17%. Therefore, the volumetric threshold shear strain, γ_{tv} for the unsaturated soil specimen JM50-200 can be taken as 0.1% – 0.17%.

Figure 6.10 Identification of volumetric threshold shear strain for unsaturated soil specimen JM50-200.

Using the above procedure, volumetric threshold shear strain, γ_{tv} for nine unsaturated soil specimens; three reconstituted and six undisturbed, were identified. Figures 6.11, 6.12 and 6.13 show the change of vertical strain with shear strain for different loading cycles for NTU (reconstituted), JM, and AMK (undisturbed) soil specimens, respectively. The change of vertical strain with shear strain curves were plotted for loading cycles 2, 5, and 10.

The volumetric threshold shear strains of the unsaturated soil specimens ranged from 0.08% to 0.35% for NTU specimens, 0.08% to 0.17% for JM specimens, and 0.06% to 0.3% for AMK specimens. The higher volumetric threshold shear strain values tended to be associated with higher initial matric suctions. From Figures 6.11 through 6.13, it can be seen that the permanent settlement tended to occur at larger shear strains for specimens with higher initial matric suctions. The cyclic volumetric threshold shear strains γ_{tv} obtained for the unsaturated residual soil specimens are summarized in Table 6.2.

Figure 6.11 Relationship between change of vertical strain and shear strain for different loading cycles for NTU soil specimens.

Figure 6.12 Relationship between change of vertical strain and shear strain for different loading cycles for JM soil specimens.

Figure 6.13 Relationship between change of vertical strain and shear strain for different loading cycles for AMK soil specimens.

6.5.3 Comparison of Volumetric Threshold Shear Strain with Other Study

Figure 6.14 shows the comparison of volumetric threshold shear strain with those of Vucetic (1994). It can be seen that the volumetric threshold shear strains for the saturated residual soil specimens largely fall into the band proposed by Vucetic (1994). The volumetric threshold shear strains for the unsaturated soil specimens were generally larger than the corresponding saturated soil specimens. Similar trend was observed in Vucetic (1994) for unsaturated clays. The volumetric threshold shear strains for the unsaturated soil specimens appeared to be independent of plasticity index and on average, their volumetric threshold shear strain was about 0.15%.

Figure 6.14 Comparison of volumetric threshold shear strain with those of Vucetic (1994).

6.6 Effect of Number of Loading Cycles

6.6.1 Saturated Soil

Figure 6.15 illustrates typical test results for cyclic strain-controlled test for γ_c below γ_{tv} , for saturated soil specimen JM 100. A peak-to-peak shear strain of $\pm 0.038\%$ was applied for ten loading cycles (Figure 6.15a). From Figure 6.15(b), it can be seen that there was no excess pore-water pressure build-up at this shear strain level, indicating that the volumetric threshold shear strain γ_{tv} was not exceeded. Consequently, a constant shear load was observed throughout the ten loading cycles (Figure 6.15c), as the mean effective stress remained unchanged. Therefore, the cyclic stress-strain loop remained unchanged over the ten-loading cycles (i.e., no degradation) as seen in Figure 6.15(d).

Figure 6.15 Typical test results for cyclic strain-controlled test for $\gamma_c < \gamma_{tv}$ for saturated soil specimen JM100.

Figure 6.16 illustrates typical test results for cyclic strain-controlled test for γ_c above γ_{tv} , for saturated soil specimen JM100. A peak-to-peak shear strain of $\pm 0.951\%$ was applied for ten loading cycles (Figure 6.16a). From Figure 6.16(b), it can be seen that excess pore-water pressure build-up accumulated continuously and rapidly with the number of loading cycles at this shear strain, indicating that the volumetric threshold shear strain was exceeded. Consequently, the shear load deteriorated over the ten loading cycles (Figure 6.16c), due to a reduction of mean effective stress. The stiffness degradation can clearly be seen in the cyclic stress-strain loop in Figure 6.16(d).

Figure 6.16 Typical results for cyclic strain-controlled test for $\gamma_c > \gamma_{tv}$ for saturated soil (JM100).

The effect of number of loading cycles on shear modulus for saturated soil specimen JM100 is shown in Figure 6.17. From Figure 6.17(a), it can be seen that at shear strain of 0.038%, there was fluctuation of shear modulus with number of loading cycles, most probably due to electrical noise of the instrumentations. Beyond shear strain of 0.038%, it was found that the shear modulus of the saturated soil specimen decreased with number of loading cycles (i.e. degradation), resulting from the development of excess pore-water pressure (i.e. volumetric threshold shear strain being exceeded). This finding agrees with that of Thiers and Seed (1968), Hardin and Drnevich (1972a) and Matasović and Vucetic (1995). The development of excess pore-water pressure was more prominent at larger shear strains, and consequently, the effect of number of loading cycles on shear modulus was more obvious at larger shear strain as seen in Figure 6.17(b).

The effect of number of loading cycles on damping ratio for saturated soil specimen JM 100 is shown in Figure 6.18. From Figure 6.18(a), it can be seen that at shear strain of 0.038%, similar to shear modulus, there was fluctuation of damping ratio with number of loading cycles, due to electrical noise of the instrumentations. Beyond shear strain of 0.038%, it was found that the damping ratio of the saturated soil specimen decreased with number of loading cycles, resulting from the development of excess pore-water pressure (i.e. volumetric threshold shear strain being exceeded and occurrence of permanent microstructure change). This result is similar to the findings obtained by Hardin and Drnevich (1972a) who found that damping ratio decreases with number of loading cycles for both cohesive and cohesionless soils. The development of excess pore-water pressure was more prominent at larger shear strains, and consequently, the effect of number of loading cycles on damping ratio was more obvious at larger shear strains as seen in Figure 6.18(b).

(a) Variation of shear modulus with number of loading cycles

(b)) Variation of shear modulus at different number of loading cycles with shear strain

Figure 6.17 Effect of number of loading cycles on shear modulus for soil specimen JM100.

(a) Variation of damping ratio with number of loading cycles

(b) Variation of damping ratio at different number of loading cycles with shear strain

Figure 6.18 Effect of number of loading cycles on damping ratio for soil specimen JM100.

6.6.2 Unsaturated Soil

Figure 6.19 illustrates typical test results for cyclic strain-controlled test for γ_c below γ_{tv} , for unsaturated soil specimen JM50-200. A constant shear strain of 0.079% was applied for ten loading cycles (Figure 6.19a). From Figures 6.19(b) and 6.19(c), it can be seen that there was no excess pore-water pressure build-up, and vertical strain change, respectively, at this shear strain level, indicating that the volumetric threshold shear strain was not exceeded. Consequently, a constant shear load was observed throughout the ten loading cycles (Figure 6.19d), as the net confining stress and initial matric suction remained unchanged. Therefore, the cyclic stress-strain loop remained unchanged over the ten-loading cycles (i.e., no degradation) as seen in Figure 6.19(e).

Figure 6.20 illustrates typical test results for cyclic strain-controlled test for γ_c above γ_{tv} , for unsaturated soil specimen JM50-200. A constant shear strain of 0.920% was applied for ten loading cycles (Figure 6.20a). Figures 6.20(b) and 6.20(c) show build-up of excess pore-water pressure and accumulation of vertical strain continuously and slowly with the number of loading cycles at this shear strain, respectively, indicating that γ_{tv} was exceeded. The occurrence of settlement indicated that air drained out from the void and rearrangement of soil grains had taken place (i.e., constant air pressure is maintained). With the use of axis-translation technique, positive pore-water pressure was measured and hence the pore-water pressure response should be comparable to the pore-water pressure response in saturated soil test. A change of matric suction will be represented by a change of pore-water pressure, as pore-air pressure was kept drained and constant. As the soil specimen was relatively short, i.e. 20 mm, it can be expected that the pore-water pressure remains relatively uniform throughout the specimen during the test duration of 20s. Following pore-water pressure build-up, the peak shear load deteriorated over the ten loading cycles (Figure 6.20d), due to a reduction of initial

matric suction. The stiffness degradation can clearly be seen in the cyclic stress-strain loop in Figure 6.20(e).

Figure 6.19 Typical results for cyclic strain-controlled test for $\gamma_c < \gamma_{tv}$ for unsaturated soil specimen JM50-200.

Figure 6.20 Typical results for cyclic strain-controlled test for $\gamma_c > \gamma_{tv}$ for unsaturated soil specimen JM50-200.

The effect of number of loading cycles on shear modulus for unsaturated soil specimen JM50-200 is shown in Figure 6.21. From Figure 6.21(a), it can be seen that below shear strain of about 0.1%, shear modulus remained almost constant with number of loading cycles. This was due to insignificant development of excess pore-water pressure which could result in degradation of shear modulus. Note that at shear strain of 0.037%, there was fluctuation of shear modulus with number of loading cycles, due to electrical noise of the instrumentations. Beyond shear strain of 0.1%, shear modulus decreased with number of loading cycles (i.e. degradation), resulting from the development of excess pore-water pressure and thus reduction of matric suction. The development of excess pore-water pressure was more prominent at larger shear strains, and consequently, the effect of number of loading cycles on shear modulus was more obvious at larger shear strain as seen in Figure 6.21(b). However, the degradation of shear modulus of unsaturated soil over the number of loading cycle was less significant compared to that of saturated soil.

The effect of number of loading cycles on damping ratio for unsaturated soil specimen JM50-200 is shown in Figure 6.22. From Figure 6.22(a), it can be seen that below shear strain of about 0.1%, damping ratio remained almost constant with number of loading cycles. This was due to insignificant development of vertical strain and excess pore-water pressure which could result in permanent microstructure changes in the soil. Similar to shear modulus, the damping ratio fluctuated with number of loading cycles at shear strain of 0.037%, due to electrical noise of the instrumentations. Beyond shear strain of 0.1%, damping ratio decreased with number of loading cycles, resulting from the development of vertical strain (i.e. volumetric threshold shear strain being exceeded and occurrence of permanent microstructure change) and excess pore-water pressure. The development of vertical strain and excess pore-water pressure were more prominent at larger shear strains, and consequently, the effect of number of loading cycles on damping ratio was more obvious at larger shear strain as seen in Figure 6.22(b). However, the decrement of damping ratio of unsaturated soil specimen with number of loading cycle was less significant compared to that of saturated soil specimen.

