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RELATIONSHIP OF STIFFNESS AND DAMPING RATIO WITH STRAIN FOR RESIDUAL SOILS

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**SCHOOL OF CIVIL & ENVIRONMENTAL
ENGINEERING**

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in fulfillment of the requirements for the degree of
Doctor of Philosophy

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Abstract

There is very little work done on characterization of soil stiffness and damping ratio with strain for residual soils in Singapore. This is because routine geotechnical design does not usually incorporate the non-linearity of soil stiffness and damping ratio is only needed for dynamic problems. However, the use of large strain ($>1\%$) soil stiffness to compute ground movement is over conservative as the mobilized strain due to construction loads and working loads is usually in the range of 0.01% to 1%. Furthermore, the demand for seismic analysis is increasing due to ground tremors felt in Singapore arising from earthquakes in Indonesia. Thus, the characterization of soil stiffness and damping ratio with respect to strain is important. To characterize the Singapore residual soils' stiffness-strain and damping ratio-strain relationships, pulse transmission tests (bender element tests and ultrasonic tests), cyclic simple-shear and triaxial compression tests for shear strain ranging from 0.0005% to 5% were performed. In addition, the effects of soil parameters such as void ratio, confining pressure, degree of saturation and loading conditions such as frequency and number of loading cycles on soil stiffness and damping ratio are investigated. Both compacted and undisturbed residual soils were tested. However, most of the work concentrated on compacted residual soils because Singapore residual soils are highly heterogeneous and soil characteristics such as void ratio, density and degree of saturation cannot be systematically studied. Anisotropy of soil was not considered in the study. A few tests on undisturbed residual soils were performed for comparison with the compacted residual soils.

The pulse transmission tests (bender element tests and ultrasonic tests) were evaluated with different materials to gain a better understanding of the technique. The study shows that the waveform, magnitude and frequency of the applied voltage affect the receiver signal for bender element tests. The travel time should be based on first arrival time. Acoustic coupling is essential to obtain reliable wave velocities for ultrasonic test. The L/D and L/λ_w ratios affect the measurement of compression and shear wave velocities in bender element tests and ultrasonic tests.

Signal processing is essential for bender element tests and ultrasonic tests in order to obtain clean and clear receiver signal. Both G_{max} and $E_{bulk,max}$ increase with confining pressure and decrease with void ratio. The very small strain damping ratio decreases with confining pressure and void ratio. The G_{max} is independent of degree of saturation while $E_{bulk,max}$ increases with degree of saturation. A model was proposed to relate the compression wave velocity to G_{max} and degree of saturation.

Cyclic simple shear tests were conducted on saturated compacted residual soils and saturated undisturbed residual soils. The cyclic simple shear tests show that shear modulus of residual soils increases as effective confining pressure and frequency of loading increase. The damping ratio of residual soils decreases as effective confining pressure increases. The effect of number of loading cycles on shear modulus of the residual soils depends on the cyclic volumetric thresholds shear strain. The damping ratio of the residual soils is independent of number of loading cycles.

Shear modulus and damping ratio relationships with strain for the saturated residual soils are developed by combining the test results of pulse transmission tests, cyclic simple shear tests and conventional triaxial compression tests. The relationships show that shear modulus of the residual soils decreases as shear strain increases while the damping ratio increases as shear strain increases. The relationship between G/G_{max} with γ/γ_{ref} for Singapore residual soils is unique and independent of the microstructures of the residual soils. Similar to shear modulus, the relationship between $D_{amp}/D_{amp@1\%}$ with γ/γ_{ref} for Singapore residual soils is also unique and independent of microstructures of the residual soils. Equations are proposed to relate G/G_{max} to γ/γ_{ref} and $D_{amp}/D_{amp@1\%}$ to γ/γ_{ref} for Singapore residual soils.

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List of Symbols

α	attenuation coefficient
α_G	strain rate shear modulus parameter
α_D	strain rate damping ratio parameter
β	material dependent parameter
χ	parameter related to degree of saturation
ε	axial strain
ε_{air}	strain experienced by air
ε_f	strain experienced by fluid phase
ε_{soil}	strain experienced by soil
$\varepsilon_{soil\ skeleton}$	strain experienced by soil skeleton
ε_{water}	strain experienced by water
Φ_{angle}	phase angle of cross power spectrum
γ	shear strain
$\dot{\gamma}$	strain rate
γ_h	hyperbolic shear strain for shear modulus
γ_h'	hyperbolic shear strain for damping ratio
γ_r	reference shear strain
γ_{ref}	shear strain corresponding to G/G_{max} of 0.7
γ_{ref}'	shear strain corresponding to $D_{amp}/D_{amp@1\%}$ of 0.7
λ	Lamé's constant
λ_w	wavelength
λ_p	wavelength of compression wave

λ_s	wavelength of shear wave
ν	Poisson's ratio
ρ	bulk density
ρ_a	density of air
ρ_{dmax}	maximum dry density
ρ_f	density of combined water and air
ρ_w	density of water
σ	compressive stress
σ_f	fluid stress
σ_{total}	total stress
σ'	effective stress
σ'_v	vertical effective stress
σ'_c	effective confining pressure
τ	shear stress
A	constant
A_r	maximum magnitude of frequency spectrum of receiver
A_t	maximum magnitude of frequency spectrum of transmitter
a	constant
a_g	particle acceleration
a_v	coefficient of compressibility
B	Skeptom's B value
b	constant, indicator of fabric change, particle rearrangement and deformation characteristics
c	constant

c_1	constant
C_s	correction for the condition of pores
CC_{IO}	cross-correlation function
D	diameter of specimen
D_{50}	grain diameter passing at 50 %
D_{amp}	damping ratio
D_{max}	maximum damping ratio
d_o	diameter of pore-air at atmospheric pressure
d_{31}	piezoelectric strain constant
E	Young's modulus
$E_{bulk\ soil}$	bulk modulus of soil
$E_{bulk, max}$	initial bulk modulus
E_{max}	initial Young's modulus
e	void ratio
F	applied force
F_{max}	maximum force
f	frequency
f_a	critical frequency
f_c	central frequency
f''	characteristics frequency
G	shear modulus
G_{max}	initial shear modulus
G_s	specific gravity
G	acceleration of gravity
g_{31}	voltage coefficient

H	weight factor
H_s	coefficient of solubility
h	thickness of bender element
J	constant
K	earth pressure coefficient
K_o	coefficient of lateral earth pressure
K_a	bulk modulus of air
K_f	combined bulk modulus of water and air phases
K_f^*	modified combined bulk modulus of water and air phases
K_f^{**}	modified combined bulk modulus of water and air phases
K_s	bulk modulus of soil skeleton
K_{sp}	bulk modulus of solid particles
K_w	bulk modulus of water
k	permeability
L	length of specimen
L_u	wave path length
l_b	cantilever length of bender element
M	constant
m_1	constant
m_2	constant
n	porosity
P_{ao}	atmospheric pressure
p	constant
p_l	constant
Q	quality factor

Q	constant
r	material dependent property
S	degree of saturation
SNR	signal to noise ratio
T	time shift between the signals
T_t	surface tension
T_z	transmission coefficient
t	travel time
t_s	thickness of centre shim
t_a	first arrival time
t_p	peak to peak travel time
t_t	trough to trough travel time
t_c	travel time based on cross correlation
u_a	pore-air pressure
u_w	pore-water pressure
V	applied voltage
V_a	volume of air
V_{as}	volume of air dissolved in water
V_l	one dimensional compression wave
V_o	voltage generated
V_p	compression wave velocity
V_{pd}	compression wave velocity of soil skeleton at dry state
V_s	shear wave velocity
V_{sd}	shear wave velocity of soil skeleton at dry state
V_w	volume of water

v	particle vibration velocity
v_a	compression wave velocity of air
v_p	maximum compression particle velocity
v_s	maximum shear particle velocity
v_w	compression wave velocity of water
W	width of bender element
w	water content
w_{opt}	optimum water content
X_f	free deflection
X	constant
Y_{11}^E	Young's modulus
$\dot{y}$	particle velocity
$\dot{y}_{max}$	maximum particle velocity
z	acoustic impedance of the transducer
z_s	acoustic impedance of the specimen

Chapter 1 Introduction

1.1 Background

Before 1970, it was observed that the stiffnesses of soils measured in laboratory tests and those back-calculated from actual ground movements were different (Cole and Burland 1972, St John 1975, Wroth 1975 and Burland 1979). The stiffnesses of soils measured under the dynamic and static conditions were different and soil stiffnesses were classified as “dynamic” soil stiffness and “static” soil stiffness, respectively. The difference for the stiffnesses is due to the non-linear behaviour of soils. Nowadays, it is recognized that the stress-strain behaviour of soil is highly non-linear and soil stiffness decays with strain level (Purzin and Burland 1996 and Atkinson 2000). The “dynamic” and “static” stiffnesses have been reconciled by the understanding of soil stiffness non-linearity. The “dynamic” stiffness is simply the elastic modulus at very small strain level while the “static” stiffness is the modulus at large strain level (Woods 1985). The non-linearity of soil stiffness is important for the design of any geotechnical structure such as foundation, retaining wall or tunnel, the amount of movement is dependent on soil stiffness which varies both with strain and loading (Mair 1993). For instance, foundation on stiff clays mobilizes stiffness at strain levels of 0.01% to 0.1% while tunnelling mobilizes stiffness at strain levels of 0.01% to 1%. In many practical cases, the use of complex models and analyses to calculate the deformations of geotechnical structures is not justified as allowance is not made for soil stiffness non-linearity. In order to improve the numerical models used for geotechnical analyses, soil stiffness non-linearity should be taken into account. Thus, characterizing the non-linearity of soil stiffness is important.

Nowadays, environmental induced vibrations such as machinery and equipment vibration and ground transmitted vibration by passing vehicle and construction activities are of utmost concern in highly urbanized cities such as Singapore. As the

living standard and demand for better operating conditions of the geotechnical structures increase, there are more considerations on the serviceability of the geotechnical structures due to the load induced by people, vehicles and construction activities. Recently, tremors were experienced by some buildings in Singapore due to earthquakes in Indonesia. The greater awareness of earthquake tremors will demand the need for some seismic analyses. Routine geotechnical design which employed static analysis alone is insufficient, dynamic analysis may also be required. In order to perform the dynamic analysis, the energy dissipation characteristic of soil represented by the damping ratio is essential. Damping ratio is also strain dependent like soil stiffness. It affects ground response and soil-structure interaction behaviour significantly during vibration (Lin et al. 1996). Therefore, the evaluation of damping ratio at various strains is needed.

There are many testing techniques available for the characterization of soil stiffnesses and damping ratio at various strain level: cross-hole test, down-hole test, cyclic triaxial test, cyclic simple shear test, resonant column test, torsional shear test, travel time method (i.e. pulse transmission method) and conventional triaxial test. The testing techniques can be divided into two categories namely, in-situ test and laboratory test. The applicability of the tests depends on the strain level at which the soil stiffness and damping ratio are measured. In general, two or more types of test are combined to obtain the stiffness-strain curve for soil (Doroudian and Vucetic 1995, 1998). Pulse transmission methods (e.g. bender element and ultrasonic tests), resonant column test and cross-hole test are used to determine soil stiffness below strain level of 0.001% while cyclic triaxial, simple shear and torsional shear tests are used to determine soil stiffness above strain level of 0.01%. Recently, new testing devices have been developed to determine the soil stiffness at various strain level using a single test (Tatsuoka et al. 1994 and Doroudian and Vucetic 1995). This is attributed to the availability of high precision transducers and introduction of local strain measurements.

In Singapore, residual soils cover two thirds of the land area. Characterizing the engineering properties of the local residual soil is essential for geotechnical design. To date, there is very little research done on the characterization of the local residual soil with respect to its soil stiffness non-linearity and damping ratio. There were only a few in-situ geophysical tests done on the local residual soils at limited locations. The determination of soil stiffness and damping ratio by laboratory tests in Singapore is more scarce. Most of the laboratory tests concentrated on the soil stiffness at large strain. There is little or no work done to characterize damping ratio of local residual soils. Thus, there is a need to better understand the stiffness and damping ratio with strain relationship for Singapore residual soils.

1.2 Objective and Scope

The objective of this study is to characterize the non-linearity of shear modulus and damping ratio with strain level for Singapore residual soils. To quantify the soil stiffnesses and damping ratio of local residual soils at different strain levels, bender element tests, ultrasonic tests, cyclic simple shear tests and conventional triaxial tests were performed. Both compacted and undisturbed residual soils were tested. However, most of the works concentrated on compacted residual soil because Singapore residual soils are highly heterogeneous. In order to quantify effects of soil characteristics such as void ratio, density and degree of saturation, compacted residual soils are preferred than undisturbed residual soils. Anisotropy of soil was not considered in the study. A few tests were performed on undisturbed residual soils for purpose of comparison with the compacted residual soils.

The scope of the works in this study includes:

i) Bender Element Test

A thorough investigation on the reliability of the bender element test was performed as this test is still controversial in some respects. The bender element tests provide

the very small strain shear modulus, G_{max} . Effects of confining pressure, void ratio and degree of saturation on G_{max} were investigated.

ii) Ultrasonic Test

A thorough investigation on the reliability of the ultrasonic test was performed as this test is relatively new for soil testing. The ultrasonic tests provide the very small strain shear modulus, G_{max} , very small strain bulk modulus, $E_{bulk, max}$ and very small strain damping ratio. Effects of confining pressure, void ratio and degree of saturation on both G_{max} and $E_{bulk, max}$ as well as damping ratio were investigated.

iii) Cyclic Simple Shear Test

The cyclic simple shear tests provide the shear modulus and damping ratio of Singapore residual soils for strain levels ranging from 0.003% to 1%. This is the typical strain range encountered in the field during the construction of geotechnical structures. The strain mobilized by the construction loads and working loads of geotechnical structures are less than 0.5% (Jardine et al. 1986, Burland 1989 and Tatsuoka and Kohata 1995). Generally, the strain mobilized by any loading on the geotechnical structures ranges from 0.01% to 1% (Mair 1993 and Hiecher 1996). Effects of loading conditions such as frequency and number of cycles of load application on shear modulus and damping ratio were also investigated.

iv) Conventional Triaxial Compression Test

The large strain shear modulus (>1%) for Singapore residual soils was determined using the conventional triaxial compression test.

The data obtained from various tests were collated and the stiffness-strain and damping ratio-strain relationships for Singapore residual soil were determined.

1.3 Organization of Thesis

This thesis consists of eight chapters. Chapter 1 gives an overview of the research, objectives and scope of the study. Chapter 2 presents a literature review on different methods of measuring stiffness and the factors affecting the stiffness and damping ratio as well as the characteristics of the Singapore residual soils. Chapter 3 describes the bender element test. Chapter 4 describes the ultrasonic test. Chapter 5 describes the cyclic simple shear and triaxial compression tests. Chapter 6 presents the results and discussions on the very small strain stiffness and the damping ratio of residual soils measured using bender element and ultrasonic tests under unconsolidated undrained condition. Chapter 7 contains the results and discussions on the small strain shear modulus and the damping ratio measured using cyclic simple shear and triaxial compression tests under consolidated undrained condition. Chapter 8 summarizes the major findings of the research and presents recommendations for future research works.

Chapter 2 Literature Review

The behaviour of soil under loading is non-linear and hysteretic. Thus, the shear modulus, bulk modulus and damping ratio of soils are strain dependent. The literature review described in this chapter covers materials that are of immediate concern to this study only. It includes definition of soil stiffness and damping ratio, laboratory testing techniques and the effects of soil parameters such as void ratio, confining pressure and degree of saturation on soil stiffness and damping ratio.

2.1 Soil Stiffness

Parameters used to describe soil stiffness include shear modulus, Young's modulus and bulk modulus. Shear modulus, G , is a parameter associated with the strength of soils subjected to shear deformations. It is defined as the ratio between shear stress, τ , and shear strain, γ , as shown in Equation 2.1:

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

Young's modulus, E , and bulk modulus, E_{bulk} , are parameters that relate to the strength of the soil during axial deformation. E is defined as the ratio between the axial stress, σ , and axial strain, ε , as shown in Equation 2.2:

$$E = \frac{\sigma}{\varepsilon} \quad (2.2)$$

E can be related to G using theory of elasticity,

$$E = 2G(1 + \nu) \quad (2.3)$$

where ν is the Poisson's ratio of the soil.

E_{bulk} is the constrained stiffness of the material in resisting compressive stress as shown in Equation 2.4:

$$E_{bulk} = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)} = (\lambda + 2G) \quad (2.4)$$

where λ is the Lamé's constant

Figure 2.1 illustrates the typical stiffness-strain curve for soil. The curve is also sometimes referred as the “backbone” curve. Soil stiffness is strain dependent. Soil stiffness remains nearly constant at very small strain levels and decreases as the strain level increases. According to Lo Presti (1989) and Georgiannou et al. (1991), the strain level at which stiffness starts to decrease (i.e. threshold strain level) varies with plasticity index. The behaviour of the soil is linearly elastic when the strain level is below the threshold strain. For soils with low plasticity index, the threshold strain level is about 0.001% (Lo Presti 1989 and Georgiannou et al. 1991). However, the threshold strain level increases to about 0.01% for soils with high plasticity index. Vucetic (1994) found that the threshold strain level varies from 0.0005% to 0.005% depending on the plasticity index. In general, the threshold strain level is taken as 0.01% by most researchers (Anderson and Richart 1976; Matsui et al. 1980, Macky and Saada 1984, Kim et al. 1991 and Ng and Wang 2001). The parameters to describe the soil stiffness below the threshold strain level are initial shear modulus, G_{max} , initial Young's modulus, E_{max} , and initial bulk modulus, $E_{bulk,max}$. Soil stiffness decreases dramatically with strain once the threshold strain is exceeded and the soil exhibits elasto-visco-plastic behaviour (Stallebrass 1990). If the strain level exceeded 0.1%, the soil stiffness reduces to a very small value. This non-linear stiffness characteristic is important for understanding the deformations associated with soil-structure interaction problems

as different geotechnical structure mobilizes different strain level as illustrated in Figure 2.1.

A precise evaluation of the stress-strain properties is required due to the non-linear behaviour of soil. Generally, no single laboratory test can measure soil stiffness from strain level of 0.0001% to 1%. There is a strain limit for each testing method as shown in Figure 2.1. Thus, it is a common practice to obtain the stiffness-strain curve by combining two or more types of laboratory tests (Doroudian and Vucetic 1995, 1998). Based on Figure 2.1, pulse transmission methods (e.g. bender element and ultrasonic tests) and resonant column can be used to determine soil stiffness below strain level of 0.001% while cyclic triaxial, simple shear and torsional shear tests can be used to determine soil stiffness above strain level of 0.01%.

Recently, considerable efforts have been made to develop testing devices (cyclic triaxial and cyclic torsional devices) which can directly determine the soil stiffness in a single specimen for strain level ranging from 0.0001% to 1% (Tatsuoka et al. 1994, Doroudian and Vucetic 1995 and Stokoe et al. 1995). The improvement is mainly attributed to the availability of high precision transducers and the introduction of local strain measurement.

Both approaches using either multiple laboratory tests (i.e two or more types of laboratory tests) or single laboratory test to determine soil stiffness-strain relationship has its own merits and demerits. The details are discussed in Section 2.3.

Figure 2.1 Variation of soil stiffness with strain levels (modified from Mair 1993).

2.2 Damping Ratio

The stress-strain behaviour of soil under cyclic loading is hysteretic due to its damping characteristic as shown in Figure 2.2. The damping characteristic can be evaluated as a damping ratio, D_{amp} (Jacobsen 1930):

$$D_{amp} = \frac{\text{Area of hysteresis loop}}{2\pi \left(\text{Area of triangle } OAB \text{ and } OA'B' \right)} \tag{2.5}$$

Figure 2.2 Shear stress-strain relationship of soil.

Thiers and Seed (1968), Anderson et al. (1983), Macari and Hoyos (1996) and Yashuhara et al. (2003) found that damping ratio, D_{amp} , is also dependent on strain level (Figure 2.3). Below the threshold strain level, D_{amp} is negligible. However, D_{amp} increases as the strain level increases and exceeds the threshold strain level. This means that D_{amp} is insignificant when the soil is linearly elastic but it increases as the soil moves into the elasto-visco-plastic regions.

Figure 2.3 Variation of damping ratio with strain level.

2.3 Laboratory Testing Techniques

There are many laboratory testing techniques available for the determination of shear modulus: cyclic triaxial, cyclic simple shear, resonant column and travel time method (i.e. pulse transmission method). Figure 2.1 shows the range and applicability of the laboratory tests that can be used in the determination of soil stiffness. The more common methods employed in the laboratory to determine soil stiffness are cyclic triaxial and resonant column. Cyclic simple shear is less common as it is more difficult to perform and only a few laboratories have the apparatus. The pulse transmission method is the simplest method to determine very small strain shear modulus.

As mentioned in Section 2.1, two approaches are used to determine the soil stiffness-strain relationships namely, multiple laboratory test techniques (e.g. bender element and cyclic triaxial test) and single laboratory test technique (e.g. specially designed cyclic triaxial test with local strain measurement). The more established approach is the multiple laboratory test techniques where the very small strain soil stiffness is measured by bender element test and the small strain stiffness is measured by cyclic triaxial test. Both approaches have their own merits and demerits. The merits and demerits for both approaches are compared in Table 2.1.