(a) Variation of shear modulus with number of loading cycles

(b) Variation of shear modulus at different number of loading cycles with shear strain

Figure 6.21 Effect of number of loading cycles on shear modulus for JM50-200.

(a) Variation of damping ratio with number of loading cycles

(b) Variation of damping ratio at different number of loading cycles with shear strain

Figure 6.22 Effect of number of loading cycles on damping ratio for JM50-200.

6.7 Effect of Mean Effective Stress for Saturated Soil

Shear modulus and damping ratio determined at the fifth cycle are reported in the following section following Das (1993) recommendation that the dynamic properties determined at fifth cycle are likely to provide reasonable values for all practical purposes as number of significant cycles is likely less than 20 in most seismic events. The fifth load cycle shear modulus G and damping ratio D are also approximately the average G and D for the ten loading cycles and therefore they are denoted as G_{avg} and D_{avg} . Typical uncorrected and “corrected” shear moduli (i.e. before and after applying Eq. 3.16) and damping ratios (i.e. before and after applying Eq. 3.20) with shear strains at various mean effective stresses for AMK undisturbed residual soil specimens are shown in Figures 6.23 and 6.24, respectively. The uncorrected shear modulus and damping ratio of other saturated soil specimens (JM and NTU) are given in Appendix D. The amounts of shear modulus and damping ratio corrections of saturated residual soil specimens are tabulated in Table 6.3.

The effects of mean effective stress σ'_m on “corrected” G_{avg} and D_{avg} are shown in Figures 6.24 (AMK undisturbed residual soil specimens) and 6.25 (JM undisturbed residual soil specimens), and 6.26 (NTU reconstituted residual soil specimens). For all the tested soils, G_{avg} increases whereas D_{avg} decreases with increasing mean effective stress σ'_m . This observation agrees with observations for sands (Silver and Seed 1971a; Tatsuoka et al. 1978), clays (Kokusho et al. 1982), and residual soils (Macari and Hoyos 1996).

Figure 6.23 Uncorrected shear modulus and damping ratio for saturated AMK specimens.

Figure 6.24 "Corrected" shear modulus and damping ratio for saturated AMK specimens.

Table 6.3 Uncorrected and “corrected” shear modulus and damping ratio for saturated residual soil specimens.

Specimen no.	Shear strain, γ_c (%)	Average shear modulus, G_{avg} (MPa)			Average damping ratio, D_{avg} (%)		
		Uncorrected	“Corrected”	Amount of correction, [“Corrected” – Uncorrected]	Uncorrected	“Corrected”	Amount of correction, [“Corrected” – Uncorrected]
NTU25	0.041	8.03	8.03	0	28.53	16.26	12.27
	0.089	4.59	4.59	0	28.58	19.52	9.05
	0.186	2.64	2.64	0	30.54	23.81	6.74
	0.481	1.28	1.28	0	30.95	25.35	5.60
	0.978	0.74	0.74	0	31.03	26.15	4.88
NTU50	0.042	11.01	11.01	0	23.12	12.91	10.21
	0.087	6.77	6.77	0	24.99	17.88	7.11
	0.183	4.25	4.25	0	26.67	21.73	4.94
	0.477	2.19	2.19	0	27.52	23.80	3.72
	0.974	1.33	1.33	0	27.65	24.60	3.05
NTU100	0.038	16.51	16.98	0.41	19.79	10.88	8.91
	0.083	10.80	10.80	0	18.11	12.10	6.01
	0.177	7.08	7.08	0	20.77	16.95	3.82
	0.464	3.75	3.75	0	24.27	21.73	2.54
	0.953	2.29	2.29	0	25.91	23.99	1.92
NTU200	0.036	21.27	26.78	5.51	14.93	6.14	8.79
	0.079	15.70	15.70	0	14.93	9.94	4.99
	0.168	12.16	12.16	0	15.77	12.93	2.84
	0.443	7.48	7.48	0	17.81	16.11	1.70
	0.924	4.62	4.62	0	19.24	17.99	1.26
JM50	0.039	11.99	11.99	0	21.47	9.90	11.57
	0.084	8.11	8.11	0	21.60	14.52	7.08
	0.179	5.49	5.49	0	23.35	18.85	4.50
	0.469	2.99	2.99	0	24.40	21.22	3.18
	0.964	1.55	1.55	0	28.70	26.00	2.70
JM100	0.038	14.95	14.95	0	20.04	6.03	14.01
	0.083	10.61	10.61	0	17.65	11.44	6.21
	0.177	7.46	7.46	0	19.84	16.11	3.73
	0.462	4.48	4.48	0	20.72	18.32	2.40
	0.951	2.98	2.98	0	23.72	22.13	1.59
JM200	0.037	18.66	20.24	1.58	13.82	3.78	10.04
	0.080	14.57	14.57	0	13.24	7.82	5.42
	0.170	10.98	10.98	0	15.46	12.38	3.08
	0.447	7.15	7.15	0	16.74	14.95	1.79
	0.923	5.01	5.01	0	20.48	19.36	1.12
AMK50	0.041	9.54	9.54	0	25.80	12.32	13.48
	0.088	5.86	5.86	0	24.75	15.13	9.62
	0.183	3.67	3.67	0	26.70	19.81	6.89
	0.477	1.91	1.91	0	28.64	23.88	4.76
	0.974	1.12	1.12	0	30.07	26.38	3.70
AMK100	0.038	14.21	14.21	0	20.82	10.26	10.56
	0.082	10.22	10.22	0	17.90	11.37	6.53
	0.176	6.98	6.98	0	20.45	16.48	3.97
	0.440	4.11	4.11	0	21.86	19.05	2.80
	0.954	2.34	2.34	0	25.23	23.30	1.93
AMK200	0.039	17.15	17.15	0	16.78	7.69	9.09
	0.082	12.74	12.74	0	16.17	10.75	5.42
	0.173	9.48	9.48	0	17.47	14.19	3.28
	0.454	5.86	5.86	0	19.10	17.10	2.00
	0.940	3.69	3.69	0	23.68	22.35	1.33

Figure 6.25 Effect of mean effective stress on shear modulus and damping ratio for saturated JM specimens.

Figure 6.26 Effect of mean effective stress on shear modulus and damping ratio for saturated NTU specimens.

6.8 Effect of Matric Suction for Unsaturated Soil

Shear modulus and damping ratio determined at the fifth cycle denoted as G_{avg} and D_{avg} are reported in the following section. Typical uncorrected and “corrected” shear moduli (i.e. before and after applying Eq. 3.16) and damping ratios (i.e. before and after applying Eq. 3.20) with shear strains at various initial matric suctions for AMK undisturbed residual soil specimens are shown in Figures 6.27 and 6.28, respectively. The uncorrected shear modulus and damping ratio of other unsaturated soil specimens (JM and NTU) are given in Appendix E. The amounts of shear modulus and damping ratio corrections of unsaturated residual specimens are tabulated in Table 6.4.

The effects of matric suction on G_{avg} and D_{avg} of the tested residual soil specimens are shown in Figures 6.28 (AMK soil specimens), 6.29 (JM soil specimens), and 6.30 (NTU soil specimens). For reconstituted soil specimens, it can be seen that G_{avg} increases and D_{avg} decreases with increasing matric suction (Figure 6.30). This trend is in good agreement with Lenart (2006) who performed resonant column test for reconstituted silt for shear strains below 0.03%. Lenart (2006) suggested that at small shear strains ($\gamma_c < 0.03\%$), the shear modulus increases and damping ratio decreases with decreasing moulding water content. Similar to reconstituted soil specimens, Figures 6.28 and 6.29 show that G_{avg} increases and D_{avg} decreases with increasing matric suction for undisturbed soil specimens.

The variations of G_{avg} and D_{avg} with matric suction for different shear strains are shown in Figures 6.31 (AMK soil specimens), 6.32 (JM soil specimens), and 6.33 (NTU soil specimens). It can be seen that G_{avg} increases and D_{avg} decreases with matric suction for all the tested soil specimens. Significant increment of G_{avg} and decrement of D_{avg} were found for matric suctions ranging from 0 to 200 kPa (i.e. boundary effect zone and early stage of transition zone). Beyond matric suction of

200 kPa, G_{avg} and D_{avg} tend toward a threshold value. A similar trend was obtained by Mancuso et al. (2002) who concluded that the effect of matric suction on very small strain shear modulus, G_{max} was more pronounced in the matric suction range from 0 to about 200 kPa, and at higher matric suctions, G_{max} tends toward a threshold value depending on the mean net stress level.

Figure 6.27 Uncorrected shear modulus and damping ratio for unsaturated AMK specimens.

Figure 6.28 “Corrected” shear modulus and damping ratio for unsaturated AMK specimens.

Table 6.4 Uncorrected and “corrected” shear modulus and damping ratio for unsaturated residual soil specimens.

Specimen no.	Shear strain, γ_c (%)	Average shear modulus, G_{avg} (MPa)			Average damping ratio, D_{avg} (%)		
		Uncorrected	“Corrected”	Amount of correction, “Corrected” – Uncorrected	Uncorrected	“Corrected”	Amount of correction, “Corrected” – Uncorrected
NTU100-100	0.039	18.18	18.18	0	15.92	7.35	8.57
	0.082	13.30	13.30	0	13.76	8.26	5.51
	0.173	9.16	9.16	0	15.92	12.40	3.52
	0.455	5.82	5.82	0	17.59	15.49	2.10
	0.938	3.78	3.78	0	21.52	20.11	1.41
NTU100-200	0.037	18.53	21.00	2.47	15.21	5.94	9.27
	0.079	17.51	17.51	0	11.53	6.75	4.78
	0.166	11.92	11.92	0	13.59	10.46	3.13
	0.434	7.97	7.97	0	15.83	14.08	1.75
	0.911	5.24	5.24	0	18.69	17.52	1.17
NTU100-400	0.037	21.86	28.82	6.96	12.72	4.14	8.58
	0.078	18.16	21.45	3.29	10.45	5.34	5.11
	0.158	15.13	15.13	0	10.98	8.11	2.87
	0.416	11.64	11.64	0	14.06	12.69	1.37
	0.879	8.27	8.27	0	16.45	15.60	0.85
JM50-100	0.038	16.22	17.57	1.36	17.23	7.08	10.15
	0.080	13.23	13.23	0	16.67	11.19	5.48
	0.169	10.51	10.51	0	15.88	12.66	3.22
	0.440	7.32	7.32	0	17.08	15.28	1.80
	0.917	5.20	5.20	0	19.41	18.26	1.15
JM50-200	0.037	17.79	20.04	2.25	14.89	4.90	9.99
	0.079	16.07	16.07	0	13.65	8.68	4.97
	0.165	13.26	13.26	0	11.89	8.95	2.95
	0.434	10.38	10.38	0	13.90	12.49	1.41
	0.920	7.78	7.78	0	17.24	16.44	0.80
JM50-400	0.037	18.53	22.31	3.77	13.31	3.40	9.91
	0.078	18.92	18.92	0	11.69	7.20	4.49
	0.161	14.87	14.87	0	10.98	8.17	2.82
	0.425	11.38	11.38	0	10.98	9.54	1.43
	0.904	9.37	9.37	0	14.24	13.50	0.74
AMK50-100	0.040	13.05	13.05	0	19.79	9.15	10.63
	0.084	10.37	10.37	0	18.77	12.76	6.01
	0.176	7.96	7.96	0	17.87	14.15	3.72
	0.460	5.05	5.05	0	20.16	17.96	2.20
	0.948	3.25	3.25	0	23.25	21.72	1.53
AMK50-200	0.039	15.42	15.42	0	15.95	5.39	10.56
	0.081	12.57	12.57	0	14.64	8.68	5.96
	0.173	9.91	9.91	0	15.87	12.62	3.25
	0.452	6.96	6.96	0	17.25	15.46	1.79
	0.936	4.62	4.62	0	20.21	19.01	1.21
AMK50-400	0.039	17.48	17.48	0	13.92	4.40	9.53
	0.081	13.80	13.80	0	11.99	6.32	5.67
	0.170	11.20	11.20	0	12.59	9.35	3.24
	0.442	7.94	7.94	0	16.45	14.78	1.67
	0.926	5.20	5.20	0	16.97	15.78	1.20

Figure 6.29 Effect of matric suction on average shear modulus and average damping ratio for JM specimens.

Figure 6.30 Effect of matric suction on average shear modulus and average damping ratio for NTU specimens.

(a) Variation of average shear modulus with matric suction

(b) Variation of average damping ratio with matric suction

Figure 6.31 Variations of average shear modulus and average damping ratio with matric suction for AMK specimens.

(a) Variation of average shear modulus with matric suction

(b) Variation of average damping ratio with matric suction

Figure 6.32 Variations of average shear modulus and average damping ratio with matric suction for JM specimens.

(a) Variation of average shear modulus with matric suction

(a) Variation of average damping ratio with matric suction

Figure 6.33 Variations of average shear modulus and average damping ratio with matric suction for NTU specimens.

6.9 Degradation of Cyclic Shear Stiffness

Figure 6.34 shows a typical shear stiffness degradation curve for a cyclic strain-controlled test. The degradation of cyclic shear stiffness can be expressed by the lowering of the tips (i.e. F_t at the applied horizontal displacement) of the hysteresis loops in each subsequent cycle. From Figure 6.34, it can be seen that the maximum shear load for each cycle, F_t , decreases with the number of cycles. The degradation of shear modulus at the N^{th} cycle can be characterized using a degradation index, δ_i (Idriss et al. 1978; Tan and Vucetic 1989; Lee and Sheu 2007). With the aid of Figure 6.34, the degradation index, δ_i , can be expressed as follows:

$$\delta_i = \frac{G_N}{G_1} = \frac{(F_{tN} / \delta_h)}{(F_{t1} / \delta_h)} = \frac{F_{tN}}{F_{t1}} \tag{6.1}$$

where G_1 and F_{t1} are the shear modulus, and shear load, in the first loading cycle, respectively, G_N and F_{tN} are the shear modulus, and shear load, at the N^{th} loading cycle, respectively, and δ_h is the applied horizontal displacement.

Figure 6.34 Typical degradation cyclic loop for a strain-controlled test (JM100). It has been found empirically that for a constant strain-controlled test, δ_i follows an

approximately linear relationship with N in a log-log plot (Idriss et al. 1978; Vucetic 1988, 1990; Tan and Vucetic 1989; Lee and Sheu 2007). For a given shear strain level, δ_i can be expressed as follows:

$$\delta_i = \frac{F_{tN}}{F_{t1}} = N^{-t_\delta} \tag{6.2}$$

where t_δ is known as degradation parameter and it describes the negative slope of the relationship:

$$t_\delta = -\frac{\log \delta_i}{\log N} \tag{6.3}$$

The degradation parameter, t_δ , measures the rate of change of δ_i . A soil with a high t_δ indicates the soil is more prone to degradation and more susceptible to liquefaction at a particular shear strain. Figure 6.35 shows the determination of t_δ .

Figure 6.35 Relationship between degradation index and the number of loading cycles of soil specimen JM100.

Figure 6.36 shows the variation of t_δ with shear strain, γ_c , for the tested residual soil

specimens. It can be seen that t_δ is strongly dependent on γ_c . The values of t_δ increase with γ_c for both saturated and unsaturated residual soil specimens. Similar trend was obtained by Idriss et al. (1978); Vucetic and Dobry (1988); Tan and Vucetic (1989); and Matasović and Vucetic (1995) for saturated clay. From Figure 6.36, it can be seen that t_δ for unsaturated soil specimens was lower than that of saturated soil specimens. The difference is due to the faster rate of pore-water pressure generation in saturated soil specimens, and thus faster reduction of shear modulus with number of loading cycles, compared with unsaturated soil specimens. Figure 6.36 also shows that for saturated soil specimens, t_δ decreased with mean effective stress. For unsaturated soil specimens, t_δ was almost unaffected by the initial matric suction as suggested in Figure 6.36.

The comparison of t_δ of the residual soil specimens with t_δ of cohesive soils from other studies is shown in Figure 6.37. From Figure 6.37(a), it can be seen that for saturated soil specimens, no trend was observed between t_δ and plasticity index. However, the data largely fall into the band for plasticity index range from 11 to 58. For unsaturated soil specimens, t_δ is insensitive to the plasticity index and the initial matric suction (Figure 6.37b).

6.10 Estimation of Very Small Strain Shear Modulus

The relationship of shear modulus with shear strain is usually expressed as normalized shear modulus with respect to a very small strain shear modulus, i.e. G/G_{max} . Due to the limitation of the GCTS cyclic simple shear apparatus as discussed in Chapter 3, measurement of G_{max} was not possible. However, G_{max} can be estimated from available empirical correlations. A summary of the empirical correlations to estimate G_{max} is listed in Table 6.5. It should be noted that these empirical equations were developed for transported soils i.e., sands and clays. Empirical correlations developed from the study of transported soils may not be

valid when applied to residual soils (Wesley 2009). As such, these equations need to be developed for residual soils.

Figure 6.36 Variation of degradation parameter with shear strain.

(a) Saturated soil specimens

(b) Unsaturated soil specimens

Figure 6.37 Comparison of degradation parameter with other studies.

Table 6.5 Empirical equations for determination of G_{max}

Soil type	General form of equation	Functions			Reference
		A	$f(e), K_z, (max)$	$g(PI, OCR)$	
Cohesive	$G_{max} = Af(e)(\sigma'_m)^{\eta}$	3230	$\frac{(2.97 - e)^2}{(1 + e)}$	0.5	Hardin and Black (1968)
		445	$\frac{(4.4 - e)^2}{(1 + e)}$	0.5	Marcuson and Wahls (1972)
		4500	$\frac{(2.97 - e)^2}{(1 + e)}$	0.5	
		141	$\frac{(7.32 - e)^2}{(1 + e)}$	0.6	Kokusho et al. (1982)
	$G_{max} = Af(e)(\sigma'_v)^{\eta}$	9600	$\frac{1}{(1 + 1.2e^2)}$	0.5	Vrettos and Savidis (1999)
		5000	$\frac{1}{e^{1.5}}$	0.5	Shibuya and Tanaka (1996)
		600	$\frac{1}{e^{1.3}}$	0.5	Jamiolkowski et al. (1995)
	$G_{max} = Af(e)(\sigma'_m)^{\eta}(p_{atm})^{(1-\eta)}$	358	$\frac{1}{e^{1.21}}$	0.57	D'Elia and Lanzo (1996)
		506	$\frac{1}{e^{1.1}}$	0.42	
		3230	$\frac{(2.97 - e)^2}{(1 + e)}$	0.5	Hardin and Drnevich (1972b)
	$G_{max} = Af(e)(\sigma'_m)^{\eta}g(OCR, PI)$	1576	$\frac{(2.97 - e)^2}{(1 + e)}$	0.5	Kim and Novak (1981)
		625	$\frac{1}{(0.3 + 0.7e)^2}$	0.5	Hardin (1978)
		677	$\frac{1}{(0.3 + 0.7e)^2}$	0.487	Athanasopoulos (1981)