The main disadvantage of Approach 1 or the so called multiple laboratory test techniques is the degree of matching between the test techniques employed at the same strain level. It is believed that considerable inconsistency can be produced due to variation in specimens, different shearing modes and strain rates. However, several researchers (Goto et al. 1991, Viggiani 1992, Georgiannou et al. 1991, Jovičić and Coop 1997 and Tatsuoka et al. 1999) have shown that the very small strain soil stiffness obtained using dynamic test (i.e. bender element test, ultrasonic test and resonant column test) is consistent with that measured using static test (i.e. specially designed cyclic simple shear test and cyclic triaxial test). This is because the variation in specimens can be eliminated or minimized by careful preparation of the specimens. In addition, the very small strain soil stiffness is independent of

strain rate at strain levels less than 0.0001%. Approach 1 was used in this study due to availability of equipment.

Table 2.1 Merits and demerits of different test approaches to determine soil stiffness

Test Approach	Merits	Demerits
Approach 1 – multiple laboratory test techniques <ul style="list-style-type: none">• Bender element and cyclic triaxial tests• Resonant column and cyclic triaxial tests• Ultrasonic and cyclic triaxial tests	<ul style="list-style-type: none">• A more common method thus the testing knowledge is more established. For instance, cyclic triaxial test is generally known to be able to measure soil stiffness for strain levels ranging from 0.01% to 1%.• The testing apparatus is more commonly available in laboratories.• The determination of very small strain soil stiffness (i.e strain <0.001%) using pulse transmission test (i.e. bender element and ultrasonic tests) is relatively easy and simple. In addition, it is a non-destructive test with high repetition. Thus, it is less time consuming.• Both tests generally can be conducted simultaneously therefore this approach is less time consuming.	<ul style="list-style-type: none">• Two specimens are sometimes used, variation in the specimen exists.• The shearing mode and strain rate for the two tests are different which may result in inconsistency of the results obtained.
Approach 2 – single laboratory test technique <ul style="list-style-type: none">• Specially designed cyclic triaxial test with local strain measurement• Specially designed cyclic simple shear test with local strain measurement	<ul style="list-style-type: none">• A single specimen is used thus the variation in the specimens is eliminated.• The same strain rate and shearing mode is employed and therefore less inconsistency in results.	<ul style="list-style-type: none">• The test apparatus is not commonly available in many laboratories.• The testing knowledge is less established than that employed in Approach 1.• It is generally more time consuming.

2.3.1 Pulse Transmission Method

There are two types of pulse transmission method namely bender element and ultrasonic testing. The underlying principle in the pulse transmission test remains the same regardless if it is a bender element test or an ultrasonic test, that is, the

measurement of the travel time of a wave being transmitted from one end of the soil specimen to the other end of the soil specimen. The differences in pulse transmission tests are mainly due to the experimental setup and the frequency of the waves used. Currently, the literature shows that bender element is the more popular of the two pulse transmission methods. In order to understand the principle of pulse transmission method, an understanding of wave propagation is needed.

2.3.1.1 Wave Propagation

Waves are generated in a continuous medium due to disturbance in the medium. The two main types of waves generated are body waves and surface waves (Das 1993). Examples of body waves are compression wave and shear wave. Examples of surface waves are Rayleigh wave and Love wave.

Compression wave (P wave), also known as primary, longitudinal and dilatational wave, involves successive compression and rarefaction of the medium through which it passes. Figure 2.4 shows deformations produced by the compression wave. Compression wave is analogous to sound wave; the motion of an individual particle is parallel to the direction of compression wave travel. Compression wave can travel through solids and fluids. It travels faster than other waves in geologic materials as geologic materials are stiffer in compression. Based on Biot's theory, there are two types of compression waves (Biot 1956a): first kind compression wave and second kind compression wave. Figure 2.5 shows the deformations caused by the two types of compression waves. The first kind compression wave, which is a faster compression wave, has a higher velocity as the soil is stiffer in this mode of deformation. It is also known as P wave, compression wave or dilatational wave of first kind. The second kind compression wave is known as the Biot wave, compression wave or dilatational wave of second kind.

Figure 2.4 Compression wave propagation (from Bolt 1988).

Figure 2.5 Deformation characteristics of first kind and second kind compressor waves.

Shear wave (S wave) also known as secondary, transverse or rotational wave, causes shear deformations as it travels through a material. Figure 2.6 shows the deformations produced by shear wave. The motion of an individual particle is perpendicular to the direction of shear wave travel. Fluid has no shear stiffness and therefore cannot sustain shear wave.

Figure 2.6 Shear wave propagation (from Bolt 1988).

The elastic wave propagation theory for both compression and shear waves can be found in any soil dynamics textbooks such as Principles of Soil Dynamics (Das 1993).

From the compression wave velocity, V_p , the bulk modulus, E_{bulk} , can be obtained using elastic wave propagation theory:

$$E_{bulk,i} = \rho V_{p,i}^2 \quad (2.6)$$

where ρ is the bulk density of soil and the subscript i refers to the direction in which the wave propagates.

From the shear wave velocity, V_s , the shear modulus, G , can be obtained using elastic wave propagation theory:

$$G_{ij} = \rho V_{ij}^2 \quad (2.7)$$

where the subscript j refers to the direction in which the shear wave is polarized.

The wave propagation theory is the basis for determining bulk modulus and shear modulus when pulse transmission techniques such as bender element and ultrasonic testing are employed. The strain level associated with bulk modulus and shear modulus using pulse transmission technique is less than 0.001% (Das 1993). Therefore, the bulk modulus and shear modulus measured using this pulse transmission technique are termed as very small strain bulk modulus, $E_{bulk, max}$, and very small strain shear modulus, G_{max} . However, the actual strain level at which the soil stiffness is measured is normally unknown and is one of the concerns in using this technique.

2.3.1.2 Bender Element

Bender element consists of two sheets of piezoelectric ceramic material such as lead zirconate titanate, barium titanate or lead titanate sandwiching a centre shim of brass, stainless steel or other ferrous nickel alloys to give it strength. It is an electromechanical transducer capable of converting mechanical energy into electrical energy and vice versa. When a driving voltage is applied to the bender element, the polarization will cause a bending displacement and the bender element acts as a signal generator. When the bender element is forced to bend, a voltage is generated and the bender element acts as a signal receiver.

The original concept of using piezoelectric elements for geotechnical applications can be traced to the work of Lawrence (1963, 1965) who used piezoelectric crystals to generate one-dimensional compression waves through sand and glass beads. Shirley (1978) was the first to use piezoceramic bender element for generating and receiving shear waves in laboratory tests. Over the decades, bender elements have found their way into different geotechnical test apparatuses. Schultheiss (1981) described the use of piezoceramic bender elements in oedometer and triaxial apparatuses. Bates (1989), Sasitharan et al. (1994), Brignoli et al. (1996), Brocanelli and Rinaldi (1998), Goto et al. (1999), Huang (2000), Pennington et al. (2001) measured shear wave velocity in triaxial specimens using piezoceramic

bender elements. Dyvik and Madshus (1985) measured small-strain stiffness, G_{max} , of soil specimens in resonant column, oedometer and direct simple shear apparatuses using bender elements. Zeng and Ni (1998), Lohani et al. (1999) and Kawaguchi et al. (2001) measured G_{max} in an oedometer using bender elements. Agarwal and Ishibashi (1991) used bender elements in a triaxial cubical box device. Blewett et al. (2000) measured shear wave velocity of saturated sand using bender elements in a large container.

Characteristics of Bender Element

A literature survey on shear wave velocity measurements using bender elements was conducted. Most researchers did not mention the type of bender element used. Amongst the different types of piezoelectric ceramics, lead zirconate titanate (PZT) is the most commonly used. Shirley and Hampton (1978) and Bates (1989) used PZT4 bender elements. Brignoli et al. (1996) used PZT5A and PZT5HN bender elements. Argawal and Ishibashi (1991) and Huang (2000) used PZT5A bender elements while Pennington et al. (2001) used PZT5B. Depending on the polarization, there are two types of bender element: x-poled and y-poled. From the energy point of view, there is no difference between x-poled and y-poled bender elements (Germano 2002). Both the x-poled and y-poled bender elements act similarly when connected in a series connection and a parallel connection, respectively. The important parameters of a transmitter bender element are the free deflection X_f and the maximum force generated F_{max} . For an x-poled cantilever bender element in series connection (Piezo Systems 2000),

$$X_f = \frac{3}{2} d_{31} \left(\frac{l_b}{h} \right)^2 \left(1 + \frac{t_s}{h} \right) V H \quad (2.8a)$$

$$F_{max} = \frac{3}{8} Y_{11}^E d_{31} \left(\frac{h}{l_b} \right) W \left(1 + \frac{t_s}{h} \right) V H \quad (2.8b)$$

For a y-poled cantilever bender element in parallel connection (Piezo Systems 2000),

$$X_f = 3d_{31} \left(\frac{l_b}{h} \right)^2 \left(1 + \frac{t_s}{h} \right) VH \quad (2.9a)$$

$$F_{\max} = \frac{3}{4} Y_{11}^E d_{31} \left(\frac{h}{l_b} \right) W \left(1 + \frac{t_s}{h} \right) VH \quad (2.9b)$$

where d_{31} is the piezoelectric strain constant, Y_{11}^E is the Young's modulus, l_b is the cantilever length of the bender element, W is the width of the bender element, h is the thickness of the bender element, t_s is thickness of the centre shim ($\ll h$), V is the applied voltage and H is an empirical weighting factor (≥ 1). For the receiver bender element, the important parameter is the voltage generated, V_0 . For an x-poled bender element in series connection (Piezo Systems 2000),

$$V_0 = \frac{3}{2} g_{31} \left(\frac{Fl_b}{Wh} \right) \left(1 - \frac{t_s^2}{h^2} \right) H \quad (2.10)$$

For a y-poled bender element in parallel connection (Piezo Systems 2000),

$$V_0 = \frac{3}{4} g_{31} \left(\frac{Fl_b}{Wh} \right) \left(1 - \frac{t_s^2}{h^2} \right) H \quad (2.11)$$

where g_{31} is the piezoelectric voltage constant and F is the applied force. As can be seen from Equations 2.8 and 2.9, the y-poled bender element in parallel connection is more suited as a transmitter as it needs a lower voltage to generate motion compared to the x-poled bender element in series connection. The motion sensitivity of the y-poled bender element in parallel connection in terms of deflection per unit of applied voltage is greater by a factor of two compared with the x-poled bender element in series connection. This is accomplished by a reduction in impedance of 4:1 (Germano 2002). The x-poled bender element in

series connection is more suitable as a receiver as it generates a higher output voltage per unit force applied to the tip of the bender element (Equations 2.9 and 2.10). However, it is more difficult to make a parallel connection and in most bender element tests, the x-poled bender element in series connection is used. Typical values of Y_{11}^E , d_{31} and g_{31} , for PZT4, PZT5A, PZT5B and PZT5H piezoceramic elements are summarized in Table 2.2.

Table 2.2 Piezoelectric constants of commercially available bender elements

Piezoelectric Constant	Bender Element Type								
	PZT4		PZT5A			PZT5B	PZT5H		
	SPK*	MEC	SPK	MEC	PSI	SPK	SPK	MEC	PSI
Elastic Modulus, $Y_{11}^E \times 10^{10} \text{ N/m}^2$	7.9	8.1	7.4	6.2	6.6	7.4	5.9	5.9	6.2
Displacement Coefficient, $d_{31} \times 10^{-12} \text{ Vm/N}$	-125	-125	-170	-177	-190	-180	-275	-264	-320
Voltage Coefficient, $g_{31} \times 10^{-3} \text{ Vm/N}$	-11.4	-10.6	-10.6	-11.1	-11.6	-10.6	-9.0	-8.9	-9.5

*SPK: SPK Electronics Co., Ltd., Taiwan

MEC: Morgan Electro Ceramics, U.K.

PSI: Piezo Systems. Inc., U.S.A.

The dimensions of the bender element are important as it affects the tip deflection of the transmitter element (Equations 2.8a and 2.9a) and the output voltage of the receiver element (Equations 2.10 and 2.11). The dimensions of the bender elements that have been used are summarized in Table 2.3. The length of bender element used varies from 6 to 32 mm and the width of the bender element used varies from 6 to 15 mm. However, tip deflection and output voltage of the bender element are only dependent on the cantilever length of the bender (l_b) and the cantilever length generally used ranges from 3 to 6 mm. The width of the bender element (W) has a significant effect on the maximum force (F_{max}) generated by the transmitter bender element (Equations 2.8b and 2.9b) and the output voltage (V_0) of the receiver bender element. A larger W increases the maximum force and reduces the output

voltage. A narrow bender element is preferred as the receiver as a higher output voltage is generated for a given force F . The thickness of bender element (h) affects tip deflection (X_f) and maximum force (F_{max}) of the transmitter bender element and output voltage (V_0) of the receiver bender element. A larger h decreases X_f and increases F_{max} of the transmitter bender element and reduces V_0 of the receiver bender element. However, the thickness of the bender element used varies in a small range (from 0.5 to 1 mm) with most of the bender elements used having a thickness of 0.5 mm (Table 2.3). When comparing the performances of the bender element tests reported in the literature, there is a need to be aware of the bender element's characteristics in relation to Equations 2.8 to 2.11. The applied voltage V in Equations 2.8 and 2.9 can partially compensate for the differences in dimensions of the bender element used to produce the same tip deflection X_f and maximum force F_{max} . A larger voltage applied to the transmitter element will generate a stronger wave to impinge on the receiver element. The stronger wave provides a larger applied force F and therefore a larger output voltage V_0 from the receiver bender element (Equations 2.10 and 2.11).

Table 2.3 Dimensions of bender elements used by other researchers

Reference	Length (mm)	Width (mm)	Thickness (mm)
Shirley and Hampton (1978)	25.4	6.4	0.5
Dyvik and Madshus (1985)	12.7	10.0	0.5
Bates (1989)	15.0	10.0	0.5
Argawal and Ishibashi (1991)	6.4	6.4	0.6
Souto et al. (1994)	12.7	10.0	0.5
Tanizawa et al. (1994)	20.0	10.0	1.0
Brignoli et al. (1996)	20.0	10.0	0.5
Arulnathan et al. (1998)	15.0	15.0	1.0
Zeng and Ni (1999)	18.0	15.0	-
	15.0	6.0	-
Huang (2000)	31.8	12.7	0.4
Diaz-Rodriguez et al. (2001)	12.7	8.0	6.0
Pennington et al. (2001)	12.0	10.0	1.0

Definition of Path Length (L_u)

In the bender element test, the shear modulus is determined using Equation 2.7. Hence, accurate measurements of the distance between the transmitter and the receiver bender elements (L) and the travel time of the wave from the transmitter to the receiver (t) are required. As the bender element is usually intruded into the soil specimen, there is ambiguity in the definition of L . Many researchers have addressed the above problem in different ways. Dyvik and Madshus (1985) had mounted a set of bender elements into a resonant column apparatus and performed the tests by using both techniques. It was found that the two sets of results agreed with each other if the shear wave was assumed to propagate from the tip of the transmitter element to the tip of the receiver element i.e L is the length of the specimen minus the length of protrusions of the bender elements or L_u . Brignoli et al. (1996) had used the same approach but varied the penetration of the bender element from 3% to 14% of the total length of soil specimen. They compared the results from bender element tests with the results of shear plate tests and resonant column tests on identical soil specimens. The results were shown to be consistent for all the tests if the shear wave is assumed to travel from the tip of the transmitter element to the tip of the receiver element. Viggiani (1995) used a different method to prove that the path length is the full length of soil specimen minus the protrusion length of the bender elements into the soil specimen. She carried out bender element tests on a set of reconstituted samples of Speswhite kaolin of different lengths in isotropic compression under different confining pressure. The specimen length is plotted against travel time for each confining pressure. It was found that for each confining pressure the test data fall on straight lines with an intercept of 6 ± 0.9 mm on axis. As the bender elements protruded about 3 mm into the soil specimens, this confirmed that the wave path length should be taken as the distance between the tips of the bender element i.e. L_u .

Determination of Travel Time

In the literature survey, it was found that there are four methods available for determination of arrival time or the travel time of the shear wave. They are first arrival time, travel time between the characteristic points, cross correlation and cross power.

First Arrival Time

The travel time is defined as the first arrival time of the receiver signal (t_a) as shown in Figure 2.7.

Figure 2.7 Definition of travel times.

Travel Time between Characteristic Points

The travel time is defined as the time difference between the characteristic peak-to-peak (t_p) or trough-to-trough (t_t) for the transmitter and receiver signals as shown in Figure 2.7.

Cross Correlation

The travel time is taken as the time shift that produces the peak cross correlation between signals recorded by the transmitter and receiver elements (T) as shown in

Figure 2.8. It is originally developed for the interpretation of cross-hole tests (Mancuso et al. 1989). In this method, the time domain data is decomposed into a group of harmonic waves of known frequency and amplitude using fast fourier transform (FFT). The cross-correlation function, CC_{IO} , is a measure of the degree of correlation of input $I(t)$ and output $O(t)$ signals. The analytical expression is

$$CC_{IO}(t) = \lim_{t \rightarrow \infty} \frac{1}{t} \int O(t)I(t+T)dt \quad (2.12)$$

where t is time and T is time shift between the signals.

Figure 2.8 Cross-correlation method for determining travel time.

The algorithm to determine the time shift (T) corresponding to the maximum value of cross correlation is given by:

$$L_I(f) = \text{FFT}(I(t))$$

$$L_O(f) = \text{FFT}(O(t)) \quad (2.13)$$

$$G_{IO}(f) = L_I^*(f) \cdot L_O(f)$$

$$CC_{IO}(t) = \text{IFFT}(G_{IO}(f))$$

where $L_I(f)$ and $L_O(f)$ are the transmitter and receiver signals in frequency domain respectively, $G_{IO}(f)$ is the cross power spectrum, $L_I^*(f)$ is the complex conjugate of $L_I(f)$, $CC_{IO}(t)$ is the cross correlation of transmitter and receiver signals and IFFT is the inverse of fast fourier transformation.

Cross Power

According to Bodare and Massarch (1984) and Mancuso et al. (1989), a group travel time of the wave for a range of frequencies between the transmitter and receiver elements can be evaluated by linear interpolation of the absolute cross power spectrum phase diagram. The phase velocity for each frequency is

$$V_s = \lambda_w f = \frac{L_u}{t} \quad (2.14)$$

where λ_w is wavelength, f is output frequency, and t is travel time. The wavelength can be expressed as follows:

$$\lambda_w = \frac{2\pi L_u}{\Phi_{angle}} \quad (2.15)$$

where Φ_{angle} is the phase angle of the cross power spectrum.

For each transmitter and receiver signals that is cross-powered, the cross power spectrum, $G_{IO}(f)$, can be obtained. In addition, the phase angle at which the peak magnitude occurs can be found. The tests can be repeated for a range of frequency and the phase angle is plotted against the frequency. Based on Equations 2.14 and 2.15,

$$\begin{aligned}
 V_s &= \lambda_w f = \frac{L_u}{t} \\
 \frac{L_u}{t} &= f \left(\frac{2\pi L_u}{\Phi_{angle}} \right) \\
 t &= \frac{\Phi_{angle}}{2\pi f}
 \end{aligned} \tag{2.16}$$

Hence, the travel time is the gradient of the graph of phase angle against frequency divided by 2π .

2.3.1.3 Ultrasonic Test

Ultrasonic test makes use of ultrasonic wave to measure the compression and the shear wave velocities of soils. Ultrasonic waves are waves having frequencies higher than 20 kHz (Kundu 2000). The ultrasonic test consists of piezoelectric crystals that are bonded to a platen. Similar to the bender element, the crystals vibrate upon voltage excitation and generate a voltage upon mechanical excitation. The platens are placed at both ends of a soil specimen. As such the piezoelectric crystals are not in intimate contact with the soil specimen and therefore, the wave signals are expected to be weaker. Normally, an acoustic coupling material is applied between the soil specimen and platens to improve the contact condition and hence, obtain a better signal. The advantage of ultrasonic test over bender element test is that ultrasonic test is less prone to short circuiting due to water penetration into piezoelectric material. This is because the insertion of bender element into hard soil material may damage the waterproofing which allows water to penetrate through it and comes into contact with the bender element directly. The ultrasonic test does not have this problem as the piezoelectric crystals are sealed. The path length is defined as the length of the specimen and the travel time is determined based on first arrival.

2.3.1.4 Sources of Error

As most laboratory tests involve soil specimens of a finite size, there is reflection and refraction of the wave at the specimen's boundaries as it travels from the transmitter platen to the receiver platen. The interference of incident and reflected waves at rigid boundaries can affect the interpretation of travel time (Arulnathan et al. 1998). In addition, near field influence has caused uncertainty in the determination of the first arrival time. Some researchers have taken first arrival as first deflection point of the receiver signal while others have taken first arrival as first reversal point of the receiver signal (refer to Figure 2.9).