Cohesive	$G_{\max} = A \times g(OCR)$	47750				$OCR^{\kappa},$ $\kappa = 0.42$	Athanasopoulos (1994)
	$G_{\max} = (\sigma'_m)^{\eta} \times g(PI)$	-	-		1	$285(2PI)^{\eta}$	Zen et al. (1987)
	$G_{\max} = A(\sigma'_m)^{\eta} \times g(PI)$	280	-		0.82	$\left[\frac{1}{(PI - I_r)} \right]^{(PI - I_r)}$	Romo (1995)
	$G_{\max} = f(e) \times (\sigma'_m)^{\eta} \times g(PI)$	-	$\frac{1}{(0.4 + 0.7e)}$		1	$358(3.8PI)^{\eta}$	Kagawa (1992)
	$G_{\max} = Af(e)(\sigma'_m)^{\eta}$	7000	$\frac{(2.17 - e)^2}{(1 + e)}$		0.5	-	Hardin and Richard (1963)
Cohesionless		3300	$\frac{(2.97 - e)^2}{(1 + e)}$		0.5	-	
		9000	$\frac{(2.17 - e)^2}{(1 + e)}$		0.38	-	Iwasaki et al. (1978)
		8400	$\frac{(2.17 - e)^2}{(1 + e)}$		0.5	-	Kokusho (1980)
		7000	$\frac{(2.17 - e)^2}{(1 + e)}$		0.5	-	Yu and Richart (1984)
	$G_{\max} = AK_{2,(\max)}(\sigma'_m)^{\eta}$	1000	$K_{2,(\max)} = 0.6(D_r) + 16.3,$ $D_r \text{ in } \%$		0.5	-	Seed and Idriss (1970)

Notes: $I_r = \frac{(LL - w)}{PI}$, D_r = relative density.

Published literatures have shown that G_{max} is a function of void ratio, mean effective stress, overconsolidation ratio, and plasticity index. Over the years, a number of equations have been suggested for G_{max} which are summarized in Table 6.5. Almost all the equations can be derived from the following generic form:

$$G_{max} = Af(e)(\sigma'_m)^\eta g(OCR, PI) \quad (6.4)$$

The classic equation suggested by Hardin and Black (1968) can be obtained by substituting $f(e) = \frac{(B-e)^2}{1+e}$, and $g(OCR, PI) = (OCR)^\kappa$ into Eq. (6.4):

$$G_{max} = A \times \frac{(B-e)^2}{1+e} \times (\sigma'_m)^\eta \times (OCR)^\kappa \quad (6.5)$$

where A , B , η are constants, e is void ratio, σ'_m is mean effective stress, OCR is overconsolidation pressure, and κ is a function of plasticity index, PI . For $OCR=1$, Equation 6.5 is reduced to as follows:

$$G_{max} = A \times \frac{(B-e)^2}{1+e} \times (\sigma'_m)^\eta \quad (6.6)$$

In order to make Equation 6.6 dimensionally consistent, G_{max} and mean effective stress are normalized with atmospheric pressure, p_{atm} ($p_{atm} \approx 100$ kPa), which can be written as:

$$G_{max} = A \times \frac{(B-e)^2}{(1+e)} \times \left(\frac{\sigma'_m}{P_{atm}} \right)^\eta \times P_{atm} \quad (6.7)$$

In this study, the value of B was fixed at 2.97 as suggested by Hardin and Black (1968). Attempts were made to develop correlations between parameters A and η in Equation 6.7 and basic soil properties for residual soils.

A total of 71 residual soils from Borden et al. (1996) were used to develop the correlation for G_{max} (Table 6.6). The soil properties consisted of water content, w (from 12.6% to 50.9%); void ratio, e (from 0.49 to 1.78); degree of saturation, S (from 39.1% to 98.7%); liquid limit, LL (from 0 to 92%); plastic limit, PL (from 0 to 61%); plasticity index, PI (from 0 to 31%); fines content, P_{200} (from 15.3% to 94.2%); and mean effective stress, σ'_m (from 25 kPa to 100 kPa).

Briefly, the steps adopted to develop the correlation for G_{max} are as follows:

- 1) Using Equation 6.7, two fitting parameters: A and η were obtained using a minimization algorithm to determine the best matched $G_{max,(est)}$. The quantity to minimize is the sum of the squared normalized residuals, SSNR defined as follows:

$$SSNR = \sum_{i=1}^N \left(\frac{G_{max,i} - G_{max,i(est)}}{G_{max,i}} \right)^2 \quad (6.8)$$

where $G_{max,i(est)}$ is the estimated value of $G_{max,i}$ and N is the number of data.

- 2) The fitting parameters, A and η , for each calibration data set are tabulated in Table 6.6. A ranges from 135 to 490, and η ranges from 0.28 to 0.51.
- 3) Using regression analyses, correlation was made between parameters A and η and basic soil properties such as w , e , S , PI , and P_{200} . Other possible combined parameters such as, Se , S/w , e/S , e^2 , e^3 , and eP_{200} were also attempted in the correlation. Table 6.7 presents the coefficient of determination, R^2 , for the various parameters.

From Table 6.7, it can be seen that A correlated well with S/e , with a R^2 of 0.49 (i.e., the highest for the correlation between parameter A and basic soil properties). However, no correlation was found between parameter η and basic soil properties. An average value of 0.47 for η was adopted. This value is in between the reported value for sand, i.e. 0.38 (Iwasaki et al. 1978) and clay, i.e. 0.5 (Hardin and Black

Table 6.6 Data from Borden et al. (1996) and fitted parameters according to Equation 6.7

No.	w (%)	e	G _s	S (%)	LL	PL	PI	Sand (%)	P ₂₀₀ (%)	σ' _m (kPa)	G _{max} (MPa)	Fitting parameters		USCS
												A	η	
1	50.3	1.78	2.74	77.6	78	56	22	5.8	94.2	25	29.55	855.2	0.28	MH
2	36.6	1.48	2.74	67.7	78	56	22	5.8	94.2	25	29.27	550.2	0.38	
3	39.2	1.25	2.81	88	57	43	14	10.3	89.7	25	15.13	226.8	0.49	
4	40.7	1.31	2.81	87	57	43	14	10.3	89.7	25	13.69	226.3	0.49	
5	41.9	1.35	2.81	87	57	43	14	10.3	89.7	25	15.94	276.0	0.48	
6	34.8	1.42	2.76	67.5	52	46	6	15.7	84.3	25	36.17	603.8	0.37	
7	35.6	1.43	2.72	67.7	56	43	13	19.5	80.5	25	35.35	603.1	0.37	
8	38.9	1.64	2.79	66.2	92	61	31	27.7	72.3	25	32.47	964.8	0.50	
9	36.6	1.48	2.74	67.7	78	56	22	5.8	94.2	50	34.03	518.2	0.45	
10	50.9	1.48	2.86	98.7	59	45	14	8.9	91.1	50	20.11	312.8	0.48	
11	39.2	1.25	2.81	88	57	43	14	10.3	89.7	50	21.04	225.0	0.50	ML
12	40.7	1.31	2.81	87	57	43	14	10.3	89.7	50	21.50	252.6	0.49	
13	41.9	1.35	2.81	87	57	43	14	10.3	89.7	50	20.33	255.1	0.49	
14	35.6	1.43	2.72	67.7	56	43	13	19.5	80.5	50	42.72	596.4	0.45	
15	35.6	1.43	2.72	67.7	56	43	13	19.5	80.5	50	44.03	607.7	0.44	
16	33.5	1.41	2.79	66	75	46	29	22.6	77.4	50	70.53	910.0	0.39	
17	38.9	1.64	2.79	66.2	92	61	31	27.7	72.3	50	46.76	908.8	0.39	
18	36.6	1.48	2.74	67.7	78	56	22	5.8	94.2	100	40.48	450.4	0.50	
19	39.2	1.25	2.81	88	57	43	14	10.3	89.7	100	25.36	192.2	0.50	
20	40.7	1.31	2.81	87	57	43	14	10.3	89.7	100	27.51	229.8	0.50	
21	41.9	1.35	2.81	87	57	43	14	10.3	89.7	100	30.49	272.0	0.50	
22	33.1	1.43	2.85	65.9	75	48	27	16.4	83.6	100	53.23	543.3	0.50	
23	35.6	1.43	2.72	67.7	56	43	13	19.5	80.5	100	52.17	532.5	0.50	
24	38.9	1.64	2.79	66.2	92	61	31	27.7	72.3	100	62.51	928.7	0.50	
25	23.6	1.17	2.66	53.6	35	-	NP	32.9	67.1	25	29.46	365.4	0.45	
26	22.8	1.24	2.72	50.2	36	-	NP	35.0	65.0	25	30.10	406.8	0.43	
27	18.4	1.16	2.69	42.7	37	-	NP	36.2	63.8	25	35.29	418.1	0.43	
28	37.8	1.50	2.69	68.0	44	34	10	42.0	58.0	25	38.21	697.7	0.33	
29	36.8	1.42	2.69	70.0	44	34	10	42.0	58.0	25	30.19	523.3	0.39	
30	15.3	0.86	2.74	48.8	35	30	5	22.3	77.7	50	59.18	343.8	0.48	
31	19.5	0.93	2.75	57.5	33	-	NP	29.1	70.9	50	46.47	300.8	0.49	
32	26.8	1.14	2.67	63.0	34	-	NP	30.2	69.8	50	45.61	402.2	0.47	
33	23.6	1.17	2.66	53.6	35	-	NP	32.9	67.1	50	39.25	364.4	0.48	
34	22.8	1.24	2.72	50.2	36	-	NP	35.0	65.0	50	39.79	410.5	0.47	
35	37.8	1.50	2.69	68.0	44	34	10	42.0	58.0	50	47.35	729.8	0.42	
36	36.8	1.42	2.69	70.0	44	34	10	42.0	58.0	50	38.37	526.3	0.45	
37	15.3	0.86	2.74	48.8	35	30	5	22.3	77.7	100	73.07	304.4	0.50	

No.	w (%)	e	G_s	S (%)	LL	PL	PI	Sand (%)	P_{100} (%)	σ'_m (kPa)	G_{max} (MPa)	Fitting parameters		USCS
												A	η	
38	19.5	0.93	2.75	57.5	33	-	NP	29.1	70.9	100	60.29	278.8	0.50	ML
39	26.8	1.14	2.67	63.0	34	-	NP	30.2	69.8	100	61.51	391.8	0.50	
40	23.6	1.17	2.66	53.6	35	-	NP	32.9	67.1	100	50.72	338.6	0.50	
41	22.8	1.24	2.72	50.2	36	-	NP	35.0	65.0	100	54.54	406.8	0.50	
42	37.8	1.50	2.69	68.0	44	34	10	42.0	58.0	100	61.27	706.0	0.50	
43	36.8	1.42	2.69	70.0	44	34	10	42.0	58.0	100	46.37	465.3	0.50	
44	29.6	1.18	2.69	67.0	44	34	10	42.0	58.0	100	43.79	296.9	0.50	
45	26.2	1.01	2.69	69.8	-	-	NP	35.6	64.4	25	63.71	558.3	0.37	SM-ML
46	26.2	1.01	2.69	69.8	-	-	NP	35.6	64.4	25	65.55	598.3	0.40	
47	23.9	1.32	2.75	49.7	29	-	NP	46.8	53.2	25	33.38	501.1	0.41	
48	22.9	1.24	2.75	50.6	29	-	NP	46.8	53.2	25	34.52	458.5	0.42	
49	24.6	0.89	2.74	75.0	48	40	8	52.0	48.0	25	43.42	354.6	0.45	
50	30.5	1.00	2.74	84.0	48	40	8	52.0	48.0	25	43.01	401.4	0.43	
51	26.2	1.01	2.69	69.8	-	-	NP	35.6	64.4	50	76.36	543.1	0.45	
52	29.6	1.14	2.71	70.6	-	-	NP	36.0	64.0	50	68.31	589.6	0.44	
53	23.9	1.32	2.75	49.7	29	-	NP	46.8	53.2	50	42.76	498.5	0.46	
54	24.6	0.89	2.74	75.0	48	40	8	52.0	48.0	50	67.15	404.8	0.47	
55	30.5	1.00	2.74	84.0	48	40	8	52.0	48.0	50	66.69	471.3	0.46	
56	26.2	1.01	2.69	69.8	-	-	NP	35.6	64.4	100	88.85	463.5	0.50	
57	29.6	1.12	2.75	72.9	-	-	NP	43.6	56.4	100	97.62	602.7	0.50	
58	23.9	1.32	2.75	49.7	29	-	NP	46.8	53.2	100	57.17	485.4	0.50	
59	24.6	0.89	2.74	75.0	48	40	8	52.0	48.0	100	89.41	389.5	0.50	
60	30.5	1.00	2.74	84.0	48	40	8	52.0	48.0	100	85.85	441.1	0.50	
61	12.6	0.84	2.60	39.1	-	-	NP	74.3	25.7	25	63.95	460.3	0.42	SM
62	14.8	0.79	2.60	48.4	-	-	NP	69.6	30.4	50	99.35	511.4	0.45	
63	12.6	0.84	2.60	39.1	-	-	NP	74.3	25.7	50	82.78	460.9	0.46	
64	15.0	0.87	2.60	45.0	-	-	NP	70.6	29.4	50	83.57	485.6	0.46	
65	16.6	0.49	2.81	96.1	-	-	NP	80.4	19.6	50	33.73	116.3	0.51	
66	24.7	0.72	2.79	96.6	-	-	NP	84.7	15.3	50	36.23	174.1	0.50	
67	14.8	0.79	2.60	48.4	-	-	NP	69.6	30.4	100	127.90	480.4	0.50	
68	12.6	0.84	2.60	39.1	-	-	NP	74.3	25.7	100	105.87	428.2	0.50	
69	15.0	0.87	2.60	45.0	-	-	NP	70.6	29.4	100	98.05	414.6	0.50	
70	16.6	0.49	2.81	96.1	-	-	NP	80.4	19.6	100	51.91	125.5	0.50	
71	24.7	0.72	2.79	96.6	-	-	NP	84.7	15.3	100	55.86	189.3	0.50	
											Average	260	0.47	

Note: NP indicates non-plastic.

1968; Hardin and Drnevich 1972a). As parameter A is mostly affected by soil type (Hardin and Black 1968; Hardin and Drnevich 1972a), the effect of S/e could be incorporated into $f(e)$ as follows:

$$f(e) = \frac{e(2.97 - e)^2}{S(1 + e)} \tag{6.9}$$

and the proposed G_{max} estimation can be written as:

$$G_{max} = A \times \frac{e(2.97 - e)^2}{S(1 + e)} \times \left(\frac{\sigma'_m}{p_{atm}} \right)^{0.47} p_{atm} \tag{6.10}$$

Attempts were made to correlate A with soil type (i.e., PI , P_{200}). However, no correlation was found between A and PI or P_{200} (Figures 6.38 and 6.39). The values of A vary from 135 to 490, with an average value of 260. Therefore, Equation 6.10 was used with $A = 260$.

Table 6.7 Coefficient of determination, R^2 , for the various correlations between basic soil properties and fitting parameters

Parameter	A	η
w	0.06	0.05
e	0.33	0.13
S	0.20	0.02
PI	0.21	0.02
P_{200}	0.09	0.03
Se	0.06	0.04
S/w	0.32	0.08
S/e	0.49	0.19
e^2	0.36	0.15
e^3	0.39	0.17
eP_{200}	0.17	0.07

Figure 6.38 Relationship between A and PI .

Figure 6.39 Relationship between A and P_{200} .

6.11 Evaluation of the Proposed G_{max} Estimation

The performance of Equation 6.10 was examined using independent data sets from the published literature. There is very limited data on dynamic properties of residual soils. Data from Macari and Hoyos (1996) for saturated undisturbed residual soil specimens as tabulated in Table 6.8 were used for evaluation.

Figure 6.40 shows the performance of Equation 6.10. It can be seen that estimated G_{max} of the residual soil, is within $\pm 15\%$ of the measured G_{max} . Therefore, Equation 6.10 was used to estimate the G_{max} of the tested residual soil specimens in order to obtain the normalized shear modulus degradation curve for comparison with other studies.

Table 6.8 Data from Macari and Hoyos (1996) used for evaluation of Equation 6.10

Void ratio, e	LL	PL	PI	Sand (%)	P_{200} (%)	USCS	Sampling depth (m)	σ'_m (kPa)	G_{max} (MPa)
1.07	35	18	17	48	52	CL	0.0-0.6	34.5	26.0
								69	34.9
								138	48.9
								275.8	68.3
1.03	39	23	16	49	51	CL	1.2-1.8	34.5	30.0
								69	36.5
								138	51.0
								275.8	74.5
0.98	42	28	14	53	47	ML	2.1-2.4	34.5	33.6
								69	41.6
								138	54.0
								275.8	74.9
0.94	44	34	10	57	43	ML	2.4-2.7	34.5	35.3
								69	44.3
								138	59.2
								275.8	76.7

Figure 6.40 Performance of the proposed estimation.

6.12 Comparison with Other Residual Soil Studies

Figure 6.41 shows the residual soils used for comparison. It can be seen that soil samples from JM and NTU are classified as CL whereas soil sample from AMK is MH. Other residual soils which fall into coarse-grained soils group are classified as SM, SM-ML (Borden et al. 1996). Basic soil properties of residual soils and their testing parameters are summarized in Table 6.9. Comparison of the residual soils in terms of sampling depth, annual rainfall, latitude of the residual soils is presented in Table 6.10.

Figure 6.41 Plasticity chart for residual soils used for comparison.

Table 6.9 Summary of testing parameters of residual soils used for comparison

Reference	Void ratio, e	S (%)	LL	PL	PI	Sand (%)	Silt and clay (%)	USCS	σ'_m (kPa)	Test
Borden et al. (1996)	1.25-1.78	65.9-98.7	52-92	43-61	6-31	5.8-27.7	72.3-94.2	MH	25, 50, 100	Resonant column, torsional shear
	0.86-1.5	42.7-70	33-44	30-34	5-10	22.3-42	58-77.7	ML	25, 50, 100	
	0.89-1.32	49.7-84	29-48	40	8	35.6-52	48-64.4	SM-ML	25, 50, 100	
	0.49-0.87	39.1-96.6	-	-	NP	69.6-84.7	15.3-30.4	SM	25, 50, 100	
Macari and Hoyos (1996)	1.07	100	35	18	17	48	52	CL	34.5, 69, 138, 275.8	Resonant column
	1.03	100	39	23	16	49	51	CL	34.5, 69, 138, 275.8	
	0.98	100	42	28	14	53	47	ML	34.5, 69, 138, 275.8	
	0.98	100	42	28	14	53	47	ML	34.5, 69, 138, 275.8	
Ng et al. (2000)	0.52-0.81	100	48	35	13	40-45	55-60	CL	200	Local transducer, Self-boring pressuremeter
Viana da Fonseca et al. (2006)	0.50	74	-	-	-	-	-	-	46	Resonant column
	0.66	80	-	-	-	-	-	-	92	

Table 6.10 Comparison of residual soil in different locations

References	Sampling depth (m)	Annual rainfall (mm)	Latitude	Location
Borden et al. (1996)	0.9 – 5.3	1052.4	35° 54' N	Raleigh, North Carolina, USA
Macari and Hoyos (1996)	0 – 2.7	1329.4	18° 27' N	San Juan, Puerto Rico
Ng et al. (2000)	34 - 38	2382.7	22° 15' N	Hong Kong
Viana da Fonseca et al. (2006)	4 and 8	1265	41° 9' N	Porto, Portugal
This study	2.5 – 3.5	2150	1° 22' N	Singapore

Figure 6.42 shows the comparison of G/G_{max} of residual soils from North America, Caribbean, Europe and Asia. As residual soil is a product from weathering process, it seems that its engineering properties, i.e. normalized shear modulus G/G_{max} , is related to annual rainfall (Figure 6.42). The normalized shear modulus degradation curve of the residual soils reported in Borden et al. (1996), Macari and Hoyos (1996), and Viana da Fonseca et al. (2006) where annual rainfall is between 1000 mm – 1300 mm are clustered together at the upper part of the graph while the normalized shear modulus degradation curve of the residual soils reported in Ng et al. (2000) where annual rainfall is 2400 mm are clustered at the lower part of the graph.