Mancuso and Vinale (1988) showed that the first deflection of signal may not correspond to the arrival of the wave but to the arrival of the near field component. The wave generated is non-uniform within the near field and thus results in ambiguity in determining arrival time. Sanchez-Salinero et al. (1986) and Brignoli et al. (1996) showed experimentally the existence of near field effect masking the first arrival of the wave. Arroyo et al. (2002) found that signal distortion was not due to near-field effects alone and signal distortion still occurred beyond the Stokes' source near field. Viggiani and Atkinson (1995) concluded that (a) travel time cannot be reliably determined from the first arrival time of the receiver signal due to near field effects; (b) the most accurate travel time is given by the cross-correlation or the cross-power of the transmitter and the receiver signals; and (c) the use of characteristic points to determine travel time is a simple alternative provided that it was proven first to be consistent with those given by the more rigorous cross-correlation method. However, Santamarina and Fam (1997) pointed out that the determination of travel time using cross-correlation is only valid if both the input and output signals are of the same 'nature'. Gajo et al. (1997) found the cross-correlation method underestimated the arrival time in their bender element tests. Arulnathan et al. (1998) found that travel time based on characteristic peaks or cross-correlation between transmitter and receiver signals is incorrect because of: (a) wave interference at the boundaries; (b) phase lag or signal distortion; and (c) near-field effects. Arulnathan et al. (1998) further suggested that travel time should

be determined from the second wave arrival which is less affected by wave interference at the boundaries or the transfer functions relating electrical signals to physical waves. However, they admitted that the second wave arrival still suffers from near-field effects. Kawaguichi et al. (2001) concluded that the use of characteristic points to determine travel time is only acceptable at high frequencies.

Figure 2.9 Near field effect.

2.3.2 Cyclic Simple Shear

The use of simple shear device can be traced back to Kjellman (1951) who described the Royal Swedish Geotechnical Institute (SGI) direct shear equipment built in 1936. This apparatus used a rubber hose placed around a cylindrical specimen, surrounded by aluminium rings to prevent change in diameter. Roscoe (1953) introduced the Cambridge type simple shear apparatus which accepts square specimen. This apparatus tested cuboidal specimen with rigid sidewalls. The intention of the design is to provide uniform shear strain on the vertical and horizontal faces of the specimen. Norwegian Geotechnical Institute (NGI) developed a simple shear apparatus in 1961 which is an adaptation of SGI direct shear apparatus. In this apparatus, the cylindrical specimen is confined in a rubber membrane reinforced by spiral wires. This so called NGI-type simple shear apparatus is shown in Figure 2.10. The Cambridge type and NGI-type simple shear apparatus are the most commonly used devices in modern simple shear testing. Among the two, NGI-type apparatus is more widely used since it used a cylindrical

specimen. Many simple shear devices were developed by making modifications to the NGI-type simple shear apparatus (Casagrande and Rendon 1978, Franke et al. 1979, Ishihara and Yamazaki 1980, Boulanger et al. 1993 and Doroudian and Vucetic 1995).

Figure 2.10 NGI-type simple shear apparatus (from Bjerrum and Landva 1966).

Normally, a soil specimen with a diameter of 60 to 80 mm and height of 20 to 30 mm is used in the simple shear apparatus. The soil specimen is subjected to a vertical effective stress (σ_v') and a cyclic shear stress (τ) as shown in Figure 2.11. The horizontal load required to deform the specimen is measured by a load cell. Shear deformation is measured by a linear variable differential transformer. The shear modulus can be determined by using Equation 2.1. In addition, damping ratio can also be determined using Equation 2.5.

Figure 2.11 Basic principle of simple shear test.

2.4 Factors Affecting Soil Stiffness and Damping Ratio

The soil stiffness and damping ratio of a soil depend on many factors. The factors include strain levels, void ratio, confining pressure, degree of saturation, stress history, strain history, plasticity index, strain rate, number of loading cycles, relative density, geological age, temperature of soil, types of loading, cementation, degree of anisotropy and etc. To make the scope of this study amenable to the time frame of the candidature only some of the factors are studied. These factors are strain level, confining pressure, void ratio, degree of saturation, strain rate or frequency of loading and number of cycles. The effects of these factors on soil stiffness and damping ratio based on the findings of Hardin and Richart (1963), Hardin and Black (1966), Silver and Seed (1971), Hardin and Drnevich (1972), Stephenson (1978), Lodde and Stokoe (1982), Stokoe et al. (1985), Seed et al. (1986), Dobry and Vucetic (1987), Lo Presti (1989), Tan and Vucetic (1989), Stallebrass (1990), Georgiannou et al. (1991), Lo Presti et al. (1996), Vucetic et al. (1998), Hsu (2002), Matešić and Vucetic (2003) are summarized in Table 2.4.

Table 2.4 Factors affecting soil stiffness and damping ratio of soil

Factor	E_{bulk}	G	D_{amp}
Strain level ↑	Decrease	Decrease	Increase
Confining pressure ↑	Increase	Increase	Decrease or constant
Void ratio ↑	Decrease	Decrease	Decrease or constant
Degree of saturation ↑	Increase	Constant	Constant
Strain rate ↑	Increase	Increase	Increase or constant
Number of cycles ↑	Increase or decrease	Increase or decrease	Constant or decrease

2.4.1 Confining Pressure

Confining pressure affects the shear modulus and bulk modulus in two ways: void ratio changes due to primary consolidation and compression of air void and strengthening of physical–chemical bonds and increase in particle contact (Cascante and Santamarina 1996). In general, both shear modulus and bulk modulus increase with increasing effective confining pressure. Both initial shear modulus (G_{max}) and initial bulk modulus ($E_{bulk,max}$) are function of effective confining pressure as shown in Figure 2.12 (Hardin and Richart 1963, Hardin and Black 1966, Hardin and Drnevich 1972, Stokoe et al. 1985, Saxena et al. 1988 and Nakagawa et al. 1997). Both G_{max} and $E_{bulk,max}$ vary approximately with effective confining pressure according to a power law given by (Hardin and Richart 1963 and Cascante and Santamarina 1996):

$$G_{max} \text{ or } E_{bulk,max} = a\sigma_c'^b \quad (2.17)$$

where σ_c' is the effective confining pressure, a and b are constants.

The exponent b in Equation 2.17 is an indicator of fabric changes, particle rearrangement and deformation during loading (Cascante and Santamarina 1996). Fabric changes are greatly affected by surface roughness, degree of frustration of particle rotation, particle size distribution and inter-particle forces (Cascante and Santamarina 1996). On the other hand, particle deformation depends mainly on the inter-particle contact response, material properties and state of assembly. For a

constant fabric, the exponent b is 0.33 for spherical contacts between linear elastic materials i.e. Hertz contact (Duffy and Mindlin 1957, Deresiewicz 1974, Petrakis and Dobry 1987) and 0.50 for elastic cone to plane contact (Lee and Radok 1960) and plastic sphere contact. Goddard (1990) showed that the increase in coordination number among spherical particles due to buckling of particle chains could modify the exponent b for spherical contacts between linear elastic materials from 0.33 to 0.50. Table 2.5 shows the exponent b values for different types of soils obtained by other researchers.

Damping ratio (D_{amp}) is also a function of effective confining pressure. In general, D_{amp} decreases as effective confining pressure increases (Hardin and Drnevich 1972, Tatsuoka et al. 1978 and Vucetic et al. 1998). However, the influence of effective confining pressure on D_{amp} depends very much on the plasticity index of the soil. The effect of effective confining pressure becomes negligible if the soil has a high plasticity index.

Figure 2.12 Variation of shear and bulk modulus with confining pressure for Ottawa sand (modified from Hardin and Richart 1963).

Table 2.5 Exponent *b* value obtained by other researchers

Reference	Soil Type	<i>S</i> (%)	#Time of Measurement (mins)	<i>b</i> value	
				<i>E_{bulk, max}</i>	<i>G_{max}</i>
Wilson and Miller (1962)	Sand	100	End of primary consolidation	0.40	0.50
Hardin and Richart (1963)	Ottawa sand Quartz sand	0 -100	End of primary consolidation	0.46 – 0.80	0.46 –0.88
			48		0.04
			90		0.10
			792		0.26
Chiang and Chae (1972)	Cemented soil	-	-	-	0.50
Stokoe and Lodde (1978)	San Francisco Bay mud	100	End of primary consolidation	-	0.42 –0.90
			2		0.38
			10		0.42
Acar and El Tahir (1986)	Cemented sand	-	-	-	0.44
Chang and Woods (1987)	Cemented sand	-	-	-	0.16 – 0.70
Saxena et al. (1988)	Cemented sand	-	-	-	0 –0.58
Viggiani and Atkinson (1995)	London clay	100	End of primary consolidation	-	0.66
Cascante and Santamarina (1996)	Lead shot	0	3000	-	0.48 –0.50
			180		0.08
Baig et al. (1997)	Cemented sand	-	-	-	0 – 0.10
Jovičić and Coop (1997)	Dogs Bay sand	100	End of primary consolidation	-	0.68
Jovičić and Coop (1997)	Ham River sand	100	End of primary consolidation	-	0.60
Nakagawa et al. (1997)	Monterey sand	0-100	End of primary consolidation	0 – 0.40	0.40 –0.50
Viana da Fonseca et al. (1997)	Granitic Saprolite	100	End of primary consolidation	-	0.82
Ng and Wang (2001)	Decomposed granite	100	End of primary consolidation	-	0.70

time after application of confining pressure

2.4.2 Void Ratio

Both shear modulus and bulk modulus are function of void ratio. Based on resonant column test results, Hardin and Richart (1963) found that *G_{max}* decreased with increasing void ratio. Shear wave velocity which is related to initial shear modulus (Equation 2.7) can be related to void ratio by a linear function. Stephenson (1978) performed ultrasonic testing on compacted silty clay and found that the both *G_{max}* and *E_{bulk, max}* vary with void ratio. Both shear wave velocity and compression wave velocity which are related to *G_{max}* (Equation 2.7) and *E_{bulk, max}* (Equation 2.6), respectively, vary linearly with void ratio. The relationships between the shear wave

velocity and compression wave velocity with void ratio can be generalized in the following manner (Hardin and Richart 1963):

$$V_p \text{ or } V_s = (m_1 - m_2 e) \quad (2.18)$$

where e is the void ratio and m_1 and m_2 are constants. Figure 2.13 shows the variation of compression and shear wave velocities with void ratio obtained by Stephenson (1978).

Figure 2.13 Variation of wave velocity for silty clay with void ratio (modified from Stephenson 1978).

2.4.3 Degree of Saturation

Shear modulus is hardly affected by the degree of saturation (Miura et al. 2001). This is because water has no shear strength; and therefore water plays no part in resisting shear stress. Figure 2.14 shows the variation of shear wave velocity with Skempton's B value, which is an indicator of degree of saturation.

Figure 2.14 Variation of shear wave velocity with degree of saturation (modified from Miura et al. 2001).

Initial bulk modulus increases with degree of saturation. Kitsunezaki (1986) showed that a small amount of gas in the soil increases the compressibility and dramatically decreases the compression wave velocity (Figure 2.15). Miura et al. (2001) had investigated the relationship of compression wave velocity with Skempton's B value (Figure 2.16). Figure 2.16 shows that compression wave velocity increases with Skempton's B value. This is because the bulk modulus of fluid increases as Skempton's B value increases, thus bulk modulus of the soil increases.

Figure 2.15 Effect of gas saturation on compression wave velocity (modified from Kitsunezaki 1986).

Figure 2.16 Variation of compression wave velocity with degree of saturation (modified from Miura et al. 2001).

2.4.4 Strain Rate or Frequency of Loading

In general, bulk modulus, shear modulus and Young's modulus were found to increase with strain rate (Whitman 1957, Yong and Japp 1967, Olson and Parola 1967, Akai et al. 1975, Lacasse 1979, Isenhowe and Stokoe 1981, Dobry and Vucetic 1987, Lo Presti et al. 1996 and Matešić and Vucetic 2003). Soils behave like an extremely viscous fluid at high shearing rate and offer a larger shearing resistance. Matešić and Vucetic (2003) studied the effect of strain rate on shear modulus of sand and clay at strain rate ranging from 0.0005%/s to 0.1%/s. They showed that the shear modulus varies approximately linearly with the logarithm of strain rate (Figure 2.17). However, the increase of soil stiffness with respect to strain rate diminishes as the strain level decreases (Santucci de Magistris et al. 1999). The soil stiffness exhibits strain rate independency at strain levels less than 0.0001% (Tatsuoka et al. 1999).

There are very few studies on the effect of strain rate on damping ratio. Vucetic et al. (1998) studied the damping ratio of sand and clay for frequencies from 0.01 Hz to 0.1 Hz. They found that frequency of loading or strain rate does not have

significant effect on damping ratio. Hsu (2002) studied the effect of frequency of loading on damping ratio of sand and clay. He found that the damping ratio either varies linearly with logarithm of strain rate or increases mildly with strain rate for strain rates of 0.001%/s to 0.05%/s at frequencies from 0.01 Hz to 1 Hz. In other words, damping ratio either increases with strain rate or hardly changes with strain rate.

Figure 2.17 Variation of shear modulus with strain rate (modified from Matešić and Vucetic 2003).

2.4.5 Number of Loading Cycles

Hardin and Drnevich (1972) studied the shear modulus of sand and cohesive soil specimens for 1 to 100 cycles of loading. They found that the shear modulus decreases for cohesive soils and increases for sands as the number of loading cycle increases. Silver and Seed (1971) also found that the shear modulus of sand increases with the number of cycles of load applied. Most of the increase in shear modulus takes place in the first ten cycles after which the rate of increase is relatively small as shown in Figure 2.18. However, Vucetic and Dobry (1988) and Tan and Vucetic (1989) found that the shear modulus decreases with increasing number of loading cycles for clay. The decrease in shear modulus is not significant

for number of loading cycles less than 100. The amount of degradation depends very much on plasticity index.

Figure 2.18 Variation of shear modulus with number of cycles (from Silver and Seed 1971).

Silver and Seed (1971) showed that number of loading cycles has a significant effect on damping ratio of sand. Number of loading cycles is more important in determination of damping ratio as compared to shear modulus. However, Vucetic (1991) found that number of loading cycles does not have much effect on damping ratio of clayey soil. The level of influence depends on the strain levels, number of loading cycles applied and plasticity index.

2.5 Residual Soil

Soil parameters such as shear modulus, bulk modulus and damping ratio are important input in geotechnical analysis. There are many factors affecting the shear modulus, bulk modulus and damping ratio of soils. Most of the published literature concentrates on sands and clays. Seed and Idriss (1970), Hardin and Drnevich (1972) and Stokoe et al. (1980) had summarized the data for sands, saturated clay

and silts. There is very little research available on the relationship of shear modulus, bulk modulus and damping ratio with strain levels for residual soils. In this thesis, the shear modulus, bulk modulus and damping ratio of Singapore residual soils are determined and their relationship with strain levels are investigated in order to have a better understanding of the behaviour of Singapore residual soils.

There are two main types of residual soils in Singapore namely, residual soils derived from Bukit Timah Granite and Jurong Formation (PWD 1976). Bukit Timah Granite and its residual soil occupy the central part of Singapore while Jurong Formation which consists of sedimentary rock and its residual soil occupy most of the western and southwestern of the island (Figure 2.19). The residual soils are generally heterogeneous, highly textured and relatively stiff.

Poh et al. (1985), Yong et al. (1985) and Winn et al. (2001) conducted a number of basic index properties tests on residual soils in Singapore. Generally, residual soils derived from Bukit Timah Granite have sand contents from 10% to 90%. The liquid limit varies from 20% to 80% while the specific gravity ranges from 2.55 to 2.75. The “static” bulk modulus which is the modulus at large strain is 10 MPa to 137 MPa. The compression and shear wave velocities are in the range of 1650 m/s and 200 m/s, respectively (Veijayaratnam et al. 1993). Residual soils derived from Jurong Formation have sand contents from 10% to 50%. The liquid limit varies from 30% to 65% while the specific gravity ranges from 2.60 to 2.75. The “static” bulk modulus at large strain is 20 MPa to 250 MPa. The compression and shear wave velocities are in the range of 1600 m/s and 280 m/s, respectively (Veijayaratnam et al. 1993).

In this study, residual soils derived from both Bukit Timah Granite and Jurong Formation were used. Both Bukit Timah Granite and Jurong Formation residual soils used in this study were obtained from sites in Nanyang Technological University and in the Mandai area, respectively. The Bukit Timah Granite residual

soil was obtained from two different locations in Mandai namely, site I and site II (Figure 2.19). The two sites are 3 km apart from each other. The Bukit Timah Granite residual soil between depths of 3 m to 5 m was obtained from site I using an excavator. The soil obtained was used to prepare compacted soil specimens. The Bukit Timah Granite residual soil from site I consists of 98.5% sand and 1.5% fine-grained materials (i.e. silt and clay). According to Unified Soil Classification System (USCS), it is classified as poorly graded sand (SP). The Bukit Timah Granite residual soil obtained in site II was high quality undisturbed soil samples obtained using a Mazier sampler. Mazier sampler (triple tube core barrel) shown in Figure 2.20, is suitable for collecting undisturbed soil samples of stiff soil such as residual soil. Details of the Mazier sampler can be found in Mazier (1974). The undisturbed soil samples were obtained from depths of 8.5 m to 11.5 m. The Bukit Timah Granite residual soil from site II consists of 78% sand and 22% fine-grained materials. According to Unified Soil Classification System (USCS), it is classified as silty sand (SM).

The Jurong Formation residual soil in Nanyang Technological University was obtained from two locations namely, site III and site IV (Figure 2.19). The two sites are 50 m apart from each other. The Jurong Formation residual soil at site III was obtained by excavator. The soil obtained was used to prepare compacted soil specimens. The Jurong Formation residual soil from site III consists of 85% sand and 15% silt and clay. According to Unified Soil Classification System (USCS), it is classified as clayey sand (SC). The Jurong Formation residual soil obtained from site IV was high quality undisturbed soil specimens obtained by Mazier sampler. The undisturbed soil specimens in site IV were obtained from depth ranging from 8.5 m to 11 m. The Jurong Formation residual soil obtained from site IV consists of 88% of sand and 12% fine-grained materials (silt and clay). According to Unified Soil Classification System (USCS), it is also classified as clayey sand (SC).

Figure 2.19 Singapore geologic map (from PWD 1976).

Figure 2.20 Mazier sampler (from Mazier 1974).

Table 2.6 shows the basic index properties of the residual soils used in this study. The grain size distributions of the soils are shown in Figure 2.21. The basic index properties of the residual soils used are typical of the Bukit Timah Granite and Jurong Formation residual soils.

Table 2.6 Basic index properties of soil used in this study

Soil	Site	Properties	Values	Remarks
Bukit Timah Granite residual soil	I (depth ≈3m-5m) excavation	Specific gravity, G_s	2.56	Compacted soil specimen USCS: SP
		Liquid limit, LL (%)	-	
		Plastic limit, PL (%)	-	
		Plasticity index, PI (%)	-	
		Grain size distribution (%)		
	II (depth ≈8.5m-11.5m) Mazier sampler	- sand	98.5	Undisturbed soil specimen USCS: SM
		- silt*	0.5	
		- clay*	1	
		Specific gravity, G_s	2.45	
		Liquid limit, LL (%)	49	
		Plastic limit, PL (%)	35	
		Plasticity index, PI (%)	14	
Jurong Formation residual soil	III (depth ≈3m-5m) excavation	Grain size distribution (%)		Compacted soil specimen USCS: SC
		- sand	85	
		- silt*	5	
		- clay*	10	
		Specific gravity, G_s	2.67	
	IV (depth ≈8.5m-11m) Mazier sampler	Liquid limit, LL (%)	31	Undisturbed soil specimen USCS:SC
		Plastic limit, PL (%)	19	
		Plasticity index, PI (%)	12	
		Specific gravity, G_s	2.72	
		Liquid limit, LL (%)	30	
		Plastic limit, PL (%)	21	
		Plasticity index, PI (%)	9	
		Grain size distribution (%)		
		- sand	88	
		- silt*	9	
		- clay*	3	

* clay is classified as soil particles with grain diameter less than 5 μm and silt is classified as soil particles with grain diameter between 75 μm and 5 μm .

Figure 2.21 Grain size distributions of residual soils.

2.6 Summary

Definitions of shear modulus, bulk modulus, Young’s modulus and damping ratio are given. The pulse transmission techniques and simple shear test are reviewed. The factors affecting soil stiffness and damping ratio, strain level, confining pressure, void ratio, strain rate and number of loading cycles are reviewed. It is found that most of the research on the stiffness and damping ratio of soils concentrate on sand and clay. There is very little research done on the stiffness and damping ratio of Singapore residual soils. Singapore residual soils were formed by in-situ weathering of Bukit Timah Granite and sedimentary rock of Jurong Formation. Thus, their engineering properties differ from sand, silt and clay. Therefore, the empirical correlations, design parameters and mathematical models for conventional soils such as sand, silt and clay may not be applicable to Singapore residual soils.