Figure 6.42 Comparison of G/G_{max} of residual soils.

The G/G_{max} values of the tested saturated residual soil specimens are shown in Figure 6.43. It can be seen that the normalized shear modulus curves for AMK soil specimens (Figure 6.43a), JM soil specimens (Figure 6.43b), and NTU soil specimens (Figure 6.43c), tested at different mean effective stresses fall into a narrow band, suggesting that the effect of mean effective stress on normalized shear modulus is not significant. Similar trend was observed by Viana da Fonseca et al. (2006).

The average annual rainfall of Singapore is about 2150 mm, similar to that of Hong Kong's (WMO 2010). From Figure 6.43, it can be seen that the normalized shear modulus for all the tested residual soil specimens is slightly higher than those obtained by Ng et al. (2000). Note that the sampling depth in this study (i.e. 2.5m – 3.5m) is shallower than Ng et al. (2000) (i.e. 34m – 38m). For annual rainfall 1000 mm – 1300 mm, the range of normalized shear modulus degradation curve of the residual soil reported in Macari and Hoyos (1996) is higher than those of Borden et

al. (1996) and Viana da Fonseca et al. (2006). Note that the sampling depth in Macari and Hoyos (1996) (i.e. 0 – 2.7m) is shallower than those in Borden et al. (1996) (i.e. 0.9m – 5.3m), and Viana da Fonseca et al. (2006) (i.e. 4m and 8m). This suggests that sampling depth of residual soil has an effect on normalized shear modulus, in which the normalized shear modulus increases with shallower sampling depth.

Figure 6.44 shows the G/G_{max} of the tested unsaturated residual soil specimens. It can be seen that for AMK soil specimens (Figure 6.44a), JM soil specimens (Figure 6.44b), and NTU soils specimens (Figure 6.44c), the normalized shear modulus curve increases as matric suction increases. This suggests that the normalized shear modulus decays at a slower rate with shear strain for unsaturated soils than that for saturated soils.

Figure 6.45 shows the comparison of damping ratio of residual soils from North America, Caribbean, and Europe. The damping ratio of residual soils with respect to shear strain in Asia remains to be characterized, as damping ratio was not measured in Ng et al. (2000) study. From Figure 6.45, it can be seen that the damping ratio of residual soils reported in Viana da Fonseca et al. (2006) is within the range of damping ratios of the residual soils reported in Borden et al. (1996). The range of damping ratio of the residual soils reported in Macari and Hoyos (1996) is lower than those of Borden et al. (1996) and Viana da Fonseca et al. (2006). Note that the sampling depth for Macari and Hoyos (1996) (i.e. 0 – 2.7m) is shallower than Borden et al. (1996) (i.e. 0.9m – 5.3m), and Viana da Fonseca et al. (2006) (i.e. 4m and 8m). This suggests that sampling depth of residual soil has an effect on damping ratio, in which the damping ratio decreases with shallower sampling depth.

Figure 6.43 Comparison of G/G_{max} of saturated residual soil specimens with other studies.

(a) AMK soil specimens

(b) JM soil specimens

(c) NTU soil specimens

Figure 6.44 Comparison of G/G_{max} of unsaturated residual soil specimens with other studies.

Figure 6.45 Comparison of damping ratio of residual soils.

The damping ratio of the tested saturated residual soil specimens is shown in Figure 6.46. It can be seen that the damping ratio for AMK soil specimens (Figure 6.46a), JM soil specimens (Figure 6.46b), and NTU soils specimens (Figure 6.46c), decreases with increasing mean effective stress. Similar trend was observed in Macari and Hoyos (1996). However, due to data scatter, no conclusive relationship between damping ratio and mean effective stress was established by Borden et al. (1996). Nevertheless, the damping ratio of the tested residual soil specimens generally follows a defined trend in which the damping ratio increases with shear strain, and largely falls into the residual soil band from other parts of the world.

(a) AMK soil specimens

(b) JM soil specimens

(c) NTU soil specimens

Figure 6.46 Comparison damping ratio of saturated residual soil specimens with other studies.

The sampling depth of residual soil in this study is 2.5m – 3.5m, almost comparable to that of Macari and Hoyos (1996) (0 – 2.7m). From Figure 6.46, it can be seen that the damping ratios of the tested residual soil specimens are higher than those obtained by Macari and Hoyos (1996). The tested residual soil specimens are from locations with high annual rainfall (2150 mm), almost twice as high as the residual soils of Macari and Hoyos (1996) (1300 mm). This suggests that annual rainfall which affects rate and extent of weathering has an effect on damping ratio of residual soil, in which the damping ratio increases for residual soils in localities with higher annual rainfall for a given soil sampling depth. The parent rock weathered under high annual rainfall have more minerals than those weathered under low annual rainfall as shown in Figure 6.47.

Figure 6.47 Influence of annual rainfall on weathering products (Strakhov 1967).

Figure 6.48 shows the damping ratio of the tested unsaturated residual soil specimens. It can be seen that for AMK soil specimens (Figure 6.48a), JM soil specimens (Figure 6.48b), and NTU soil specimens (Figure 6.48c), the damping ratio decreases as the matric suction increases. The damping ratio of the tested unsaturated residual soil specimens generally follows a defined trend in which the damping ratio increases with shear strain, and largely falls into the residual soil band from other parts of the world.

(a) AMK soil specimens

(b) JM soil specimens

(c) NTU soil specimens

Figure 6.48 Comparison damping ratio of unsaturated residual soil specimens with other studies.

6.13 Summary

A total of 95 strain-controlled cyclic simple shear tests on 12 undisturbed and 7 reconstituted residual soil specimens under saturated and unsaturated conditions have been presented. The volumetric threshold shear strains for both saturated and unsaturated residual soil specimens have been identified. The volumetric threshold shear strains for unsaturated soils were larger than those of saturated soils. Once the volumetric threshold shear strain was exceeded, the number of loading cycles affected shear modulus and damping ratio. Both the shear modulus and damping ratio decreased with number of loading cycles, due to a permanent microstructure change. As the excess pore-water pressure generation rate was faster in saturated soil specimens than unsaturated soil specimens, the degradation parameter for saturated soil specimens was higher than that for unsaturated soil specimens. Consequently, the shear stiffness of saturated soil specimens deteriorated at a faster rate than the shear stiffness of unsaturated soil specimens. For saturated soil specimens, the effect of mean effective stress on shear modulus and damping ratio has been investigated. The results showed that shear modulus increased and damping ratio decreased with mean effective stress. Since shear modulus and damping ratio data of residual soil is scarce, the results obtained in this study provided additional data to existing literature. By modifying the GCTS simple shear apparatus, a suction-controlled system was developed, enabling the effect of matric suction on shear modulus and damping ratio to be investigated. It was found that shear modulus increased and damping ratio decreased with increasing matric suction. Using residual soil data from the literature, a correlation for residual soil's G_{max} with void ratio, mean effective stress and degree of saturation was developed and evaluated. The estimated G_{max} had a discrepancy of $\pm 15\%$ from the experimental G_{max} . The correlation was used to estimate G_{max} of the residual soil specimens, thus normalized G/G_{max} can be plotted. The normalized G/G_{max} and damping ratio of residual soil specimens were compared with other studies. The

comparison showed that the differences in G/G_{max} and damping ratio of residual soil were mainly due to different weathering condition.

Chapter 7

Conclusion and Recommendations

7.1 Conclusion

As a product of weathering, residual soils generally exist above the groundwater table and therefore are unsaturated in nature. This thesis investigated the cyclic simple shear properties of residual soils under saturated and unsaturated conditions. The cyclic simple shear tests were performed at shear strains ranging from about 0.03% to 1%, which is within the shear strain range of a number of geotechnical applications such as retaining walls, foundations, and tunnels. Unsaturated residual soil specimens at a known matric suction was prepared using the pressure plate apparatus before being transferred to the cyclic simple shear apparatus equipped with suction-controlled system to investigate its cyclic simple shear properties. The unsaturated residual soil specimens tested were in the transition zone of SWCC while the saturated soil specimens tested were in boundary effect zone of SWCC. The effects of mean effective stress, matric suction, and number of loading cycles on shear modulus and damping ratio of residual soils were also investigated.

Based on the test results, the major findings in this thesis are summarized below:

(a) Cyclic simple shear apparatus

Two commercial cyclic simple shear apparatuses; GCTS and GDS, were evaluated. Cyclic simple shear tests on a standard sand, Ottawa 20-30, using both the apparatuses revealed that there was a rocking movement on the top platen. As a result, the shear modulus below shear strain of 0.03% was much lower than those from resonant column and cyclic torsional shear tests, and thus the limiting shear strain for both GCTS and GDS cyclic simple shear apparatuses was suggested to be

0.03%. For a stiffer sand specimen tested, the rocking problem became more significant, resulting in lower shear modulus at small shear strains ranging from 0.03% to 0.1%. A correction factor $c(G)$ based on actual horizontal displacement ratio, R_δ (Equations 3.13 - 3.15) was proposed to correct the shear modulus. In addition, there was friction in the cyclic simple shear apparatuses. The friction load was accounted for shear modulus determination following the suggestion in ASTM D6528-07 (2007). A detailed correction procedure based on friction energy was described for damping ratio determination. If the friction energy was left uncorrected, the damping ratio of the soil with shear strain will not follow an orderly trend.

(b) Equilibrium time of soil specimen in pressure plate apparatus

An investigation was carried out on fine-grained reconstituted residual soil specimens and coarse-grained sand-kaolin mixture to investigate the equilibrium time of a pressure plate test and the effect of specimen height on the equilibrium time, using a high suction tensiometer. For drying curve, matric suctions ranging from 5 kPa – 400 kPa and an increment of matric suction ranging from 5 kPa – 302 kPa were used whereas for wetting curve, matric suctions ranging from 25 kPa – 200 kPa and a decrement of matric suction ranging from 75 kPa - 279 kPa were used. Using matric suction measurement from high suction tensiometer, the equilibrium time for the tested soil specimens was found to be 1 to 8 days for drying process, and 3 to 8 days for wetting process, irrespective of the specimen height. Using a combination of pressure plate test and high suction tensiometer measurement, corrected rates of gravimetric water content change of 0.15 %/day and 0.08 %/day were suggested as the criteria for “equilibrium” to take place in pressure plate test for drying and wetting processes, respectively. Applicability of the proposed criterion for drying process was examined during the preparation of unsaturated residual soil specimens for cyclic simple shear tests. The measured

matric suctions at the “equilibrium” time determined from the proposed criterion showed a maximum difference of 10% of the applied matric suctions. This confirms that the proposed equilibrium criterion can be used with reasonable confidence, in the event where high suction tensiometer is not available to measure the matric suction.

(c) Soil-water characteristic curve using one-point measurement

A simplified method to estimate the SWCC of fine-grained and coarse-grained soils was developed. Essentially, this method required one measured SWCC point and empirical correlations which involve D_{50} and a variable parameter x related to the parameters of Fredlund and Xing’s (1994) SWCC equation for coarse-grained soils and a variable parameter x related to the parameters of Fredlund and Xing (1994) SWCC equation for fine-grained soils to estimate the SWCC. Sensitivity analyses revealed that better SWCC estimation using the proposed method tends to favour the one-point SWCC measurement at matric suctions of 10 kPa and 500 kPa for coarse-grained and fine-grained soils, respectively. The proposed method has been evaluated for a total of 62 soils with 31 soils each for fine-grained and coarse-grained soils. The results showed that the proposed method is simpler and performed better than existing one-point methods. The proposed method was used to estimate the SWCC of soils used in the cyclic simple shear tests. The three zones (boundary effect, transition, and residual zones) in SWCC for the soils used were distinguished. As a result, the unsaturated soil specimens can be prepared in either transition or residual zone and thus separated from those of saturated soil specimens in boundary effect zone.

(d) Cyclic simple shear properties of residual soil specimens

A total of 95 strain-controlled cyclic simple shear tests on 12 undisturbed and 7 reconstituted residual soil specimens under saturated and unsaturated conditions

were conducted. The volumetric threshold shear strains for unsaturated soil were found to be larger than those of saturated soil. Once the volumetric threshold shear strain was exceeded, shear modulus and damping ratio are affected by number of loading cycles. Both the shear modulus and damping ratio decreased with number of loading cycles, due to a permanent microstructure change. As the excess pore-water pressure generation rate was faster in saturated soil specimens than in unsaturated soil specimens, the degradation parameter δ_i for saturated soil specimens was higher than for unsaturated soil specimens. Consequently, the shear stiffness of saturated soil specimens deteriorated at a faster rate than unsaturated soil specimens. For saturated residual soil specimens, the effect of mean effective stress on shear modulus and damping ratio was investigated. Shear moduli for saturated residual soil specimens ranged from 0.7 MPa to 26.8 MPa while the damping ratios for saturated residual soil specimens ranged from 3.8% to 26.4% under the mean effective stresses 25, 50, 100, and 200 kPa for shear strain range from 0.036% to 0.978%. The results also showed that shear modulus increased and damping ratio of saturated soils decreased with mean effective stress for a given shear strain. Since shear moduli and damping ratios of residual soils are scarce, the results obtained in this study provided additional data to existing literature. The GCTS cyclic simple shear apparatus was modified to include a suction-controlled system, thus enabling the effect of matric suction on shear modulus and damping ratio of unsaturated residual soils to be investigated. Shear moduli for unsaturated residual soil specimens ranged from 3.3 MPa to 28.8 MPa while the damping ratios for unsaturated residual soil specimens ranged from 3.4% to 21.7% under initial matric suctions of 100, 200, and 400 kPa for shear strain range from 0.037% to 0.948%. It was also found that shear modulus increased and damping ratio decreased with increasing matric suction for a given shear strain. Using residual soils data from the literature, a correlation for residual soil's G_{max} with void ratio, mean effective stress and degree of saturation (Equation 6.10) was developed and

evaluated. The estimated G_{max} had a discrepancy of $\pm 15\%$ from the experimental G_{max} obtained in the literature. The correlation was used to estimate G_{max} of the residual soil specimens, thus normalized G/G_{max} can be plotted. The normalized G/G_{max} and damping ratio of residual soil specimens were compared with other studies. The comparison showed that the differences in G/G_{max} and damping ratio of residual soil were mainly due to different weathering conditions.

7.2 Recommendations and Future Research

Comprehensive testing of residual soils under saturated and unsaturated conditions are not possible within this candidature period as testing soils under unsaturated condition are very time consuming. In order to have a more comprehensive understanding of the dynamic properties of residual soils, further research and experimental investigations in the following areas are suggested:

- (a) In the present study, dynamic properties of residual soils obtained using cyclic simple shear apparatus only covered the shear strains from about 0.03% to 1%. Other apparatuses such as bender element and resonant column can be used to bridge the gap for shear strains from 0.001% to 0.03%, such that the behaviour of residual soils from very small to large shear strain can be fully described.
- (b) Unreinforced membrane was used in the cyclic simple shear tests of this study. The improvement in cyclic simple test results using reinforced membrane was not fully investigated in this study. It is therefore recommended that the effects of reinforced membrane on the cyclic simple shear properties of unsaturated soils be investigated in future.
- (c) The very small shear strain shear modulus, G_{max} was not measured in the present study. To enable a rigorous comparison of normalized shear modulus G/G_{max} with other studies, G_{max} should be measured.

- (d) Only cyclic simple shear tests at 0.5 Hz were performed in the present study. It is recommended cyclic simple shear tests at other frequencies to be performed to investigate the effect of frequency on shear modulus and damping ratio of residual soils.
- (e) For unsaturated soil test, the present study only focused on the drying behaviour of residual soil. The wetting behaviour of residual soil should be investigated to examine the difference between drying and wetting behaviours.
- (f) Residual soils used in the present study had a relatively high air-entry and residual values. Hence, the tested unsaturated soil specimens were only in boundary effect and transition zones, as limited by the 5-bar capacity ceramic disk and 1000 kPa triaxial cell. Testing residual soil with a lower air-entry value and residual value will enable cyclic behaviour of unsaturated soil in the three distinct zones to be studied, and thus a more complete understanding of soil behaviour in the three zones.
- (g) A number of equations have been suggested for the shear modulus-shear strain and damping ratio-shear strain relationships (Hardin and Drnevich 1972b; Borden et al. 1996; Stokoe et al. 1999). These equations were not evaluated in this study and should be investigated.
- (h) Cyclic simple shear tests were only performed on a limited number of residual soil samples. More tests should be performed to obtain a more comprehensive shear modulus-shear strain and damping ratio-shear strain relationships, as residual soils exhibit large spatial variation due to variable weathering.

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Appendix A

Friction Tests for GCTS and GDS Cyclic Simple Shear Apparatuses

Figure A1 Friction test for GCTS cyclic simple shear apparatus.

(b) $\delta_{Lvd t} = 0.0101 \text{ mm}$

(b) $\delta_{Lvd t} = 0.0202 \text{ mm}$

(c) $\delta_{Lvd t} = 0.0402 \text{ mm}$

(d) $\delta_{Lvd t} = 0.1002 \text{ mm}$

(e) $\delta_{Lvd t} = 0.2002 \text{ mm}$

Figure A2 Friction test for GDS cyclic simple shear apparatus.

Appendix B

Drying Behaviour of Soil Specimens in Pressure Plate Test

Figure B3 Variations of water content, measured matrix suction, and corrected rate of gravimetric water content change with time for SKM20-1 during drying.

Figure B4 Variations of water content, measured matrix suction, and corrected rate of gravimetric water content change with time for SKM40-1 during drying.

Figure B5 Variations of water content, measured matrix suction, and corrected rate of gravimetric water content change with time for SKM80-1 during drying.

Figure B6 Variations of water content, measured matric suction, and corrected rate of gravimetric water content change with time for SKM20-2 during drying.

Figure B7 Variations of water content, measured matrix suction, and corrected rate of gravimetric water content change with time for SKM40-2 during drying.

Figure B8 Variations of water content, measured matrix suction, and corrected rate of gravimetric water content change with time for SKM80-2 during drying.

Appendix C

Wetting Behaviour of Soil Specimens in Pressure Plate Test

Figure C2 Variations of water content, measured matric suction, and corrected rate of gravimetric water content change with time for NTU80 during wetting.

Figure C5 Variations of water content, measured matrix suction, and corrected rate of gravimetric water content change with time for SKM80-1 during wetting.

Figure C7 Variations of water content, measured matric suction, and corrected rate of gravimetric water content change with time for SKM40-2 during wetting.

Figure C8 Variations of water content, measured matrix suction, and corrected rate of gravimetric water content change with time for SKM80-2 during wetting.

Appendix D

Cyclic Simple Shear Tests of Saturated Soil Specimens

Figure D1 Test results for constant height cyclic test of NTU25 at $\gamma_c = 0.041\%$.

Figure D2 Test results for constant height cyclic test of NTU25 at $\gamma_c = 0.089\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure D3 Test results for constant height cyclic test of NTU25 at $\gamma_c = 0.186\%$.