Chapter 3 Bender Element Test

Bender element test is a pulse transmission test. Generally, it is used to determine G_{max} at strain levels below 0.001% using the wave propagation theory. In the bender element test, a pair of bender elements is used whereby one acts as signal transmitter and the other acts as signal receiver. A schematic of the bender element test set-up is shown in Figure 3.1. By measuring the travel time of the wave, the shear wave velocity, V_s is determined as follows:

$$V_s = \frac{L_u}{t} \quad (3.1)$$

where L_u is the length of the specimen minus the length of protrusions of the bender element and t is the time required for the wave to travel from the transmitter to the receiver. From the shear wave velocity, the very small strain shear modulus or initial shear modulus, G_{max} can be obtained using elastic wave propagation theory:

$$G_{max} = \rho V_s^2 \quad (3.2)$$

where ρ is the bulk density of the soil specimen. The G_{max} determined in this study is the very small strain shear modulus or initial shear modulus due to horizontally polarized shear wave propagated in the vertical direction.

Figure 3.1 Bender element test set up.

3.1 Experimental Set Up

The basic bender element test system consists of a pair of bender elements, function generator and a digital oscilloscope (Dyvik and Madshus 1985, Viggiani and Atkinson 1995, Arulnathan et al. 1998, Lohani et al. 1999 and Callisto and Rampello 2002). The more complete test set-up is that shown in Figure 3.1 used by Brignoli et al. (1996), Gajo et al. (1997) and Huang (2000). Most bender element test systems incorporate the signal amplifier for receiver signal but not the power amplifier for applied voltage (Shirley and Hampton 1978, Bates 1989, De Alba and Baldwin 1991, Gohl and Finn 1991, Sasitharan et al. 1994, Goto et al. 1999, Haegeman and Menge 2001 and Pennington et al. 2001). The signal amplifier for the receiver signal is not necessary if the receiver signal is strong enough. Figure 3.3 shows the experimental set-up used in this study. The bender elements used were x-poled PZT5A (Piezo PSI5A T220-A4-103X) whose values of Y_{11}^E , d_{31} and g_{31} are shown in Table 2.1.

The dimension of the bender element is 31.8 x 3.2 x 0.51 mm. The bender element was halved to produce two strips of dimensions 15.9 x 3.2 x 0.51 mm. A series

connection was employed due to ease of wiring. The bender element was coated with epoxy glue for waterproofing. The bender element was then positioned in a cylindrical sleeve with a cantilever length of 5 mm and fixed in position using epoxy glue. The sleeve was slotted into the platen (top cap or bottom pedestal) of the triaxial cell (Figure 3.2). A tight-fit was achieved between the sleeve and platen by means of O-rings. The gap between sleeve and platen was further sealed using silicon rubber. The other components of the test set-up are function generator (Hewlett Packard model 33120A), power amplifier (Piezo EPA-104) and digital oscilloscope (Hewlett Packard model 54610B). The excitation voltage produced by the function generator (20 V_{pp}) was amplified by the power amplifier and applied to the transmitter bender element in the bottom pedestal. The power amplifier, Piezo EPA-104, is capable of amplifying the function generator voltage of 20 V_{pp} to 200 V_{pp} . The wave traveling through the soil specimen was detected by the receiver bender element in the top cap. Both the applied voltage and the receiver signal were recorded on the digital oscilloscope. The oscilloscope has two recording channels and a maximum sampling rate of 20 MHz. A sampling rate of 1 MHz was used in this study. The data recorded by the digital oscilloscope was transferred to a computer for signal processing.

Figure 3.2 Set up of bender element in triaxial platens.

Figure 3.3 Bender element test set up in triaxial system.

It is generally assumed that the response of the transmitter bender element is instantaneous on application of a voltage signal. Finite element analysis conducted by Arulnathan et al. (1998) showed that there was a phase lag between the applied voltage and the response of the transmitter. To check the phase lag between the applied voltage and the response of the bender element, the transmitter and receiver bender elements were placed in contact and voltage was applied to the transmitter bender element. The applied voltage and the receiver signal shown in Figure 3.4 revealed that there is no noticeable phase lag. There is also a possibility that the wave may travel through other paths other than the soil specimen (Brignoli et al. 1996). This was checked by testing the bender element test system with no soil specimen between the platens and with the triaxial cell full of water. The receiver bender element did not response when a voltage was applied to the transmitter bender element.

Figure 3.4 Transmitter and receiver signals for bender elements in direct contact.

3.2 Test Input Parameters

Despite the popularity and simplicity of the bender element test, there is still a lack of standardization in procedure to run the bender element tests. The literature showed different types of excitation voltages of the transmitter bender element were used. As the strength of the receiver bender element signal is dependent on the applied voltage to the transmitter bender element, there is a need to investigate the effect of the applied voltage. The applied voltage has three main parameters: waveform, magnitude and frequency. Table 3.1 summarizes the waveform, magnitude and frequency of applied voltage and soil types tested by others.

To investigate the effects of the three parameters, three types of soil were used: sand (dry), kaolin (fully saturated) and Jurong Formation residual soil from site III (partially saturated). The index properties of the soils are summarized in Table 3.2. The grain size distributions are shown in Figure 3.5. The sand specimen was prepared in three layers using tamping to achieve a bulk density of 1.97 Mg/m^3 . The kaolin specimen was prepared from consolidating kaolin slurry at an initial water content of 200% under a vertical load of 200 kPa in a 30 cm diameter consolidation tank. The final water content of the kaolin is 52.8% and the bulk density is 1.64 Mg/m^3 . The Jurong Formation residual soil was compacted using Modified Proctor

method (ASTM D1557-95 1997). The bulk density of the Jurong Formation residual soil specimen is 2.20 Mg/m^3 and the water content is 11.09%. The kaolin and compacted Jurong Formation residual soil specimens were trimmed to a diameter of 50 mm using a soil lathe.

Table 3.1 Test input parameters used in the literature

Reference	Voltage Applied			Soil Type	Method of Interpretation
	Waveform	Magnitude (V_{pp})	Frequency		
Shirley and Hampton (1978)	Sine	40	4 kHz	Clay	First arrival time (deflection)
Dyvik and Madshus (1985)	Square	20	5 – 100 Hz	Clay	First arrival time (reversal)
Bates (1989)	Square	10	2 Hz	Sand	First arrival time (?)
Argawal and Ishibashi (1991)	Sine	80 – 150	13 kHz	Glass sphere	First arrival time (?)
De Alba and Baldwin (1991)	Sine	10	3.5 kHz	Sand	First arrival time (?)
Gohl and Finn (1991)	Square	20	30 Hz	Sand	First arrival time (P wave being picked before S wave, deflection)
Sasitharan et al. (1993)	Square	30	0.02	Sand	First arrival time (?)
Viggiani and Atkinson (1995)	Square Sine	20	50 Hz (square) 1 – 10 kHz (sine)	Clay	First arrival time (deflection, reversal) Characteristics points Cross correlation Cross power
Brignoli et al. (1996)	Sine	20	3 – 10 kHz	Clay Sand	First arrival time (reversal)
Gajo et al. (1997)	Sine	75	10 kHz	Sand	First arrival time (deflection)
Arulnathan et al. (1998)	Sine	20	1 – 5 kHz	Organic soil	First arrival time (?) Characteristics points Cross correlation Cross power
Brocanelli and Rinaldi (1998)	Square	20	-	Sand	First arrival time (start of first rise of significant peak)
Zeng and Ni (1998)	Square	10	-	Sand	First arrival time (deflection)
Goto et al. (1999)	Square	10	5 kHz	Sand	First peak
Lohani et al. (1999)	Square	20	50 Hz	Clay	First arrival time (reversal)
Huang (2000)	Square	20	50 Hz	Sand	First arrival time (P wave being picked before S wave, deflection)
Blewett et al. (2000)	Sine		0.2 – 10 kHz	Sand	
Diaz-Rodriguez et al. (2001)	Square	20	7 Hz	Silty clay	First peak
Kawaguchi et al. (2001)	Square Sine	20	100 Hz (square) 1 – 8 kHz	Clay	First arrival time (average of first rise and peak)
Lings and Greening (2001)	Sine	20	10 kHz	Sand	First arrival time (deflection)
Pennington et al. (2001)	Sine	20	8 – 25 kHz	Clay	First arrival time (deflection)
Yamashita and Suzuki (2001)	Sine Square	20	2 – 20 kHz	Sand	First arrival time (deflection)
Callisto and Rampello (2002)	Square Sine	20	50 Hz (square) 10 kHz (sine)	Clay	Average time of first arrival and reversal point Characteristics points

? not specified

Table 3.2 Basic index properties of soils used

Material	Properties	Values	USCS
Sand	Specific gravity, G_s	2.63	SP-SM
	Maximum density (Mg/m^3)	2.18	
	Minimum density (Mg/m^3)	1.39	
	Liquid limit, LL (%)	26	
	Plastic limit, PL (%)	23	
	Plasticity index, PI (%)	3	
	Grain size distribution (%)		
	- sand	90	
	- silt*	8	
	- clay*	2	
Jurong Formation residual soil	Specific gravity, G_s	2.67	SC
	Liquid limit, LL (%)	31	
	Plastic limit, PL (%)	19	
	Plasticity index, PI (%)	12	
	Grain size distribution (%)		
	- sand	85	
	- silt*	5	
	- clay*	10	
Kaolin	Specific gravity, G_s	2.65	SM
	Liquid limit, LL (%)	74	
	Plastic limit, PL (%)	46	
	Plasticity index, PI (%)	28	
	Grain size distribution (%)		
	- sand	88	
	- silt*	3	
	- clay*	9	

* clay is classified as soil particles with grain diameter less than 5 μm and silt is classified as soil particles with grain diameter between 75 μm and 5 μm .

Figure 3.5 Grain size distributions of soils used.

3.2.1 Waveform

Based on Table 3.1, two types of waveform are commonly used: square and sinusoidal. Square wave comprises a large number of frequency components within the original signal. During wave passage through the soil specimen, the square wave will be subjected to more distortion compared to a sinusoidal wave which comprises only one frequency (Blewett et al. 2000). Jovičić et al. (1996) found difficulty in determining the arrival time for a square wave signal due to the signal distortion. There was uncertainty whether the first deflection, the reversal point or some other point was the actual arrival time. Viggiani and Atkinson (1995) recommended the use of sinusoidal wave as it is less distorted as it comprises mainly one frequency.

In this study, the use of both square wave and sinusoidal wave in the bender element tests was assessed. Figure 3.6 shows the processed receiver signal for square wave and sinusoidal wave of the three soil specimens. It can be seen that the square wave causes greater ambiguity in arrival time determination than the sinusoidal wave. Firstly, the receiver signals for square wave do not resemble the original transmitter signals. The sharp and immediate rise in the signal as well as the flat portion in the signal was lost. Secondly, the receiver signals for the square wave have more distortion at the beginning of the receiver signal. However, the first arrival time for the sinusoidal wave will correspond to one of the arrival times for square wave. This shows that both square wave and sinusoidal wave are able to give a similar arrival time. Amongst the three soil specimens, kaolin has the clearest receiver signal due to its uniformity in terms of particle size and distribution, and high damping characteristics. A number of the frequency components in the square wave signal have been damped out before it reaches the receiver bender element. Sinusoidal wave was found to be a better choice than square wave as the distortion of the receiver signal at the beginning is greatly reduced. The subsequent tests described in this study used sinusoidal wave as the input signal.

A comparison of the various methods of determining travel times for the soil specimens using bender element test and ultrasonic test is shown in Table 3.3 where t_a is the travel time based on first deflection, t_p is the travel time based on characteristic peak, t_t is the travel time based on characteristic trough and t_c is the travel time based on the cross-correlation method. The ultrasonic test was conducted using the ultrasonic test system by GCTS (2002) which will be discussed in Chapter 4. In the ultrasonic test, the input signal frequency was 76 kHz much higher than those usually used in bender element test. Table 3.3 shows that the travel times for both the bender element test and the ultrasonic test are consistent if the first deflection of the receiver signal was used for the bender element test.

Note: t_1 = first deflection, t_2 = first reversal

Figure 3.6 Transmitter and receiver signals for different types of material.

Table 3.3 Comparison of travel times for bender element test and ultrasonic system

Material	Travel Time (μ s)	
	Bender Element	Ultrasonic *
Sand	$t_a = 209$ $t_p = 339$ $t_t = 326$ $t_c = 300$	$t_a = 203$
Jurong Formation residual soil #	$t_a = 236$ $t_p = 401$ $t_t = 394$ $t_c = 375$	$t_a = 226$
Kaolin	$t_a = 266$ $t_p = 335$ $t_t = 330$ $t_c = 295$	$t_a = 263$

Jurong Formation residual soil specimen of density 2.06 Mg/m^3 and water content of 10.25%

* t_a was calculated from the velocity obtained from ultrasonic test as the sample length was different from that in the bender element test.

3.2.2 Applied Voltage

The magnitude of applied voltage to the transmitter bender element does not affect the shape or appearance of the signal but it increases the strength of the signal and its relative amplitude to noise. In order to study the effect of the magnitude of applied voltage, a term called signal to noise ratio, *SNR* is defined (Carlson 1986):

$$SNR = 10 \log \frac{\text{Signal power}}{\text{Noise power}} \quad (3.3)$$

The signal and noise power can be obtained from the power spectrum or frequency spectrum of the signal. Bendat and Piersol (1993) recommended that signals with *SNR* of less than 6 dB be discarded. The effect of magnitude of applied voltage on signal quality was examined by applying four different voltages from 5 to 70 V_{pp} to the soil specimens. Figure 3.7 shows the frequency spectra of receiver signals for the different applied voltages. As the applied voltage increases, the intensity or power spectrum magnitude of the receiver signal increases. At 5 V_{pp} , no signal was detected by the receiver bender element for the sand specimen. The effect of *SNR*

on the receiver signal is shown in Figure 3.8. Above a *SNR* of 6 dB, the distortion of the receiver signal at the beginning is lesser and it becomes easier to pick the first deflection of the receiver signal. To obtain a receiver signal with *SNR* of 6 dB or better, the applied voltage should be 20 V_{pp}, 40 V_{pp} and 70 V_{pp} for the kaolin, Jurong Formation residual soil and sand specimens, respectively. The higher input voltage generated a stronger shear wave relative to near-field effects and minimized the distortion of the receiver signal. Therefore *SNR* should be used as a criterion when selecting the appropriate voltage to be applied to the transmitter bender element in any bender element test system. In this study, a supply voltage of 40 V_{pp} is used for the subsequent Jurong Formation residual soil in order to obtain a high signal to noise ratio and a less complex receiver signal.

Figure 3.7 Frequency spectra for receiver signal of different materials.

(a) Sand

(b) Jurong Formation residual soil

(c) Kaolin

Figure 3.8 Effect of SNR on receiver signals.

3.2.3 Frequency of Applied Voltage

Input signal frequencies from 0.005 to 20 kHz have been used (Table 3.1). Wave velocity V , frequency f and wavelength λ_w are related (Equation 2.29). For a given wave velocity, wavelength decreases as frequency increases. There are experimental evidences suggesting that near-field components affects the shape of the receiver signal and causes uncertainty in determining the arrival of the shear wave in bender element tests (Viggiani and Atkinson 1995, Jovičić et al. 1996 and Brignoli et al. 1996). The near-field effects are quantified in terms of the ratio of

wave path length and wavelength, L_{tt}/λ_ω (Sanchez-Salinero et al. 1986). The use of high frequency sinusoidal pulse was recommended to reduce the near-field effect as it ensures the separation of the near field coupled compression and shear waves (Jovičić et al. 1996 and Brignoli et al. 1996). Theoretical analyses performed by Sanchez-Salinero et al. (1986) showed that near-field effects are significant when L_{tt}/λ_ω is less than one. Jovičić et al. (1996) and Brignoli et al. (1996) found that the near-field effect decreases as L_{tt}/λ_ω increases. Gajo et al. (1997) found the near-field components negligible in their experiments when L_{tt}/λ_ω is between 6 and 9. However, Arulnathan et al. (1998) found that the receiver signal deteriorates as L_{tt}/λ_ω increases and attributed the deterioration to the effect of the ratio of wavelength and cantilever length of the bender element (λ_ω/l_b). Arroyo et al. (2002) suggested that L_{tt}/λ_ω greater than 1.6 may be used to eliminate near-field effects in the frequency domain but is inappropriate in the time domain where it is signal dependent.

Bender element tests were performed on the soil specimens at frequencies of 0.5, 2, 4, 8 and 16 kHz. A typical set of signals for the Jurong Formation residual soil specimen is shown in Figure 3.9. The near-field components observed before the first prominent peak reduces as the input frequency increases. The near-field components are absent at an L_{tt}/λ_ω ratio of 4.2. The receiver signals for all three soil types at various L_{tt}/λ_ω ratios are shown in Figure 3.10. A lower bound of L_{tt}/λ_ω can be obtained from the recommendations of ASTM D2845 (1997). ASTM D2845 suggested that for reliable measurement of wave velocity, the specimen length should be at least 10 times the average grain diameter and the wavelength should be at least three times the average grain diameter. These two requirements suggest an L_{tt}/λ_ω ratio of 3.33 and some support for this can be seen in Figures 3.9 and 3.10. The *SNR* of the receiver signals in Figures 3.9 and 3.10 were greater than 6 dB.

Figure 3.9 Effect of applied frequency on receiver signals.

Figure 3.10 Variation of shear wave velocity with L_{tt}/λ_w ratio.

3.3 Estimation of Strain Level for Bender Element Test

Generally in bender element test, no indication is given of the strain levels at which the stiffness are being measured. It is widely accepted that the shear modulus derived from measurement of the wave velocity corresponds to strain levels of less than 0.001% (Das 1993). In this study, way had been established to estimate the strain level for bender element test.

In bender element tests, the strain levels associated with the shear wave velocity cannot be measured directly as the deflection of the bender element depends on the coupling between the soil and transmitter bender element. However, the receiver bender element signal when the bender elements are in direct contact (Figure 3.4) can be used to provide an estimate of the strain level. For the bender elements used in this study, Equation 2.8 reduces to (Piezo Systems 2000):

$$X_f = \frac{3l_b^2 V_o d_{31}}{2h^2} \quad (3.4)$$

where l_b is the cantilever length of the bender element, h is the thickness of bender element, V_o is the output voltage for the receiver signal when the bender elements are in direct contact and d_{31} is the piezoelectric strain constant of bender element. By assuming that the element tip deflections of the bender element given by Equation 3.4 are equivalent to the soil particle vibration, the particle velocity $\dot{y}_{\max}$ may be estimated from the time history of X_f i.e.

$$\dot{y}(t) = \dot{X}_f(t) = \frac{3l_b^2 \dot{V}_o(t) d_{31}}{2h^2} \quad (3.5)$$

With $\dot{y}(t)$, the shear strain γ can be estimated using (White 1965):

$$\gamma = \frac{\dot{y}_{\max}}{V_s} \quad (3.6)$$

where $\dot{y}_{\max}$ is the maximum particle velocity from Equation 3.5 and V_s is the shear wave velocity.

Using this method, the shear strain levels of the bender element tests conducted were found to be in the range of 1 to 5 x 10⁻⁴%. Dyvik and Madshus (1985), Pennington (1999) and Haegeman and Menge (2001) estimated maximum shear strains of the order of 10⁻³%, 10⁻⁴% and 10⁻⁴%, respectively, for their bender element tests.

Chapter 4 Ultrasonic Test

Ultrasonic test, another pulse transmission test, is not commonly used to determine the shear modulus and bulk modulus of soil as soil has high damping characteristics. Furthermore, the piezoelectric crystals are not in intimate contact with the soil specimen therefore, the wave signals are expected to be weaker. However, the problem is solved by introducing acoustic couplant in between the platen and soil specimen. In addition, signal processing tools can be employed to remove the noise, to produce a “clean” receiver signal.

In this test, compression and shear wave velocities are determined as the length of the soil specimen (L) divided by the first arrival time of the waves (t).

$$V_p \text{ or } V_s = \frac{L}{t} \quad (4.1)$$

where V_p is the compression wave velocity and V_s is the shear wave velocity.

The very small strain bulk modulus (i.e. initial bulk modulus), $E_{bulk, max}$, and very small strain shear modulus (i.e. initial shear modulus), G_{max} can be determined by the following equations:

$$E_{bulk, max} = \rho V_p^2 \quad (4.2)$$

$$G_{max} = \rho V_s^2 \quad (4.3)$$

where ρ is the bulk density of the soil specimen. The $E_{bulk, max}$ determined in this study is the very small strain bulk modulus due to a compression wave propagated in the vertical direction. The G_{max} determined in this study is the very small strain

shear modulus due to horizontally polarized shear wave propagated in the vertical direction.

4.1 Experimental Set Up

There are three modes of operation in an ultrasonic test system (Kundu 2000): pulse-echo, through-transmission and pitch-catch. In the pulse-echo mode, a single transducer is used for both pulse generation and pulse detection. In the through-transmission mode, two transducers are used: one for pulse generation and one for pulse detection. In the pitch-catch mode, the two transducers are on the same side of the specimen. The through-transmission mode is more suitable for measuring wave velocities and attenuations of geomaterials (Krautkrämer and Krautkrämer 1990). The ultrasonic test system used in this study is from GCTS (2001) and operates in the through-transmission mode. The ultrasonic test system is meant to be used within a triaxial apparatus. Figure 4.1 shows the schematic of the ultrasonic test system. The ultrasonic test system consists of a pair of 70 mm diameter platens containing the transducers, a pulse generator and receiver (DPR 300 pulser and receiver) and a high-speed data acquisition system consisting of the ultrasonic interface unit with compression and shear waves switching circuit and a personal computer. A huge advantage of this ultrasonic test system is that the raw signal can be stored and retrieved for further signal processing with other signal processing software, thereby enabling a better understanding and interpretation of the signal.