Figure D4 Test results for constant height cyclic test of NTU25 at $\gamma_c = 0.481\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure D5 Test results for constant height cyclic test of NTU25 at $\gamma_c = 0.978\%$.

Figure D6 Test results for constant height cyclic test of NTU50 at $\gamma_c = 0.042\%$.

Figure D7 Test results for constant height cyclic test of NTU50 at $\gamma_c = 0.087\%$.

Figure D8 Test results for constant height cyclic test of NTU50 at $\gamma_c = 0.183\%$.

Figure D9 Test results for constant height cyclic test of NTU50 at $\gamma_c = 0.477\%$.

Figure D10 Test results for constant height cyclic test of NTU50 at $\gamma_c = 0.974\%$.

Figure D11 Test results for constant height cyclic test of NTU100 at $\gamma_c = 0.038\%$.

Figure D12 Test results for constant height cyclic test of NTU100 at $\gamma_c = 0.083\%$.

Figure D13 Test results for constant height cyclic test of NTU100 at $\gamma_c = 0.177\%$.

Figure D14 Test results for constant height cyclic test of NTU100 at $\gamma_c = 0.464\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure D15 Test results for constant height cyclic test of NTU100 at $\gamma_c = 0.953\%$.

Figure D16 Test results for constant height cyclic test of NTU200 at $\gamma_c = 0.036\%$.

Figure D17 Test results for constant height cyclic test of NTU200 at $\gamma_c = 0.079\%$.

Figure D18 Test results for constant height cyclic test of NTU200 at $\gamma_c = 0.168\%$.

Figure D19 Test results for constant height cyclic test of NTU200 at $\gamma_c = 0.443\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure D20 Test results for constant height cyclic test of NTU200 at $\gamma_c = 0.924\%$.

Figure D21 Test results for constant height cyclic test of JM50 at $\gamma_c = 0.039\%$.

Figure D22 Test results for constant height cyclic test of JM50 at $\gamma_c = 0.084\%$.

Figure D23 Test results for constant height cyclic test of JM50 at $\gamma_c = 0.179\%$.

Figure D24 Test results for constant height cyclic test of JM50 at $\gamma_c = 0.469\%$.

Figure D25 Test results for constant height cyclic test of JM50 at $\gamma_c = 0.964\%$.

Figure D26 Test results for constant height cyclic test of JM100 at $\gamma_c = 0.083\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure D27 Test results for constant height cyclic test of JM100 at $\gamma_c = 0.177\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure D28 Test results for constant height cyclic test of JM100 at $\gamma_c = 0.462\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure D29 Test results for constant height cyclic test of JM100 at $\gamma_c = 0.951\%$.

Figure D30 Test results for constant height cyclic test of JM200 at $\gamma_c = 0.037\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure D31 Test results for constant height cyclic test of JM200 at $\gamma_c = 0.080\%$.

Figure D32 Test results for constant height cyclic test of JM200 at $\gamma_c = 0.170\%$.

Figure D33 Test results for constant height cyclic test of JM200 at $\gamma_c = 0.447\%$.

Figure D34 Test results for constant height cyclic test of JM200 at $\gamma_c = 0.923\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure D35 Test results for constant height cyclic test of AMK50 at $\gamma_c = 0.041\%$.

Figure D36 Test results for constant height cyclic test of AMK50 at $\gamma_c = 0.088\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure D37 Test results for constant height cyclic test of AMK50 at $\gamma_c = 0.183\%$.

Figure D38 Test results for constant height cyclic test of AMK50 at $\gamma_c = 0.477\%$.

Figure D39 Test results for constant height cyclic test of AMK50 at $\gamma_c = 0.974\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure D40 Test results for constant height cyclic test of AMK100 at $\gamma_c = 0.038\%$.

Figure D41 Test results for constant height cyclic test of AMK100 at $\gamma_c = 0.082\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure D42 Test results for constant height cyclic test of AMK100 at $\gamma_c = 0.176\%$.

Figure D43 Test results for constant height cyclic test of AMK100 at $\gamma_c = 0.440\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure D44 Test results for constant height cyclic test of AMK100 at $\gamma_c = 0.954\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure D45 Test results for constant height cyclic test of AMK200 at $\gamma_c = 0.039\%$.

Figure D46 Test results for constant height cyclic test of AMK200 at $\gamma_c = 0.082\%$.

Figure D47 Test results for constant height cyclic test of AMK200 at $\gamma_c = 0.173\%$.

Figure D48 Test results for constant height cyclic test of AMK200 at $\gamma_c = 0.454\%$.

Figure D49 Test results for constant height cyclic test of AMK200 at $\gamma_c = 0.940\%$.

Figure D50 Uncorrected shear modulus and damping ratio of saturated JM soil specimens.

Figure D51 Uncorrected shear modulus and damping ratio of saturated NTU soil specimens.

Appendix E

Cyclic Simple Shear Tests of Unsaturated Soil Specimens

Figure E1 Test results for constant load cyclic test of NTU100-100 at $\gamma_c = 0.039\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure E2 Test results for constant load cyclic test of NTU100-100 at $\gamma_c = 0.082\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure E3 Test results for constant load cyclic test of NTU100-100 at $\gamma_c = 0.173\%$.

Figure E4 Test results for constant load cyclic test of NTU100-100 at $\gamma_c = 0.455\%$.

Figure E5 Test results for constant load cyclic test of NTU100-100 at $\gamma_c = 0.938\%$.

Figure E6 Test results for constant load cyclic test of NTU100-200 at $\gamma_c = 0.037\%$.

Figure E7 Test results for constant load cyclic test of NTU100-200 at $\gamma_c = 0.079\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure E8 Test results for constant load cyclic test of NTU100-200 at $\gamma_c = 0.166\%$.

Figure E9 Test results for constant load cyclic test of NTU100-200 at $\gamma_c = 0.434\%$.

Figure E10 Test results for constant load cyclic test of NTU100-200 at $\gamma_c = 0.911\%$.

Figure E11 Test results for constant load cyclic test of NTU100-400 at $\gamma_c = 0.037\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure E12 Test results for constant load cyclic test of NTU100-400 at $\gamma_c = 0.078\%$.

Figure E13 Test results for constant load cyclic test of NTU100-400 at $\gamma_c = 0.158\%$.

Figure E14 Test results for constant load cyclic test of NTU100-400 at $\gamma_c = 0.416\%$.

Figure E15 Test results for constant load cyclic test of NTU100-400 at $\gamma_c = 0.879\%$.

Figure E16 Test results for constant load cyclic test of JM50-100 at $\gamma_c = 0.038\%$.

Figure E17 Test results for constant load cyclic test of JM50-100 at $\gamma_c = 0.080\%$.

Figure E18 Test results for constant load cyclic test of JM50-100 at $\gamma_c = 0.169\%$.

Figure E19 Test results for constant load cyclic test of JM50-100 at $\gamma_c = 0.440\%$.

Figure E20 Test results for constant load cyclic test of JM50-100 at $\gamma_c = 0.917\%$.

Figure E21 Test results for constant load cyclic test of JM50-200 at $\gamma_c = 0.037\%$.

Figure E22 Test results for constant load cyclic test of JM50-200 at $\gamma_c = 0.165\%$.

Figure E23 Test results for constant load cyclic test of JM50-200 at $\gamma_c = 0.434\%$.

Figure E24 Test results for constant load cyclic test of JM50-200 at $\gamma_c = 0.920\%$.

Figure E25 Test results for constant load cyclic test of JM50-400 at $\gamma_c = 0.037\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure E26 Test results for constant load cyclic test of JM50-400 at $\gamma_c = 0.078\%$.

Figure E27 Test results for constant load cyclic test of JM50-400 at $\gamma_c = 0.161\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure E28 Test results for constant load cyclic test of JM50-400 at $\gamma_c = 0.425\%$.

Figure E29 Test results for constant load cyclic test of JM50-400 at $\gamma_c = 0.904\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure E30 Test results for constant load cyclic test of AMK50-100 at $\gamma_c = 0.040\%$.

Figure E31 Test results for constant load cyclic test of AMK50-100 at $\gamma_c = 0.084\%$.

Figure E32 Test results for constant load cyclic test of AMK50-100 at $\gamma_c = 0.176\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure E33 Test results for constant load cyclic test of AMK50-100 at $\gamma_c = 0.460\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure E34 Test results for constant load cyclic test of AMK50-100 at $\gamma_c = 0.948\%$.

Figure E35 Test results for constant load cyclic test of AMK50-200 at $\gamma_c = 0.039\%$.

Figure E36 Test results for constant load cyclic test of AMK50-200 at $\gamma_c = 0.081\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure E37 Test results for constant load cyclic test of AMK50-200 at $\gamma_c = 0.173\%$.

Figure E38 Test results for constant load cyclic test of AMK50-200 at $\gamma_c = 0.452\%$.

Figure E39 Test results for constant load cyclic test of AMK50-200 at $\gamma_c = 0.936\%$.

Figure E40 Test results for constant load cyclic test of AMK50-400 at $\gamma_c = 0.039\%$.

Figure E41 Test results for constant load cyclic test of AMK50-400 at $\gamma_c = 0.081\%$.

Figure E42 Test results for constant load cyclic test of AMK50-400 at $\gamma_c = 0.170\%$.

(a) Shear displacement

(b) Shear load

(c) Top platen horizontal displacement

(d) Mid-height shear displacement

(e) Normal stress

(f) Normal displacement

(g) Pore-water pressure

(h) Cyclic loop

Figure E43 Test results for constant load cyclic test of AMK50-400 at $\gamma_c = 0.442\%$.

Figure E44 Test results for constant load cyclic test of AMK50-400 at $\gamma_c = 0.926\%$.

Figure E45 Uncorrected shear modulus and damping ratio of unsaturated JM soil specimens.

Figure E46 Uncorrected shear modulus and damping ratio of unsaturated NTU soil specimens.