The compression and shear wave transducers consisting of ceramic piezoelectric crystals are bonded to the inside face of the stainless steel platen. The arrangement of the compression and shear wave transducers within the platens is shown schematically in Figure 4.2. Nakagawa et al. (1996) used a similar arrangement of compression and shear wave transducers in their ultrasonic probes except that two pairs of shear wave transducers were used instead of three pairs. The piezoelectric crystals change shape when a voltage pulse is applied and generates a voltage when

they are mechanically excited. Therefore, either platen may act as the transmitter or receiver. The pulser is excited by a single voltage spike of 100 V. The data acquisition system can sample data at rates of 0.5 to 40 MHz. In this study, the sampling rate was fixed at 2 MHz. Control of the data acquisition system and its internal compression and shear wave switching circuit is performed through a software program.

The piezoelectric crystals of the compression and shear wave transducers have a natural frequency of between 200 to 1000 kHz. Depending on the manufacturing process of the piezoelectric crystals and their bonding to the platen, the operating frequencies of the compression and shear wave transducers vary from system to system. The operating frequencies of the transducers were not specified by the manufacturer. The operating frequencies of the compression and shear wave transducers were determined by performing a test with the two platens placed in face-to-face contact with each other. The frequency spectra of the compression and shear wave signals from the face-to-face test are shown in Figure 4.3. The frequency spectrum of the compression wave signal shows a single peak at 90 kHz whereas the frequency spectrum of the shear wave signal shows three peaks at 76, 78 and 90 kHz. The peaks at 76 and 78 kHz are higher than that at 90 kHz. The double peaks at 76 and 78 kHz are due to the bonding between the piezoelectric crystals and the platen face (Silk 1984). The 90 kHz peak is most likely caused by a compression wave that was generated together with the stronger shear wave. Hence, the operating frequency for compression wave is 90 kHz and about 76 kHz for shear wave. Therefore, one would expect that the frequency spectrum of the receiver signal to be similar to the transmitted signal albeit a lower amplitude due to attenuation.

Figure 4.1 Experimental set up for ultrasonic soil testing system.

Figure 4.2 Arrangement of compression and shear wave transducers.

Figure 4.3 Frequency spectra of compression and shear waves (face-to-face).

Due to the presence of the platen between the piezoelectric crystals and the test specimen, there will be a face delay caused by the transmission of the pulse through the platen. This face delay needs to be accounted for when determining the travel time of the wave through the test specimen. The face delay can be determined by two methods: (1) using standard test specimens such as steel or aluminum of various lengths as recommended in ASTM D2845 (1997), or (2) from the face-to-face test. Using the first method, the compression and shear wave travel times for aluminum specimens of 50, 150 and 200 mm lengths are shown in Figure 4.4. The y-intercepts of the straight lines give the face delays of the compression and shear waves. Using the second method, the face-to-face compression and shear wave signals are shown in Figure 4.5. If face delay is absent, the y-intercept should start at zero. Both methods give the same face delays, 2.9 μ s for the compression wave and 4.8 μ s for the shear wave.

Figure 4.4 Variation of travel time with length of aluminium.

Figure 4.5 Receiver signals for compression and shear waves (face-to-face).

4.2 Acoustic Coupling

Contact between the specimen and the ultrasonic transducer (platen) or coupling is important for reliable ultrasonic measurement (Szilard 1982, Krautkrämer and Krautkrämer 1990). For the platens and the test specimen to be in good contact, it is important that the test specimen be trimmed flat at both ends and its ends are perpendicular to its sides (ASTM D2845). A non-uniform air gap between the platens and the test specimen will alter the transmission characteristics of the ultrasonic waves. To reduce the effect of a non-uniform air-gap, an acoustic couplant such as water, glycerin, grease, petroleum jelly or oil can be used (Krautkrämer and Krautkrämer 1990). However, the thickness of the couplant should be kept as thin as possible as increasing the thickness of the couplant will degrade the transmitted signal. The differences in the compression and shear wave signals can be easily observed from the face-to-face tests with and without acoustic couplant in the frequency domain as shown in Figures 4.6 and 4.7, respectively. The acoustic couplant used was a fiberglass jelly resin. Figure 4.6 shows that the compression wave frequency spectrum with acoustic couplant has a single peak at 90 kHz and was less noisy than the compression wave frequency spectrum without acoustic couplant. Figure 4.7 shows that the effect of the acoustic couplant is less significant for the shear wave, only increasing the intensities at frequencies of 76, 78 and 90 kHz. The effect of the acoustic couplant can be quantified in terms of the signal to noise ratio, *SNR* (Carlson 1986).

The signal and noise power can be obtained from the power spectrum or frequency spectrum of the signal. The *SNR* for the compression wave shown in Figure 4.6 improves from 3 to 21 while the *SNR* for the shear wave shown in Figure 4.7 improves from 10 to 14 when acoustic couplant was used. The improvements in the compression and shear signals for aluminum, granite and kaolin with acoustic couplant are shown in Figures 4.8 and 4.9, respectively.

Figure 4.6 Frequency spectra for compression wave with and without couplant (face-to-face).

Figure 4.7 Frequency spectra for shear wave with and without couplant (face-to-face).

Figure 4.8 Frequency spectra of compression wave for different materials.

Figure 4.9 Frequency spectra of shear wave for different materials.

4.3 Determination of Wave Velocities and Attenuation

In order to have a better understanding of the ultrasonic system in terms of wave velocities and attenuation determination, eight test specimens of different material types were tested: aluminum, mild steel (Grade 43), stainless steel (304), nylon (Nylon 6), granite, Jurong Formation residual soil from site III, Bukit Timah Granite residual soil from site I and kaolin. Aluminum, mild steel, stainless steel and nylon specimens were used as reference materials as ultrasonic test information on these materials are available. The granite, Jurong Formation residual soil, Bukit Timah Granite residual soil and kaolin specimens represent the range of materials commonly encountered in geotechnical testing. The granite specimen was part of a

rock core in the Bukit Timah Granite obtained from a site in Singapore. The Jurong Formation residual soil and Bukit Timah Granite residual soil specimens were prepared using the standard Proctor compaction method (ASTM D698 1997). Based on the standard Proctor test, the optimum water content and maximum dry density of the Jurong Formation residual soil and Bukit Timah Granite residual soil were 14% and 16% and 1.85 Mg/m^3 and 1.78 Mg/m^3 , respectively. The Jurong Formation residual soil and Bukit Timah Granite residual soil specimens were compacted at four water contents, 19.5% (wet of optimum), 22.1% (wet of optimum), 10.6% (dry of optimum) and 11.7% (dry of optimum). The kaolin specimen was prepared from consolidating kaolin slurry at an initial water content of 230 % under a vertical load of 200 kPa in a 30 cm diameter rigid-wall consolidation tank. The final water content of kaolin specimen is 62.1 % and the bulk density is 1.7 Mg/m^3 . The properties of the eight specimens tested are summarized in Table 4.1. The basic index properties and grain size distributions of Jurong Formation residual soil, Bukit Timah Granite residual soil and kaolin are shown in Table 2.6, Table 3.2, Figure 2.21 and Figure 3.5, respectively.

Table 4.1 Properties and dimensions of specimens tested

Material	Density (Mg/m^3)	Water Content (%)	Dimensions: Diameter (mm) x Length (mm)
Aluminium	2.80	-	50 x 50, 50 x 150, 50 x 200
Mild steel (Grade 43)	7.84	-	50 x 50, 50 x 100, 50 x 150, 50 x 200
Stainless steel (304)	7.82	-	70 x 140
Nylon (Nylon 6)	1.23	-	65 x 50, 65 x 100, 65 x 150, 65 x 200
Granite	2.55	0.0	48 x 121
Kaolin	1.70	62.1	70 x 60
Jurong Formation residual soil:			
M1*	2.10	19.5	70 x 118
M12*	2.01	10.6	70 x 105
Bukit Timah Granite residual soil:			
B1#	2.01	22.1	70 x 100
B12#	1.94	11.7	70 x 110

* confining pressure of 800 kPa

confining pressure of 0 kPa

4.3.1 Wave Velocities

Measurement of wave velocity depends on the accurate determination of wave arrival. ASTM D2845 provides guidelines for the preparation of the specimen's surfaces where the transducers are placed. The surface should be sufficiently flat such that the unevenness is less than 0.025 mm and the opposite surfaces where the transducers are placed should be parallel to within 0.1 mm per 20 mm. ASTM D2845 also recommends that the ratio of specimen length to minimum lateral dimension should not exceed five and the specimen length should be at least 10 times the average grain size. The average grain size may be taken as D_{50} , grain diameter at 50% passing in the grain size distribution curve. These restrictions on specimen size are based on attenuation considerations of the wave. For rock specimens, Lutsch (1959) and Thill and Peng (1969) reported that a wavelength equal to the average grain size will result in almost complete attenuation of the signal thereby making identification of the wave arrival time almost impossible.

4.3.1.1 Compression Wave

For reliable measurement of compression wave velocity, ASTM D2845 further recommends that the minimum lateral dimension should be at least five times the wavelength and the wavelength should be at least three times the average grain size. For a cylindrical specimen, the lateral dimension refers to the diameter of the specimen. The restriction on the minimum lateral dimension as a function of wavelength is due to the dispersion phenomenon. Dispersion arises due to interaction of the wave with the boundary of the system causing wave mode conversion. Dispersion will not occur if the minimum lateral dimension is at least five times the wavelength, i.e. $D/\lambda_p \geq 5$ (Wasley 1973). The effect of dispersion can be checked by varying the diameter of the specimen or by varying the frequency of the pulse. For the ultrasonic test system, the frequency of the input pulse cannot be varied. Therefore, the eight test specimens were of different diameters ranging from 48 to 70 mm with length to diameter ratio L/D ranging from 1.7 to 3.3. The effects

of L/D and L/λ_p (wavelength of compression wave) on compression wave velocities measured by the ultrasonic test system are shown in Figure 4.10 for aluminum, stainless steel and nylon specimens. There is very little effect of L/D and L/λ_p ratios on the compression wave velocities when $L/D \geq 2$ and $L/\lambda_p \geq 2$, respectively.

Figure 4.10 Effect of L/D and L/λ_p on compression wave velocity.

Compression wave velocities of the eight specimens were also measured using a Pundit Mark IV system with 50 mm diameter transducers of 54 kHz (CNS Electronics Ltd 1985) and compared with those measured using the ultrasonic test

system. The Pundit Mark IV system is also an ultrasonic test system and is commonly used for non-destructive testing of concrete (Pundit is the acronym for portable ultrasonic non-destructive digital indicating tester). The operation of the Pundit Mark IV system is similar to the ultrasonic test system described in this paper and is able to measure transit time in the range of 0.1 to 9999 μs . However, the Pundit Mark IV system used is not equipped to output the receiver signal waveform. The compression wave velocities measured using the ultrasonic test system and the Pundit Mark IV system are summarized in Table 4.2 together with the dimensions of the specimens. The compression wave velocities measured using the ultrasonic test system and the Pundit Mark IV system showed good agreement for aluminum, mild steel, stainless steel and nylon but the agreement deteriorates for granite, Jurong Formation residual soil, Bukit Timah Granite residual soil and kaolin as indicated by the ratio of the compression wave velocities in Table 4.2. The differences between the compression wave velocities for the test specimens measured using the ultrasonic test system and the Pundit Mark IV system vary from 1 time to 4 times. Except for stainless steel, the Pundit Mark IV system consistently gave a lower compression wave velocity. The D/λ_p ratios for the corresponding compression wave velocities were computed using V_p/f_c where f_c is the dominant frequency of the wave (90 kHz and 54 kHz for the ultrasonic test system and Pundit Mark IV system, respectively) and summarized in Table 4.2. Most of the D/λ_p ratios are less than one except for the nylon, kaolin, Jurong Formation residual soil and Bukit Timah Granite residual soil specimens. Silaeve and Shamina (1958) showed that the limiting D/λ_p ratio can be reduced from 20 to 2 under ideal conditions of coupling of the transducer, quality of signal and specimen homogeneity whereas Stephenson (1978) reported that the limiting D/λ_p ratio for any solid is one. Therefore, it is possible that the compression wave velocity measured by different ultrasonic test systems may be different depending on the characteristics of the transducer and the test specimen.

Table 4.2 Comparison of compression wave velocities from ultrasonic test system and Pundit Mark IV system

Material	Dimensions: Diameter (mm) x Length (mm)	P wave velocity (m/s)		$V_p(U)/$ $V_p(P)$	D/λ_p	
		Pundit*	Ultrasonic		Pundit	Ultrasonic
Aluminium	50 x 200	6173	6230	1.0	0.4	0.7
Mild steel (Grade 43)	50 x 200	4601	4587	1.0	0.6	1.0
Stainless steel (304)	70 x 140	5780	5600	1.0	0.7	1.1
Nylon (Nylon 6)	65 x 200	2649	2663	1.0	1.3	2.2
Granite	48 x 121	5190	5480	1.1	0.5	0.8
Kaolin	70 x 60	530	1519	2.9	5.1	4.2
Jurong Formation residual soil: M1	70 x 118	730	1359	1.9	5.2	4.6
Bukit Timah Granite residual soil: B1	70 x 100	359	1342	3.8	10.6	4.7
B12	70 x 110	355	379	1.1	10.6	16.6

*Pundit Mark IV test was conducted at confining pressure of 0kPa.

The effect of D/λ_p on the compression wave velocity has been studied by Curtis (1969) (Figure 4.20). Curtis (1982) explained that the head of a wave pulse will propagate at the unbounded velocity. As wavelength increases to greater than the specimen's diameter, i.e. $D/\lambda_p < 1$, the velocity approaches the one-dimensional wave velocity, V_l , given by $\sqrt{E/\rho}$ asymptotically. The difference between the unbounded compression wave velocity, V_p and the one-dimensional compression wave velocity is a function of Poisson's ratio, ν , i.e.

$$\frac{V_p}{V_l} = \sqrt{\frac{(1-\nu^2)}{(1+\nu)(1-2\nu)}} \tag{4.4}$$

$$\nu = \frac{V_p^2 - 2V_s^2}{2(V_p^2 - V_s^2)} \tag{4.5}$$

The compression wave velocities, V_p , in Figure 4.20 were normalized with V_l . The theoretical V_p/V_l ratio for $\nu = 0.34$ (Poisson's ratios for brass and Duralumin are 0.33 and 0.34, respectively) given by Equation 4.4 is also shown in Figure 4.11. Depending on the D/λ_p ratio, the compression wave velocity measured by an ultrasonic test may range from V_l to V_p .

Figure 4.11 Effect of D/λ_p on compression wave velocity (modified from Curtis 1969).

For the data shown in Table 4.2, the ratio of the compression wave velocities measured using the ultrasonic test system and the Pundit Mark IV system, i.e. $V_p(U)/V_p(P)$, are plotted against Poisson's ratio ν as shown in Figure 4.12. Poisson's ratio ν was obtained from the compression and shear wave velocities measured by the ultrasonic test system using Equation 4.5. The curve given by Equation 4.3 is also plotted in Figure 4.12. Depending on D/λ_p ratio, $V_p(U)/V_p(P)$ could range between 1 and the ratio given by Equation 4.4. Figure 4.12 suggests that the compression wave velocity measured by the ultrasonic test system and Pundit Mark IV system is the same for aluminium, mild steel, stainless steel, nylon and granite.

The ratio of $V_p(U)/V_p(P)$ for kaolin, Jurong Formation residual soil and Bukit Timah Granite residual soil approaches the ratio given by Equation 4.4 suggesting that the compression wave velocity measured by ultrasonic test system is V_p whereas the compression wave velocity measured by Pundit Mark IV system is in between V_p and V_l . Figure 4.12 also suggests that the variation between V_p and V_l is insignificant for $\nu < 0.3$.

Figure 4.12 Variation of Poisson's ratio with compression wave velocity.

The compression wave velocities measured using the ultrasonic test system are compared with those reported in the literature in Table 4.3. The compression wave velocities show good agreement with the values reported in the literature.

Table 4.3 Comparison of compression wave velocities from ultrasonic test system and reported values

Material	Compression wave velocity (m/s)	
	Ultrasonic	Typical range
Aluminium	6186	6320 (MetroTek, Inc. –AN23, 1982) 6250 – 6350 (Yoseph and Ajit, 1989)
Mild steel (Grade 43)	4551	3500 – 6140 (Yoseph and Ajit, 1985)
Stainless steel (304)	5600	5850 (MetroTek, Inc. –AN23, 1982) 3500 – 5990 (Yoseph and Ajit, 1989)
Nylon (Nylon 6)	2646	1800 –2200 (Yoseph and Ajit, 1989) 2600 (Krautkrämer & Krautkrämer, 1990)
Granite	5480	5500 – 5900 (AppGeo96, 2002)
Kaolin	1519	1400 – 1600 (AppGeo96, 2002)
Jurong Formation residual soil:		
M1	1359	1400 – 1600 (AppGeo96, 2002)
M12	683	300 – 900 (AppGeo96, 2002)
Bukit Timah Granite residual soil:		
B1	1342	1400 – 1600 (AppGeo96, 2002)
B12	379	300 – 900 (AppGeo96, 2002)

4.3.1.2 Shear Wave

Reliable measurement of shear wave velocities does not impose additional restrictions on the geometry of the specimens if the first (fundamental) mode of propagation is used (Kolsky 1963 and Curtis 1982). However in shear wave velocity measurement using bender elements, near-field effects are cited as a possible source of error when the ratio of specimen length to wavelength, i.e. L/λ_s (wavelength of S wave), is less than a limiting value (Viggiani and Atkinson 1995, Jovicic et al. 1996 and Arulnathan et al. 1998). Sanchez-Salinero et al. (1986) and Arulnathan et al. (1998) suggested limiting value of L/λ_s as 1 and 2, respectively. The near-field effects were investigated using aluminum, mild steel and nylon specimens of various lengths. The effects of L/D and L/λ_s on the shear wave velocities measured using the ultrasonic test system for aluminum, stainless steel and nylon specimens are shown in Figure 4.13. The dominant frequency of the shear wave signals was in the range of 75 kHz to 78 kHz. For computation of λ_s

using $V_s=f\lambda_s$, f is taken as the dominant frequency of the face-to-face shear wave signal, i.e. 76 kHz. There is very little effect of L/D and L/λ_s ratios on the shear wave velocities when $L/D \geq 1$ and $L/\lambda_s \geq 2$, respectively.

Figure 4.13 Effect of L/D and L/λ_s on shear wave velocity.

The measured shear wave velocities of the seven specimens from the ultrasonic tests are shown in Table 4.4. Table 4.4 shows that the measured shear wave velocities are within the range reported in the literature.

Table 4.4 Comparison of shear wave velocities from ultrasonic test system and reported values

Material	Shear wave velocity (m/s)	
	Ultrasonic	Typical range
Aluminium	3160	3130 (MetroTek, Inc.-An23, 1982) 3100 (Yoseph and Ajit, 1985)
Mild steel (Grade 43)	2755	2200 – 3260 (Yoseph and Ajit, 1985)
Stainless steel (304)	3248	3270 (MetroTek, Inc.-AN23, 1982) 2200 – 3270 (Yoseph and Ajit, 1985)
Nylon (Nylon 6)	1141	1120 (Yoseph and Ajit, 1985) 1100 – 1200 (Krautkrämer & Krautkrämer, 1990)
Granite	3090	2800 – 3000 (AppGeo96, 2002)
Kaolin	235	120 – 360 (AppGeo96, 2002) 85 – 207 (Nakagawa, 1996)
Jurong Formation residual soil:		
M1	381	120 – 360 (AppGeo96, 2002) 130 – 300 (Borden et al., 1996)
M12	435	250 – 400 (Ng and Wang, 2001)
Bukit Timah Granite residual soil:		
B1	215	120 – 360 (AppGeo96, 2002) 130 – 300 (Borden et al., 1996)
B12	228	250 – 400 (Ng and Wang, 2001)

4.3.2 Attenuation

Attenuation decreases the amplitude of the signal as a function of time and distance. Measurement of attenuation in heterogeneous materials is a difficult problem (Landis and Shah 1995 and Owino and Jacobs 1999). Attenuation has been shown to be dependent on the porosity of materials (e.g. Generazio et al. 1988 and Winkler and Murphy 1995), grain size (e.g. Winkler 1983 and Prasad and Meissner 1992) and saturation levels (e.g. Tittmann et al. 1980, Prasad and Meissner 1992 and Velea et al. 2000). Attenuation is due to two basic mechanisms: geometric attenuation and material attenuation (Owino and Jacobs 1999). In geometric attenuation, the amplitude of the wave decreases as the wavefront spreads over a large volume as it propagates away from its source. Material attenuation can be divided into intrinsic (absorption) and extrinsic (scattering) (Krautkrämer and Krautkrämer 1990 and Owino and Jacobs 1999). Intrinsic losses are due to internal friction via work done at the material contacts and extrinsic losses are due to the

differences in acoustic impedances at the interfaces (or scatterers) in an inhomogeneous material.

Attenuation has been shown to be strongly dependent on frequency (e.g. Winkler and Murphy 1995 and Owino and Jacobs 1999). For a wide range of materials, attenuation can be expressed according to a power-law relationship (Krautkrämer and Krautkrämer 1990 and He 1998):

$$\alpha(f) = \beta f^r \quad (4.6)$$

where α is attenuation coefficient, f is frequency and usually spans several MHz, β and r are material-dependent parameters. The value of r typically ranges between 1 and 2 (He 1998).

As ultrasonic pulse contains different frequencies, the attenuation of the pulse at different frequency is affected to different degrees and therefore a simple comparison of the amplitudes of two pulses may not give a reliable measure of attenuation (Krautkrämer and Krautkrämer 1990). Vary (1988) has suggested transforming the two pulses from the time domain into the frequency domain to obtain the true attenuation at each frequency from the ratio of the frequency curves. Such an approach has been used by Landis and Shah (1995), He (1998), Owino and Jacobs (1999) and Wang and Cao (2001). However, the transmitter used is a broadband source, with a frequency range of a few MHz. For the ultrasonic test system used in this study, the ranges of frequency for compression and shear waves are only several tens kilohertz. Therefore, the attenuation of the material obtained from the ultrasonic test system is only for a very narrow range of frequency less than 0.1 MHz.

The attenuation of the specimen in the ultrasonic test system can be obtained from the frequency spectra of Fourier transforms of the transmitter and receiver signals (He 1998 and Wang and Cao 2001) as:

$$\alpha = \frac{1}{L} \ln \left(\frac{T_z A_t}{A_r} \right) \quad (4.7)$$

where α is the attenuation coefficient with units of Np/length (1 Np = 8.686 dB), L is the length of the specimen, A_t and A_r are the maximum magnitude of the frequency spectra of transmitter and receiver signals, respectively, and T_z is the transmission coefficient defined as

$$T_z = \frac{4zz_s}{(z + z_s)^2} \quad (4.8)$$

where z and z_s are the acoustic impedances of the transducer and the specimen, respectively. In Equations 4.7 and 4.8, T_z , A_t and z need to be determined. Tests were performed for several lengths of aluminum, mild steel and nylon specimens. The changes in $\ln(A_r)$ with specimen length are shown in Figure 4.14. The intercept of the line gives $\ln(T_z A_t)$ while the slope of the line gives $-\alpha$. Using these intercepts and slopes, the values of z , A_t for compression wave and A_t for shear wave were found to be 22.7×10^6 kg/m²s, 880 and 416, respectively. With these values, the attenuation coefficients of the seven test specimens were obtained as shown in Table 4.5. For comparison, reported attenuation coefficients are also shown in Table 4.5. Direct comparison is not possible as attenuation coefficient is frequency dependent as indicated by Equation 4.6. Table 4.5 indicates that the measured attenuation coefficients are reasonable.

(a)Compression wave

(b)Shear wave

Figure 4.14 Variation of maximum magnitude of frequency spectrum with length of specimen for compression and shear waves.

Table 4.5 Attenuation coefficients for different materials

Material	Attenuation coefficient of P wave velocity (dB/m)		Attenuation coefficient of S wave velocity (dB/m)	
	Ultrasonic	Typical value	Ultrasonic	Typical value
Aluminium	49	10 – 100	62	10 - 100
Mild steel (Grade 43)	75	10 – 100	79	10 - 100
Stainless steel (304)	69	10 – 100	70	10 -100
Nylon (Nylon 6)	63	> 10	86	> 10
Granite	63	> 100	152	> 100
Kaolin	134	> 100	213	> 100
Jurong Formation residual soil:				
M1	203	-	149	-
M12	216	-	23	-
Bukit Timah Granite residual soil:				
B1	310	-	218	-
B12	187	-	108	-

4.4 Estimation of Strain Level for Ultrasonic Testing System

As mentioned in Section 2.3.1.1, the strains associated with the measured velocities are seldom determined in pulse transmission tests. It is widely accepted that the stiffness derived from measurement of the compression and shear wave velocities corresponds to strain levels of less than 0.001% (Addo and Robertson 1992). The compression and shear wave signals recorded by the data acquisition system are the responses of the piezoelectric crystals in the platen due to vibration of the test specimen. If the vibration on the face of the platen is measured with an accelerometer, the vibration and accelerometer signals can be correlated. By associating the receiver signal to particle vibration of the test specimen, the signal levels can be related to an acceleration level. From the acceleration-time history, the particle vibration velocity, v , can be obtained using:

$$v = \int_1^2 a_g dt \tag{4.9}$$

where a_g is the particle acceleration.

The strains for the measured compression and shear wave velocities can be obtained from the particle and wave velocities (White 1965) as:

$$\varepsilon = \frac{v_{p \max}}{V_p} \quad (4.10)$$

$$\gamma = \frac{v_{s \max}}{V_s} \quad (4.11)$$

where $v_{p \max}$ and $v_{s \max}$ are the maximum compression wave particle velocity and shear wave particle velocity, respectively.

To relate the compression and shear wave signals to an acceleration level, a 500g accelerometer (PCB Series 353 quartz-shear ICP 353A03 accelerometer) with a sensitivity of 9.59mV/g was used. For the compression wave, the accelerometer was attached with its axis perpendicular to the face of the platen. For the shear wave, the accelerometer was mounted on a triaxial block with its axis parallel to the face of the platen. The accelerometer signals were correlated to the corresponding ultrasonic signals from the face-to-face tests enabling a relationship between acceleration and the output voltage of the receiver transducer in the ultrasonic test to be determined (Figure 4.15). The frequency spectra of the accelerometer signals are compared with the receiver signals in Figure 4.16. The platen acceleration constants are 0.33 g/mV and 38 g/mV for the compression and shear wave, respectively. Using these constants, the strain levels of the generated (at transmitter) and transmitted (at receiver) compression and shear waves can be determined from the compression and shear wave signals of the face-to-face tests and transit time tests, respectively. The strain levels of the wave velocities of the seven specimens using for calibration are summarized in Table 4.6. The strain level associated with the compression wave velocities is of the order of 1×10^{-4} % and that associated with the shear wave velocities is of the order of 1×10^{-5} %. Therefore, the elastic constants determined from the wave velocities using Equations 4.2 and 4.3 are the so-called very small strain stiffness.

Figure 4.15 Correlation of receiver signal between ultrasonic and accelerometer tests (face-to-face).

Figure 4.16 Frequency spectra of compression and shear waves for ultrasonic and accelerometer tests (face-to-face).

Table 4.6 Strain levels associated with compression and shear wave velocities measured using ultrasonic test system

Material	Strain level of P wave velocity (%)		Strain level of S wave velocity (%)	
	At transmitter	At receiver	At transmitter	At receiver
Aluminium	0.00003	0.00003	0.000005	0.000005
Mild steel (Grade 43)	0.00004	0.00004	0.000003	0.000003
Stainless steel (304)	0.00004	0.00003	0.000003	0.000003
Nylon (Nylon 6)	0.00007	0.00006	0.000007	0.000006
Granite	0.00004	0.00003	0.000003	0.000003
Kaolin	0.0001	0.00005	0.00003	0.000006
Jurong Formation residual soil:				
M1	0.0001	0.00002	0.00002	0.000004
M12	0.0003	0.00001	0.00002	0.000006
Bukit Timah Granite residual soil:				
B1	0.0001	0.00002	0.00004	0.000007
B12	0.0005	0.00002	0.00004	0.00001

4.5 Signal Processing

Signal processing is essential for pulse transmission test. It is required for a variety of reasons: signal is weak, signal is noisy or signal consists of a composite of a number of signals that needs to be separated (Baziw 1993). The principal purpose of signal processing is to improve signal quality and to obtain the “correct” signal. A number of signal processing tools are available.

Stacking of signals helps to reduce random noise and to improve the quality of the signal. De Alba and Baldwin (1991) suggested that stacking 35 to 100 signals depending on the level of attenuation are sufficient to reduce the noise whereas Nakagawa et al. (1996) stacked 100 signals to obtain a clear signal for their system. Stacking is applied to a series of compression and shear wave signals in ultrasonic soil testing system as shown in Figure 4.17 using 4, 12 and 32 signals in the frequency domain. The compression and shear wave signals do not show significant improvement beyond 12 sets of signals. Therefore, the signal stacking is set at 12 sets of signals. For bender element test, stacking is not used as it is not available in

the data acquisition system. The stacked signal from ultrasonic soil testing system needs further signal processing to improve its quality. Windowing and filtering can help to reduce signal noise and Gibbs' phenomenon (Hamming 1977). Gibbs' phenomenon is distortion or leakage due to discontinuities in the frequency domain. A signal truncated due to a short recording time is equivalent to multiplying the signal with a boxcar or rectangular function in the time domain in which there are discontinuities to produce an initial transient i.e. leakage in the frequency domain (Baziw 1993) as shown in Figure 4.18. To minimize Gibbs' phenomenon, the signal needs to be tapered. This can be achieved using a pair of cosine bell such as Hamming or Hanning window so that there are no discontinuities to produce the initial transient (Hamming 1977 and Baziw 1993). Figure 4.18 shows the tapered signal by Hanning window. The signal in Figure 4.18a tapered by Hanning window show the initial transient was eliminated (Figure 4.18b).

Filtering will further enhance the signal while removing unwanted components from the signal. In this report, the stacked and windowed signal was passed through a low-pass fourth order Butterworth filter (normal infinite response or IIR type filter) with a cut-off frequency that is 10 kHz above the dominant frequency of the input signal. However, passing the signal through a frequency selective filter will introduce phase distortion resulting in time delay in the filtered signal. This phase distortion needs to be corrected. As the phase-frequency relationship is non-linear and the response always contains more than one frequency, the delay due to phase shift cannot be adjusted by simply adding a constant phase shift (Li 1997). The phase shift can be corrected by either using an Ormsby filter or two passes of an IIR filter (Oppenheim and Schaffer 1975, Sunder 1981 and Lee and Trifunac 1984). The Ormsby filter is a linear phase IIR filter and hence the phase-frequency relationship is linear. If the Ormsby filter is used, the time delay introduced by the phase shift can be corrected by adding a constant time shift (Li 1997). In this paper, the two-pass IIR filter method was employed. The signal was first passed through a Butterworth filter; the order of the signal was reversed and then passed through the Butterworth filter again. The final zero-phase shift signal is the reverse order of the

two-pass signal (Li 1997). The effect of filtering alone on the signal in Figure 4.18a is shown in Figure 4.18c. The arrival time of the wave can be determined easily from the first deflection of the windowed and filtered signal as shown in Figure 4.18d.

Figure 4.17 Effect of stacking on compression and shear wave receiver signals.

Figure 4.18 Effect of windowing and filtering on receiver signal.

In summary, the procedure of signal processing employed is as follows:

- (1) Sample the signal at a sampling rate of at least twice the operating frequency of the transducer
- (2) Stack the signal 12 times to eliminate random noise. No stacking is required for bender element test.

- (3) Taper the signal in the time domain using Hanning window to eliminate Gibbs' phenomenon.
- (4) Fast fourier transform (FFT) the signal to obtain the signal in the frequency domain and determine the dominant frequency of the signal.
- (5) Pass the signal in the time domain through two passes of a fourth order, low-pass Butterworth filter with a cut-off frequency of 10 kHz above the dominant frequency of the signal to obtain the processed signal.

4.6 Comparison of Bender Element and Ultrasonic Tests

Both bender element and ultrasonic tests are pulse transmission test. Although both tests use the same testing principles, they have their own merits and demerits. Table 4.7 summarizes the merits and demerits of the bender element test and the ultrasonic test.

Table 4.7 Merits and demerits of bender element test and ultrasonic test

Test	Merits	Demerits
Bender element test	<ul style="list-style-type: none">• The piezoelectric crystals are in intimate contact with the soil specimen as the bender element protruded into the soil specimen (Figure 3.2). Therefore, the coupling between the piezoelectric crystals and soil specimen is stronger which results in a stronger signal being transmitted.	<ul style="list-style-type: none">• The waterproofing for bender element is provided by a coating of epoxy glue. The epoxy glue is not durable and is subjected to wear and tear. Thus, bender element is prone to short circuit due to water penetration into the bender element through cracks in the epoxy glue.• Once short circuit occurred, the bender element has to be changed. The process of changing the bender element is tedious and time consuming.• It is not easy to place the soil specimen in direct contact with the platens as the bender element is difficult to insert into hard and stiff soil. Care must be taken in order not to damage the bender element.• To the author's knowledge, no method is available for

Test	Merits	Demerits
		<p>determination of damping ratio or attenuation using bender element.</p> <ul style="list-style-type: none">• The volume of soil influenced by the propagation of the shear wave in the bender element test is less as compared to ultrasonic test due to the arrangement of bender element (Figure 4.19). As the width of the bender element is small (i.e. a few mm), the travel path for shear wave is narrower. Thus, the results are subjected to more variation and inconsistency compared to the ultrasonic test.
Ultrasonic test	<ul style="list-style-type: none">• The piezoelectric crystals are bonded to the inside face of the stainless steel platen. Thus, it is 100% water proofed and not prone to short circuit if the platens are properly sealed.• The piezoelectric crystals are protected by the housing in the platen and thus more robust.• It is easy to place the soil specimen in contact with the platen as the platen is flat with piezoelectric crystals inside the platens and not protruding into the soil specimen.• The attenuation or damping ratio of the soil can be determined (Section 4.3).• The volume of soil influenced by the propagation of the shear wave in the ultrasonic test is larger than in bender element test (Figure 4.20). This is because the piezoelectric crystals are arranged in a circular layout with an outer diameter of 50 mm. The travel path for the shear wave is broader and gives a more consistent result compared to the bender element test.	<ul style="list-style-type: none">• The piezoelectric crystals are bonded to inside face of platen, thus, the piezoelectric crystals are not directly in contact with the soil specimen. This arrangement weakened the signal transmitted through the soil specimen. In order to solve this problem, acoustic couplant is required.

Figure 4.19 Travel path of shear wave in bender element test.

Figure 4.20 Travel path of shear wave in ultrasonic test.

Chapter 5 Cyclic Simple Shear and Triaxial Compression Tests

The cyclic simple shear apparatus is used to determine the shear modulus of soils at strain levels greater than 0.005% but less than 1%. In the cyclic simple shear test, cyclic horizontal shear stresses are applied to the top or bottom of the soil specimen such that the deformation of the soil specimen is similar to a soil element subjected to vertically propagating shear wave (Boulanger et al. 1993, Kammerer et al. 2001 and Hsu 2002). The cyclic simple shear was found to simulate the in situ shear conditions for earthquake loading more closely than other alternative devices such as cyclic triaxial and cyclic torsional shear at a reasonable cost in terms of equipment, testing condition and specimen preparation techniques (Peacock and Seed 1968, Finn et al. 1971, Krizek and Borden 1977 and Boulanger et al. 1993). Furthermore, specimen preparation for cyclic simple shear is easier than torsional shear test which involves preparation of hollow cylindrical specimen. The shear modulus and damping ratio can be obtained directly from the stress-strain curve using Equations 2.1 and 2.5, respectively. The conventional triaxial compression test is used to determine shear modulus at strain levels greater than 1%. Combining the test results from ultrasonic test, cyclic simple shear test and triaxial compression test enables the shear modulus-strain curve for strain levels ranging from 0.0001% to 5% to be obtained. This chapter describes the cyclic simple shear apparatus used and its performance as well as the triaxial compression apparatus used.

5.1 Cyclic Simple Shear

Cyclic simple shear represents the in situ shear condition of earthquake loading well but there are some drawbacks (Amer et al. 1986). The most significant problem is boundary condition that creates non-uniformity in the stress and strain across the soil specimen. Non-uniform stress develops as the simple shear apparatus is incapable of producing complimentary shear stresses on the lateral boundaries as

shown in Figure 5.1(Amer et al. 1986). The absence of complimentary shear stresses results in error in the test.

Figure 5.1 Ideal and non-ideal simple shear states.

However, this problem can be solved by imposing a minimum specimen diameter to height ratio. Lucks et al. (1972), De Alba et al. (1976), Shen et al. (1978), Franke et al. (1979), Finn et al. (1982), Vucetic and Lacasse (1982) and Amer et al. (1986) showed that an increase of diameter to height ratio improves the uniformity of stress distribution. Diameter to height ratios of 2 to 8 were used by different researchers. Finn et al. (1982) found that soil specimen with diameter to height ratio of 2 has remarkably uniform shear stress and approaches those with larger diameter to height ratio. In addition, ASTM D3080-90 (1997) a standard test method for direct simple shear test of soil, has also recommended a minimum diameter to height ratio of 2. Thus, the cyclic simple shear test conducted in this study used a diameter to height ratio of 2.

Another source of stress non-uniformity in simple shear test is non-uniform specimen deformation during the application of vertical stress (Roscoe 1953). Both lateral and vertical deformations occurred during the application of vertical stress as the lateral boundary condition does not effectively limit the lateral expansion of the specimen. These lateral deformations will redistribute and cause non-uniformity in stress. The use of wire reinforced membranes introduced by NGI is supposed to restrain the specimen from deforming laterally. However, Budhu (1985) had some doubts on the use of wire reinforced membranes. He had conducted simple shear

test using both Roscoe-type and NGI-type apparatuses. The Roscoe-type simple shear apparatus has a rigid wall to restrain the lateral deformation to ensure a uniform stress or strain distribution while the lateral constraint in NGI-type simple shear apparatus is provided by the wire reinforced membranes only. He found that the wire reinforced membranes were not stiff enough to prevent lateral expansion. In order to solve this problem, the simple shear test used in this study does not use wire reinforced membranes. The lateral constraint is provided by applying confining pressure through the water around the specimen which is more rigid than the wire reinforced membranes. The same approach was employed by Boulanger et al. (1993) and Boulanger and Seed (1995) in their cyclic simple shear apparatus. In this case, the lateral restraint is directly controlled instead of relying on the rigidity of a wire wrapped membrane. An additional advantage of this set-up is that the lateral effective stress can be easily measured as the difference between the confining pressure and internal pore-water pressure. The maximum confining pressure that can be applied in the system is 1000 kPa.

There are two types of simple shear apparatus: NGI type and Cambridge type. The NGI type used a cylindrical specimen while the Cambridge type used a square specimen. The NGI type is more suitable for measuring shear modulus of soil due to its simplicity in specimen preparation. The cyclic simple shear apparatus used in this study is from GCTS, USA, which is a modified NGI type simple shear apparatus. Figure 5.2 shows the schematic of the cyclic simple shear testing system. The system consists of a pair of 70 mm diameter platens, electro-hydraulic power unit, triaxial cell system with normal and shear load actuators and computer interface unit with servo control, data acquisition system with signal conditioning and personal computer. The system is capable to test soil specimen subjected to cyclic stress or strain at a maximum frequency of 20 Hz. The electro-hydraulic power unit is used to apply and maintain load for the normal and shear load actuators. The unit is equipped with a 10 horse power per 30 litres per mins (i.e. 8 GPM) variable volume, constant pressure hydraulic pump and 2000 cc accumulator.

There is a 19LPM servo valve connected to each actuator for closed loop control of load or displacement.

There are two linear variable differential transformers (LVDTs) being installed on the cyclic simple shear apparatus from GCTS for measurement of normal and shear displacements. The LVDTs are capable of measuring displacement of ± 25 mm with a precision of 1 mm corresponding to shear strain of 1%. The purpose of this study is to obtain shear modulus and damping ratio at shear strain less than 1%. The LVDTs are not able to accomplish the task. Modification was made to the cyclic simple shear apparatus in order to measure shear modulus and damping ratio at shear strain ranging from 0.005% to 1%. An additional LVDT capable of measuring shear displacement of ± 1.25 mm with a precision of 0.001mm was installed (Figure 5.2). The strain-controlled cyclic simple shear tests are conducted using the ± 1.25 mm range LVDT in order to achieve the objective of this study. The advantages of this system are that:

- i) The servo system uses a proportional-integral-derivative (PID) algorithm enables it to provide a stiff response and load or deformation as prescribed.
- ii) The lateral restraint is controlled by the applying of confining pressure instead of wire wrapped membrane so that the lateral effective stress can be easily measured and isotropic condition can be easily achieved.
- iii) The system has a stiff internal support to minimize lateral compliance of the top cap.
- iv) The system has a fixed top and a sliding bottom base mounted on special linear bearings to reduce the friction between the apparatus and the soil specimen.
- v) The computer interface unit with servo control and data acquisition with signal conditioning enables the user to control the system through computer software.

- vi) The data collected are stored in the personal computer and can be retrieved for data processing.

Load cells and pore pressure transducer were installed in the system for measuring of stress and pore-water pressure. Table 5.1 summarizes the capacity and sensitivity of the sensors.

Figure 5.2 Experimental set up for cyclic simple shear.

Table 5.1 Capacity and sensitivity of the sensors for cyclic simple shear test

Sensor	Part Number	Capacity	Sensitivity	Minimum Sensitivity (ASTM D5311-92)
Normal Load	SW10 - 5k	25 kN	2.22 kN/V	100 kN/V
Shear Load	SW10 - 5k	25 kN	2.22 kN/V	100 kN/V
Normal Deformation	PR - 750 - 1000	50 mm	2.54 mm/V	25 mm/V
Shear Deformation	PR - 750 - 1000	50 mm	2.54 mm/V	25 mm/V
Shear Deformation [#]	PR - 750 - 050	1.25 mm	0.06 mm/V	25 mm/V
Confining Pressure	Validyne DP15 - 50	1000 kPa	100 kPa/V	100 kPa/V
Effective Confining Pressure	Validyne DP15 - 50	1000 kPa	100 kPa/V	100 kPa/V

[#] additional LVDT installed

5.1.1 Undrained Loading Test in Cyclic Simple Shear

In undrained simple shear loading, the boundary conditions are generally assumed to be described by a constant vertical load (i.e. total stress), zero lateral strains and zero vertical strains (Boulanger et al. 1993, Boulanger and Seed 1995). A combination of “constant volume” and “constant height” procedure is adopted to achieve the boundary condition for undrained simple shear loading. In this procedure, a constant vertical stress is applied and the change of height in the test specimen (i.e. vertical strain) is monitored to ensure that the change in vertical strain is almost insignificant or zero. This condition occurs if the simple shear deformation is not in rigorous mode (Bilé Serra and Hooker 2000 and Hsu 2002). The test specimen is fully saturated and drainage of water is mechanically prevented from the test specimen. Thus, the volume of the specimen is maintained (i.e. zero volumetric change). This implies a zero lateral strain since the area of the specimen is constant. In this study, the performance of the cyclic simple shear apparatus for the strain-controlled undrained loading test was verified using kaolin specimen.

5.1.1.1 Performance of the Cyclic Simple Shear Apparatus under Undrained Loading Condition

Kaolin was chosen to evaluate the performance of the cyclic simple shear apparatus. The basic index properties for kaolin are shown in Table 3.2 and Figure 3.5. Kaolin specimen was prepared from consolidating kaolin slurry at an initial water content of 200% under a vertical load of 200 kPa in a 30 cm diameter consolidation tank. The final water content of the kaolin is 54.12% and the bulk density is 1.7 Mg/m^3 . The kaolin specimen was trimmed to a diameter of 70 mm and a height of 35 mm using a soil lathe. The kaolin specimen was subjected to five loading cycles under an effective confining pressure of 200 kPa using the combination of ‘constant height’ and ‘constant volume’ procedure (Boulanger et al. 1993). The cyclic shear

strain applied, γ , varies from 0.0075% to 1%. A minimum sampling rate of 40 data point per cycle was logged during cyclic loading (ASTM D3999).

Figure 5.3 shows the typical test results obtained for kaolin specimen at an applied cyclic shear strain of $\pm 0.018\%$. The normal stress (i.e. total vertical stress) and the normal strain (i.e. vertical strain) remained constant throughout the cyclic loading. In addition, it is also observed that the variation of the applied shear strain with time is as prescribed and complied with the requirements in ASTM D5311-92 (1997).

Figure 5.3 Typical results obtained for kaolin specimen at an applied cyclic shear strain of $\pm 0.018\%$.

Figure 5.4 shows the stress-strain loop of the first cycle for four different applied shear strain levels. The stress-strain loops are clearly defined and the shear modulus can be determined easily. Thus, the cyclic simple shear apparatus is able to determine the shear modulus at shear strain levels between 0.0075% to 1%. The

variation of shear modulus and damping ratio with the shear strain levels for the kaolin specimen is plotted in Figure 5.5. It is observed that shear modulus decreases as the shear strain level increase, a trend which is similar to Figure 2.1. It is consistent with the trend reported in the literature (Thiers and Seed 1968, Silver and Seed 1971, Hardin and Drnevich 1972, Dobry et al. 1981, Ray and Woods 1988, Vucetic and Dobry 1991, Kagawa 1992, Macari and Hoyos 1996, Lanzo et al. 1997, Doroudian and Vucetic 1998, Rollins et al. 1998, Hsu 2002, D’Elia et al. 2003, Matešić and Vucetic 2003, Wehling et al. 2003 and Yashuhara et al. 2003).

Figure 5.4 First cycle stress-strain curves for kaolin specimen at various applied shear strain.

(1996), Guha et al. (1997), Rollins et al. (1998), Wehling et al. (2003) and Yasuhara et al. (2003). Viggiani (1992) did not determine the damping ratio for kaolin. The damping ratio for kaolin obtained by Doroudian and Vucetic (1995) does not match with the damping ratio obtained in this study because the testing condition is different (Figure 5.6). Doroudian and Vucetic (1995) did not determine the damping ratio for shear strain greater than 0.02%. Thus, comparison between the data cannot be made.

* shaded symbols are damping ratio for kaolin

Figure 5.6 Variation of G/G_{max} and damping ratio with shear strain for kaolin.

There is a unique relationship between shear modulus and damping ratio with shear strain. Figure 5.7 shows the typical shear modulus-strain and damping ratio-strain curves. In general, the shear modulus-strain and damping ratio-strain curves can be divided into four regions namely, region I, II, III and IV as shown in Figure 5.7. The description of the characteristics in each region is given in Table 5.2.

Figure 5.5 Variation of shear modulus with shear strain for kaolin.

The shear modulus for kaolin found in the literature was determined under different effective confining pressure, void ratio and plasticity index. Furthermore, the kaolin characteristics are different depending on the source. In order to compare with the shear modulus for kaolin in the literature, the shear modulus is normalized with the G_{max} obtained from ultrasonic test. The G_{max} for the kaolin in this study is 107 MPa. The G/G_{max} versus shear strain curve for kaolin in this study and those obtained by Viggiani (1992) and Doroudian and Vucetic (1995) are compared in Figure 5.6. It is found that the G/G_{max} obtained by Viggiani (1992) matches the G/G_{max} obtained in this study. However, the G/G_{max} obtained by Doroudian and Vucetic (1995) does not match with the G/G_{max} obtained in this study. This is because Doroudian and Vucetic (1995) tested kaolin (PI=21%) using a double specimen cyclic simple shear apparatus at an effective confining pressure of 150 kPa under anisotropic condition where the lateral earth coefficient was taken as 0.5. The author and Viggiani (1992) tested kaolin under an isotropic condition. Viggiani (1992) tested kaolin (PI=24%) using a cyclic triaxial at an effective confining pressure of 100 kPa under isotropic condition.

It is also observed that damping ratio for kaolin (shaded symbols in Figure 5.5) increases as the shear strain levels increases, a trend which is similar to Figure 2.3. A similar trend is obtained by Thiers and Seed (1968), Park and Silver (1975), Anderson et al. (1983), Seed et al. (1986), Kagawa (1992), Macari and Hoyos

Figure 5.7 Typical shear modulus-strain and damping ratio-strain curves.

Table 5.2 Characteristics of shear modulus-strain and damping ratio-strain curves in each region

Region	Shear Modulus-Strain Curve	Damping Ratio-Strain Curve
I	The shear modulus is at its maximum and remains approximately constant.	The damping ratio is at its minimum or very small value and remains almost constant.
II	The rate of change for shear modulus with respect to shear strain increases as shear strain increases. The curve is convex as shown in Figure 5.7.	The rate of change for damping ratio with respect to shear strain increases as shear strain increases. The curve is concave as shown in Figure 5.7.
III	The rate of change for shear modulus with respect to shear strain decreases as shear strain increases. The curve is concave as shown in Figure 5.7.	The rate of change for damping ratio with respect to shear strain decreases as shear strain increases. The curve is convex as shown in Figure 5.7.
IV	The shear modulus is at its minimum or a very small value and remains almost constant.	The damping ratio is at its maximum and remains approximately constant.

Based on Figure 5.7 and Table 5.2, if the shear modulus is at its maximum then damping ratio is at its minimum and vice versa. In addition, if the rate of change for shear modulus with respect to shear strain increases then the rate of change for damping ratio with respect to shear strain increases and vice versa. This explains the main difference between the data obtained by Doroudian and Vucetic (1995) and those in this study. For the data in this study, the Region III and Region IV were captured. However, Doroudian and Vucetic (1995) data lie in Region I and Region II.

Although there is a unique relationship between the change of shear modulus and damping ratio with shear strain, the rate of change in terms of magnitude is not the same as both shear modulus-strain and damping ratio-strain are characterised by different functions.

The shear modulus, G , function is given as:

$$G = \frac{G_{\max}}{1 + \gamma_h} \quad (5.1)$$

where γ_h is the hyperbolic shear strain for shear modulus and is defined as (Hardin and Drnevich 1972):

$$\gamma_h = \left(\frac{\gamma}{\gamma_r} \right) \left[1 + p e^{-c \left(\frac{\gamma}{\gamma_r} \right)} \right] \quad (5.2)$$

where γ_r is the reference shear strain, e is the exponential function, c and p are constants.

The damping ratio, D_{amp} , function is given as:

$$D_{amp} = \frac{D_{\max} \gamma_h'}{1 + \gamma_h'} \quad (5.3)$$

where $D_{\max}$ is the maximum damping ratio and γ_h' is hyperbolic shear strain for damping ratio. The γ_h' is defined as (Hardin and Drnevich 1972):

$$\gamma_h' = \left(\frac{\gamma}{\gamma_r} \right) \left[1 + p_1 e^{-c_1 \left(\frac{\gamma}{\gamma_r} \right)} \right] \quad (5.4)$$

where c_1 and p_1 are constants.

5.2 Triaxial Compression

The triaxial compression system used in this study is the same as that described together with ultrasonic system or bender element test (Figures 3.2 or 4.3). Thus, the shear modulus at strain level of 0.0001% and shear modulus for strain levels larger than 1% can be determined in a single triaxial apparatus. The LVDT used is capable of measuring a displacement up to 12.5 mm. The strain level is measured externally. Therefore it is subjected to errors such as compliance of the loading and load measuring and bedding effect (Baldi et al. 1988). These errors can be eliminated by measuring strain inside the triaxial cell locally on the sample. A number of methods to measure strain locally are proposed over the years (Burland and Symes 1982, Jardine et al. 1984, Goto et al. 1991, Tatsuoka et al. 1994, 1999). However, Viggiani (1992) stated that the importance of local strain measurement reduces as strain level increases and measurement of local strain becomes difficult at larger strain levels. If the specimen is consolidated in the triaxial cell, the bedding error is insignificant at large strain level except for very hard and stiff material such as soft rock (Viggiani and Atkinson 1995). Several researchers had used external strain measurement to determine the soil stiffness at strain levels larger than 1% (Viggiani 1992, Viggiani and Atkinson 1995, Boulanger et al. 1998 and Wehling et al. 2003). The results were found to be reliable. Therefore, the shear modulus for large strain levels (>1%) in this study was determined using external strain measurement. Table 5.3 shows the capacity and sensitivity of the sensors used in the triaxial apparatus.

Table 5.3 Capacity and sensitivity of the sensors for triaxial compression test

Sensor	Part Number	Capacity	Sensitivity	Minimum Sensitivity (ASTM D5311-92)
Normal Load	STACL3	5 kN	0.43 kN/V	50 kN/V
Normal Deformation	HS10B	12.5 mm	1.12 mm/V	25 mm/V
Pore-water Pressure	PDCR 810	1000 kPa	85.47 kPa/V	100 kPa/V

5.2.1 Undrained Loading Test in Triaxial Compression

The undrained triaxial compression tests were conducted in accordance with ASTM D4767-95 (1997) with a loading and reloading done at an axial strain greater than 1%. Figure 5.8 shows the typical result obtained for the kaolin specimen described in Section 5.1.1.1 at an effective confining pressure of 100 kPa. The deviator stress increases as the axial strain increases. The Young’s modulus decreases as the axial strain increases. The Young’s modulus, E , is defined as the tangential gradient of the stress-strain curve at a particular axial strain. The Young’s modulus, E , can be converted into shear modulus, G , using theory of elasticity:

$$G = \frac{E}{2(1 + \nu)} \tag{5.5}$$

where ν is the Poisson’s ratio.

Axial strain, ϵ , is also related to shear strain, γ , by

$$\gamma = (1 + \nu)\epsilon \tag{5.6}$$

The specimen was fully saturated and test was conducted under undrained condition, therefore, the Poisson’s ratio was taken as 0.5. Table 5.4 shows the shear modulus and shear strain obtained for the kaolin specimen using triaxial compression under consolidated undrained condition.

Table 5.4 Shear modulus and shear strain for kaolin specimen described in Section 5.1.1.1 at an effective confining pressure of 100 kPa

ϵ (%)	E (MPa)	γ (%)	G (MPa)
0.25	6.54	0.38	2.18
0.5	5.50	0.75	1.83

ε (%)	E (MPa)	γ (%)	G (MPa)
1.0*	1.97	1.5	0.66
1.5*	0.97	3.0	0.32
2.0*	0.56	4.5	0.19

* used for large strain shear modulus determination

Figure 5.8 Typical result obtained for kaolin specimen described in Section 5.1.1.1 at an effective confining pressure of 100 kPa.

Chapter 6 Very Small Strain Stiffness and Damping Ratio

Soil stiffness remains almost constant if the shear strain is very small. It is termed as very small strain stiffness in this study while its corresponding damping ratio is termed as very small strain damping ratio. There are many factors that affect the very small strain soil stiffness and damping ratio. Some of these factors have been investigated by others while some have not been investigated to a satisfactory extent. In this study the effects of confining pressure, void ratio and degree of saturation on the very small strain soil stiffness and damping ratio for both Bukit Timah Granite and Jurong Formation residual soils were investigated. The scope of this chapter include the initial shear modulus, initial bulk modulus and damping ratio of residual soil for both Bukit Timah Granite site I and Jurong Formation site III in Singapore.

6.1 Strain Level

Soil stiffness at very small strain is termed as initial shear modulus, G_{max} , under shear loading and initial bulk modulus, $E_{bulk,max}$, under compression loading. It is believed that there is a cyclic threshold shear strain in soil to divide the shear strain into two different categories: a) very small shear strain and b) small shear strain (Seed and Idriss 1970, Youd 1972, Anderson and Richart 1976, Stoll and Kald 1977, Lo Presti 1989, Vucetic and Dobry 1991, Georgiannou et al. 1991, Vucetic 1992, Matasovic and Vucetic 1992, Vucetic 1994 and Hsu 2002). The definition and range of cyclic threshold shear strain or very small shear strain varies. Anderson and Richart (1976) defined the threshold shear strain for clays as the strain level that if exceeded, the G_{max} decreases progressively with increasing shear strain levels. They suggested a threshold shear strain of approximately 0.01%. Stoll and Kald (1977) defined the threshold shear strain for sand as the shear strain level at which there is no tendency for permanent pore-water pressure to change. They suggested a threshold shear strain level of 0.005% to 0.006%.

Based on the literature, the definition for threshold shear strain can be divided into two types. The cyclic shear strain which corresponds to G/G_{max} of approximately 0.99 is termed as linear cyclic threshold shear strain, γ_{tl} (Vucetic 1994). However the cyclic shear strain above which a significant permanent pore-water pressure change will occur in soil but below which the soil microstructure remain practically unchanged, is termed as cyclic volumetric threshold shear strain, γ_{tv} (Vucetic 1994). Very small strain is defined as the shear strain in which the strain level is less than the linear cyclic threshold shear strain, γ_{tl} (i.e. $\gamma < \gamma_{tl}$). Both γ_{tl} and γ_{tv} are functions of plasticity index (PI), stress history and strain rate. Lo Prest (1989), Georgiannou et al. (1991), Vucetic (1994) and Hsu (2002) reported that both γ_{tl} and γ_{tv} increase with plasticity index. The linear cyclic threshold shear strain is generally assumed to be 0.0005% for soils with low plasticity index, less than 20 (Vucetic 1994). Santucci de Magistris et al. (1999) and Matešić and Vucetic (2003) showed that γ_{tl} increases with strain rate for various soils. However, the dependency of γ_{tl} on strain rate diminishes as strain rate increases. The value of γ_{tl} is virtually independent of strain rate if the strain rate is greater than 0.1%/min (Santucci de Magistris et al. 1999). In general, γ_{tl} is found to be below 0.0005% to 0.001% for both clay and sand (Santucci de Magistris et al. 1999 and Matešić and Vucetic 2003).

In this study, pulse transmission technique such as ultrasonic test and bending element test was employed to obtain soil stiffness at strains below 0.0005%. In the pulse transmission test, the damping ratio, D_{amp} , cannot be determined directly but is related to the attenuation coefficient, α , from the ultrasonic test using the following equation:

$$\alpha = \frac{2\pi f D_{amp}}{V \sqrt{1 - D_{amp}^2}} \quad (6.1)$$

where V is the wave velocity and f is the frequency.

In this study, the soil stiffness and damping ratio of very small shear strains (i.e. $\gamma < 0.0005\%$) for compacted Bukit Timah Granite site I residual soil and Jurong Formation site III residual soil were determined using bender element test and ultrasonic test. The effects of confining pressure, void ratio and degree of saturation on very small strain soil stiffness and damping ratio (i.e. $\gamma < 0.0005\%$) for compacted Bukit Timah Granite site I residual soil and Jurong Formation site III residual soil are discussed in this chapter.

6.2 Testing Program

A series of pulse transmission tests was conducted on compacted Bukit Timah Granite site I and Jurong Formation site III residual soils. The Bukit Timah Granite site I and Jurong Formation site III residual soils were compacted in accordance with ASTM D698 (denoted as standard 3) and D1557 (denoted as modified 5). The compaction curves are shown in Figures 6.1 and 6.2. Two other compaction energy levels were used for Bukit Timah Granite site I and Jurong Formation site III residual soils by modifying the standard and modified Proctor compaction tests slightly: In the standard Proctor compaction test, five layers instead of three were used (denoted as standard 5) and in the modified Proctor compaction tests, three layers instead of five were used (denoted as modified 3). These compaction curves are also shown in Figures 6.1 and 6.2 together with the 100%, 80%, 70%, 65% and 55% saturation lines. The compaction parameters for the four different compaction energy levels are shown in Table 6.1.

Figure 6.1 Compaction curves for Bukit Timah Granite site I residual soil.

Figure 6.2 Compaction curves for Jurong Formation site III residual soil.

Table 6.1 Compaction parameter for different energy levels for Bukit Timah Granite site I and Jurong Formation site III residual soils

Soil	Hammer	No of Layers	ρ_{dmax} (Mg/m ³)	W_{opt} (%)	Remarks
Bukit Timah Granite site I residual soil	Standard Proctor	3	1.78	16.00	Standard 3 ASTM D698
		5	1.81	14.00	Standard 5 Modified ASTM D698
	Modified Proctor	3	1.87	13.30	Modified 3 Modified ASTM D1557
		5	1.91	12.60	Modified 5 ASTM D1557
	Standard Proctor	3	1.85	13.90	Standard 3 ASTM D698
		5	1.90	13.50	Standard 5 Modified ASTM D698
Jurong Formation site III residual soil	Modified Proctor	3	1.94	12.00	Modified 3 Modified ASTM D1557
		5	1.99	11.00	Modified 5 ASTM D1557

The compacted specimen of size 101 mm (diameter) x 117 mm (height) was trimmed into the required diameter using a soil lathe. After trimming, the specimen was placed in a two part split mould and trimmed to the final height. The surface of the specimen was trimmed flat with a steel straight edge. For bender element test, the compacted soil specimen was trimmed to a diameter of 50 mm and a height of 110 mm. For ultrasonic test, the compacted soil specimen was trimmed to a diameter of 70 mm and a height of 110 mm.

Both bender element and ultrasonic tests were conducted under unconsolidated undrained condition (UU), the wave velocities were measured immediately after the application of confining pressure. Unconsolidated undrained (UU) test condition was used instead of consolidated undrained (CU) and consolidated drained (CD) test conditions because the pre-stress (“pre-consolidation pressure”) of the compacted soils are difficult to estimate. One of the objectives of the bender element and ultrasonic tests is to study the effect of degree of saturation and

confining pressure on the compression and shear wave velocities of unsaturated soils. Therefore, UU tests were preferred over CD and CU tests.

A total of 16 soil specimens were tested under unconsolidated undrained condition using bender element test to obtain the initial shear modulus. Another 32 soil specimens were tested under unconsolidated undrained condition using ultrasonic test to obtain initial shear modulus and initial bulk modulus. The properties of the soil specimens tested under UU condition are summarized in Tables 6.2 and 6.3.

Table 6.2 Properties of compacted residual soil specimens for bender element test (UU test condition)

Specimen Number	e	ρ (Mg/m ³)	w (%)	Remarks
B1	0.47	2.12	16.98	Standard 3
B4	0.43	2.11	12.67	Standard 3
B6	0.40	2.15	12.51	Standard 5
B7	0.36	2.17	10.41	Modified 3
B8	0.35	2.20	11.09	Modified 5
B9	0.38	2.19	13.05	Standard 5
B10	0.39	2.17	12.75	Modified 3
B11	0.35	2.20	11.58	Modified 5
B12	0.46	2.03	11.20	Standard 3
B13	0.43	2.06	10.25	Standard 5
B14	0.40	2.08	9.13	Modified 3
B15	0.38	2.09	8.04	Modified 5
B16	0.52	2.09	18.59	Standard 3
B17	0.46	2.14	17.00	Standard 5
B18	0.44	2.15	16.15	Modified 3
B19	0.41	2.18	15.33	Modified 5

Table 6.3 Properties of compacted residual soil specimens for ultrasonic test (UU test condition)

Soil	Specimen Number	e	ρ (Mg/m ³)	w (%)	Remarks
Bukit Timah Granite residual soil	T1	0.50	2.04	19.6	Standard 3
	T4	0.45	2.00	13.1	Standard 3
	T6	0.45	1.99	12.3	Standard 5
	T7	0.39	2.06	11.5	Modified 3
	T8	0.36	2.08	10.5	Modified 5
	T9	0.46	2.07	17.9	Standard 5
	T10	0.41	2.10	16.2	Modified 3
	T11	0.47	2.06	18.5	Modified 5
	T12	0.50	1.91	12.2	Standard 3
	T13	0.46	1.95	11.1	Standard 5
	T14	0.42	1.98	10.0	Modified 3
	T15	0.39	2.02	9.7	Modified 5
	T16	0.45	2.06	16.1	Standard 3
	T17	0.42	2.07	14.3	Standard 5
	T18	0.37	2.11	12.6	Modified 3
	T19	0.34	2.20	12.0	Modified 5
Jurong Formation residual soil	U1	0.52	2.10	19.5	Standard 3
	U4	0.44	2.08	11.7	Standard 3
	U6	0.42	2.10	12.2	Standard 5
	U7	0.40	2.12	11.2	Modified 3
	U8	0.36	2.19	10.9	Modified 5
	U9	0.41	2.19	15.3	Standard 5
	U10	0.39	2.21	15.2	Modified 3
	U11	0.36	2.23	14.0	Modified 5
	U12	0.47	2.01	10.55	Standard 3
	U13	0.44	2.03	9.1	Standard 5
	U14	0.43	2.04	8.8	Modified 3
	U15	0.39	2.09	8.7	Modified 5
	U16	0.45	2.11	14.4	Standard 3
	U17	0.41	2.15	13.4	Standard 5
	U18	0.37	2.19	12.4	Modified 3
	U19	0.36	2.20	11.7	Modified 5

A few ultrasonic tests were also conducted for saturated compacted residual soils under consolidated undrained condition (CU). The shear wave velocity was measured at the end of primary consolidation. The purpose of these tests is to investigate the effect of effective confining pressure on very small strain shear modulus and damping ratio. These test results enable comparison with test results

from the literature which were for saturated soils. Table 6.4 shows the properties of the saturated compacted residual soil specimens tested under CU test condition using ultrasonic test.

Table 6.4 Properties of saturated compacted residual soil specimens for ultrasonic test (CU test condition)

Soil	Specimen Number	e	ρ (Mg/m^3)	w (%)	Remarks
Bukit Timah Granite residual soil	T1A	0.52	2.02	20.1	Standard 3
	T1B	0.44	2.01	12.9	Standard 3
Jurong Formation residual soil	U1A	0.54	2.09	20.1	Standard 3
	U1B	0.46	2.05	12.3	Standard 3

6.3 Confining Pressure

It has been reported that increasing confining pressure increases soil stiffness (Hardin and Richart 1963, Stokoe et al. 1985, Winkler and Murphy 1995, Nakagawa et al. 1997 and Cascante and Santamarina 1996) and decreases attenuation (Winkler and Nur 1979, Prasad and Meissner 1992 and Winkler and Murphy 1995) and damping ratio (Hardin and Drnevich 1972, Tatsuoka et al. 1978 and Vucetic et al. 1998). The application of confining pressure changes the soil stiffness in two ways. Firstly, the pore-air volume reduces once the confining pressure is applied, thus void ratio decreases and stiffness increases. In addition, if drainage is allowed the excess pore-water pressure developed will dissipate and primary consolidation takes place which will further reduce the void ratio and increase the stiffness. Secondly, the bonding between the soil particles is strengthened and the soil particles are rearranged into a more compact structure thus increasing the stiffness.

6.3.1 Initial Shear Modulus G_{max}

Figures 6.3 and 6.4 show the variation of initial shear modulus of compacted Bukit Timah Granite residual soil and Jurong Formation residual soil with total confining pressure and effective confining pressure, respectively. The initial shear modulus increases with both total confining pressure and effective confining pressure as expected and follows the trend suggested by Hardin and Richart (1963), Stokoe et al. (1985) and Nakagawa et al. (1997). The initial shear modulus varies with both total confining pressure and effective confining pressure raised to power b (Equation 2.17). Table 6.5 summarises the exponent b for the specimen. The exponent b varies from 0.08 to 0.32 for UU test condition and from 0.40 to 0.59 for CU test condition. As G_{max} is directly related to void ratio, increasing confining pressure either total confining pressure or effective confining pressure results in a decrease in void ratio and therefore, an increase in G_{max} . Figures 6.3 and 6.4 are similar to the compression or consolidation curve (i.e. void ratio versus effective overburden pressure) reflected about the x-axis. The exponent b is analogous to coefficient of compressibility, a_v .

The variation of exponent b with degree of saturation for test conducted under UU test condition is shown in Figure 6.5. The variation of exponent b (UU test condition) with degree of saturation can be divided into two regions. For degree of saturation greater than 95%, the b values are lower, from 0.08 to 0.18. For degree of saturation less than 95%, the b values are higher ranging between 0.18 and 0.32 and the variation is about 0.02 to 0.06. For soil specimens having high degree of saturation, the compression of pore-air volume due to application of confining pressure is small as the voids are mainly filled with water which is incompressible. Soil specimens with lower degree of saturation have higher air content so the compression of pore-air volume due to application of confining pressure is larger.

The b values (UU test condition) found from bender element test are more scattered and are higher than those obtained from the ultrasonic test. This is attributed to the

different arrangement in transmitter and receiver elements and subsequently the wave travel path. For bender element test, the transmitter and receiver elements were arranged in the centre of the top and bottom platens as shown in Figure 3.2, thus, the shear wave had a narrow travel path. The portion of soil specimen being influenced by the shear wave is relatively small so the variability is more significant. For ultrasonic test, the shear wave transmitter and receiver elements were arranged in a circular layout with an outer diameter of 50 mm in the top and bottom platens as shown in Figure 4.2. Thus, the shear wave travelled through a broader path. The portion of soil specimen being influenced by shear wave is larger and less likely affected by local heterogeneity in the soil specimen. This phenomenon was further verified by the b values obtained for $E_{bulk,max}$ (discussed in Section 6.3.2).

Table 6.5 Exponent b for soil specimens

Test Condition	Specimen Number	b value	Specimen Number	b value	Specimen Number	b value
UU test condition	B1	0.18	U1	0.10	T1	0.10
	B4	0.32	U4	0.22	T4	0.20
	B6	0.25	U6	0.22	T6	0.18
	B7	0.27	U7	0.20	T7	0.23
	B8	0.29	U8	0.20	T8	0.20
	B9	0.29	U9	0.10	T9	0.10
	B10	0.31	U10	0.12	T10	0.12
	B11	0.27	U11	0.12	T11	0.08
	B12	0.25	U12	0.22	T12	0.19
	B13	0.28	U13	0.20	T13	0.18
	B14	0.30	U14	0.20	T14	0.22
	B15	0.28	U15	0.20	T15	0.23
	B16	0.13	U16	0.22	T16	0.20
	B17	0.19	U17	0.22	T17	0.22
	B18	0.19	U18	0.22	T18	0.23
	B19	0.19	U19	0.22	T19	0.23
CU test condition	T1A	0.50	U1A	0.59	-	-
	T1B	0.40	U1B	0.41	-	-

(a) Jurong Formation residual soil - Bender element test

(b) Bukit Timah Granite residual soil - Ultrasonic test

(c) Jurong Formation residual soil – Ultrasonic test

Figure 6.3 Variation of G_{max} with total confining pressure.

Figure 6.4 Variation of G_{max} with effective confining pressure.

Figure 6.5 Variation of b value with degree of saturation for test conducted under UU condition.

The exponent b values (UU test condition) found in this study were less than the typical range of 0.40 to 0.59 obtained for sand, clay and residual soil at the end of primary consolidation reported in Table 2.5. However, the b value (UU test condition) obtained agreed with those obtained for cemented sand as the cemented sand is also unsaturated specimen which does not undergo primary consolidation (i.e. dissipation of pore water pressure). This shows that the effect of pore-air volume compression in the unsaturated condition on G_{max} is not as significant as pore structure compression in the saturated condition due to consolidation for compacted residual soils.

The exponent b values (CU test condition) found in this study ranges from 0.40 to 0.59 which falls within the typical ranges for saturated soils (sand, clay and residual soils) reported in the literature. The more significant change in shear modulus due to consolidation of saturated soil specimen can be explained by the soil particles in the soil specimen undergoing more changes in contact such as slippage and breakage and particle rearrangement (Cascante and Santamarina 1996 and Cho et al. 2004). Exponent b is a function of sphericity, roundness, surface roughness, percentage of platy mica particles and state of assembly (Cascante and Santamarina 1996 and Cho et al. 2004). According to Cho et al. (2004), the exponent b increases with decreasing sphericity and roundness. The exponent b also increases with increasing surface roughness and percentage of platy mica particles (Santamarina and Cascante 1998 and Guimaraes 2002). Thus, the value of exponent b is not a constant (Rampello and Viggiani 2001 and Cho et al. 2004).

6.3.2 Initial Bulk Modulus $E_{bulk,max}$

Figure 6.6 shows the variation of initial bulk modulus of compacted Bukit Timah Granite residual soil and Jurong Formation residual soil with total confining pressure. It is found that the initial bulk modulus increases with total confining pressure. A similar relationship as that of G_{max} with confining pressure (Equation 2.17) is obtained. The initial bulk modulus increases in a manner similar to that

observed by Wilson and Miller (1962), Hardin and Richart (1963) and Nakagawa et al. (1997). The initial bulk modulus varies approximately with confining pressure raised to power b' . Table 6.6 summarises the exponent b' (UU test condition) obtained for the specimens. The exponent b' (UU test condition) ranges from 0.0 to 0.32. The variation of b' value with degree of saturation can be divided into two regions as shown in Figure 6.7. For degree of saturation greater than 95%, the b' value is low (i.e. 0.06 – 0.08). This is because the application of confining pressure does not cause significant reduction of pore-air volume as the void is filled with water which is incompressible and no drainage was allowed. For degree of saturation less than 95%, the b' value is higher because the compression of pore-air volume is larger since it has higher air content. The higher scatter in b' value is due to the arrangement of transmitter and receiver elements as explained in Section 6.3.1.

Table 6.6 Exponent b' for soil specimens

Test Condition	Specimen Number	b' value	Specimen Number	b' value
UU test condition	U1	0.06	T1	0.07
	U4	0.30	T4	0.17
	U6	0.26	T6	0.19
	U7	0.32	T7	0.21
	U8	0.22	T8	0.22
	U9	0.06	T9	0.08
	U10	0.06	T10	0.07
	U11	0.06	T11	0.08
	U12	0.22	T12	0.20
	U13	0.22	T13	0.28
	U14	0.22	T14	0.20
	U15	0.28	T15	0.26
	U16	0.22	T16	0.23
	U17	0.22	T17	0.19
	U18	0.30	T18	0.23
	U19	0.22	T19	0.20

(a) Bukit Timah Granite residual soil – Ultrasonic test

(b) Jurong Formation residual soil – Ultrasonic test

Figure 6.6 Variation of $E_{bulk,max}$ with total confining pressure.

Figure 6.7 Variation of b' value with degree of saturation for test conducted under UU condition.

6.3.3 Damping ratio of Compression and Shear Waves

The variation of damping ratio with total confining pressure and effective confining pressure for Bukit Timah Granite and Jurong Formation residual soils are shown in Figures 6.8 and 6.9, respectively. The damping ratios were obtained from the attenuation coefficient (Equation 6.1). All the specimens showed that damping ratio decreases with both total confining pressure and effective confining pressure. Based on Figure 6.8, compression wave damping ratio was found to be larger than the shear wave damping ratio. Similar observations for compression and shear wave attenuation (i.e. Q^{-1}) on Massilon sandstone with different degrees of saturation (dry, unsaturated ~95% and fully saturated) have been obtained by Winkler and Nur (1979) as shown in Figure 6.10. In Figure 6.10, attenuation is expressed in terms of a quality factor Q^{-1} which is also related to the attenuation coefficient α and damping ratio (Toksöz and Johnston 1981, Selfridge 1985):

$$\frac{1}{Q} = \frac{\alpha V}{\pi f} \quad (6.2)$$

$$\frac{1}{Q} = \frac{2D_{amp}}{\sqrt{1 - D_{amp}^2}} \tag{6.3}$$

where V is wave velocity and f is frequency.

(a) Bukit Timah Granite residual soil

(b) Jurong Formation residual soil

Figure 6.8 Variation of damping ratio with total confining pressure (UU test condition).

Figure 6.9 Variation of damping ratio with effective confining pressure (CU test condition).

(a) Compression wave

(b) Shear wave

Figure 6.10 Variation of attenuation with confining pressure (Winkler and Nur 1979).

The damping ratios obtained for tests conducted under UU condition are generally larger than those obtained reported in the literature. The main reasons attributed to the difference are listed below:

- i) Damping ratio is affected by the degree of saturation and void ratio. The soil specimen tested under UU test condition is unsaturated. The contribution of air void towards damping is significant as compared to those in the literature which is generally saturated specimen without air void. This point is further verified in Figure 6.9. Figure 6.9 shows the variation of damping ratio for shear wave with effective confining pressure tested under CU condition. The soil specimen tested under CU condition is saturated, thus, the contribution of air void towards damping is eliminated and results in a lower damping ratio. A similar finding was shown by Winkler and Nur (1979) for Massillon sandstone and Nakagawa et al. (1997) for Monterey sand. They had shown that the attenuation of unsaturated materials is about 1.5 to 40 times higher than attenuation of saturated material. And, the attenuation for compression wave in unsaturated materials is more significant than shear wave. The damping ratios of unsaturated materials ranges from 0.005 to 0.5 (Winkler and Nur 1979 and Nakagawa et al. 1997). The damping ratio for unsaturated residual soil specimens obtained in this study generally falls within the range reported.
- ii) The assembly of the soil particles in the soil specimen also affects the damping ratio. A loose state soil specimen with simple cubic packing will have a higher damping ratio than the denser one with tetrahedral or pyramidal packing (Cascante and Santamarina 1996). This point can also be illustrated by the comparison of the damping ratio in both Figures 6.8 and 6.9. The difference in the damping ratio in both Figures 6.8 and 6.9 is not only attributed to (i) but also the different in particles arrangement. The soil specimen used to obtain the result in Figure 6.9 had undergone consolidation (i.e. dissipation of excess pore water pressure), thus, the soil particles undergoes a change in contact and particle arrangement which results in different damping ratio as compared to results in Figure 6.8.

- iii) The damping ratio is also a function of strain rate or frequency (He 1998 and Santucci de Magistris 1999). The ultrasonic test was conducted at frequency of 76 kHz for shear wave and 90 kHz for compression wave. The testing frequency is generally a few orders higher than those used in the literature. The damping ratio reported in the literature is generally for frequency less than 1Hz. Thus, the damping ratio obtained is different.

6.4 Void Ratio

Both initial bulk modulus and initial shear modulus depend on void ratio of the soil. Increase in void ratio results in decrease of initial bulk and shear modulus. This is because an increase in void ratio decreases the compression and shearing resistance and subsequently reduce the bulk modulus and shear modulus, respectively.

Figure 6.11 shows the variation of compression and shear wave velocities with void ratio. It can be seen that the compression and shear wave velocities increase with decreasing void ratio except compression wave velocity of soils with degree of saturation close to 100%. Therefore, both initial bulk modulus and initial shear modulus increase as the void ratio decreases since they are related to compression and shear wave velocity, respectively. The compression wave velocity for specimens close to full saturation remains almost constant because the compression wave velocity measured is the compression wave velocity of water (i.e. $\cong 1500$ m/s). For pulse transmission technique, the wave tends to travel through the stiffer portion of the specimen. Water being incompressible has a very high bulk modulus compared to soil, thus, the wave tends to travel through the voids which are filled with water in the fully saturated specimen. Therefore, the measured compression wave velocity of the soil at this state does not reflect the compression wave velocity of the soil.

It was also observed that the compression and shear wave velocities vary linearly with void ratio. The results agree with the findings of Hardin and Richart (1963) and Stephenson (1978). The relationships between compression and shear wave velocity of residual soil with void ratio are summarised in Table 6.7.

From Figure 6.11a, it can be seen that the shear wave velocities obtained by bender element and ultrasonic soil testing system are comparable, the difference is less than 5%. This small discrepancy is attributed to the difference in system attenuation. Attenuation of the system depends on the D/λ_w ratio (Krautkrämer and Krautkrämer 1990). Attenuation increases as D/λ_w ratio decreases. The diameter of the soil specimen for bender element test is 50 mm while the diameter of the soil specimen for ultrasonic test is 70 mm. Thus, the attenuation in bender element test is more than that of the ultrasonic test as the D/λ_w ratio for bender element test is lower than that of ultrasonic test. This has caused the slight discrepancy in the determination of the arrival time.

Table 6.7 Relationship of wave velocity with void ratio

Soil	Wave	S (%)	Relationship of wave velocity with void ratio
Bukit Timah Granite residual soil	Compression wave	$60 \leq S \leq 65$	$V_p = -1584e + 1197$ (m/s)
		$70 \leq S \leq 80$	$V_p = -2314e + 1673$ (m/s)
		$85 \leq S \leq 95$	$V_p = -2697e + 1953$ (m/s)
		$S \approx 100$	$V_p = 1380$ (m/s)
	Shear wave	$60 \leq S \leq 100$	$V_s = -1512e + 1006$ (m/s) Ultrasonic test
Jurong Formation residual soil	Compression wave	$55 \leq S \leq 65$	$V_p = -3828e + 2478$ (m/s)
		$70 \leq S \leq 80$	$V_p = -4203e + 2771$ (m/s)
		$85 \leq S \leq 90$	$V_p = -3984e + 2737$ (m/s)
		$S \approx 100$	$V_p = 1360$ (m/s)
	Shear wave	$55 \leq S \leq 100$	$V_s = -1865e + 1299$ (m/s) Bender element test $V_s = -1612e + 1182$ (m/s) Ultrasonic test

(a) Shear wave velocity

(b) Compression wave velocity – Bukit Timah Granite residual soil

(c) Compression wave velocity –Jurong Formation residual soil

Figure 6.11 Variation of shear and compression wave velocities with void ratio.

Very little research has been done on the relationship between damping ratio at very small strain ($\gamma < 0.0005\%$) and void ratio for both compression and shear waves. Dobry and Vucetic (1987) found that the damping ratio for shear wave decreases as the void ratio increases. However, Hsu (2002) found that void ratio has very little effect on damping ratio for shear wave. To the author's knowledge, there is no published data on the damping ratio at strain smaller than the linear cyclic threshold strain for compression wave of soil. Figures 6.12 and 6.13 show the variation of the very small strain damping ratio from ultrasonic test with void ratio for both compression and shear waves for Bukit Timah Granite and Jurong Formation residual soils, respectively. The damping ratio for compression and shear waves decreases as void ratio increases for both Bukit Timah Granite and Jurong Formation residual soils. This finding is similar to that reported by Dobry and Vucetic (1987) for shear wave. The relationships between the very small strain damping ratio and void ratio are summarized in Table 6.8.

Table 6.8 Relationship of damping ratio at very small strain and void ratio

Soil	Wave	S (%)	Relationship of damping ratio with void ratio
Bukit Timah Granite residual soil	Compression wave	$60 \leq S \leq 65$	$D_{amp} = -0.30e + 0.24$
		$70 \leq S \leq 80$	$D_{amp} = -0.70e + 0.48$
		$85 \leq S \leq 95$	$D_{amp} = -0.50e + 0.43$
		$S \approx 100$	$D_{amp} = -0.24e + 0.54$
	Shear wave	$60 \leq S \leq 65$	$D_{amp} = -0.34e + 0.21$
		$70 \leq S \leq 80$	$D_{amp} = -0.28e + 0.20$
		$85 \leq S \leq 95$	$D_{amp} = -0.12e + 0.14$
		$S \approx 100$	$D_{amp} = -0.52e + 0.34$
Jurong Formation residual soil	Compression wave	$55 \leq S \leq 65$	$D_{amp} = -0.64e + 0.55$
		$70 \leq S \leq 80$	$D_{amp} = -2.14e + 1.22$
		$85 \leq S \leq 90$	$D_{amp} = -1.47e + 0.96$
		$S \approx 100$	$D_{amp} = -0.43e + 0.69$
	Shear wave	$55 \leq S \leq 65$	$D_{amp} = -1.83e + 0.90$
		$70 \leq S \leq 80$	$D_{amp} = -1.67e + 0.78$
		$85 \leq S \leq 90$	$D_{amp} = -0.89e + 0.48$
		$S \approx 100$	$D_{amp} = -0.44e + 0.34$

(a) Compression wave

(b) Shear wave

Figure 6.12 Variation of very small strain damping ratio for Bukit Timah Granite residual soil.

(a) Compression wave

(b) Shear wave

Figure 6.13 Variation of very small strain damping ratio with void ratio for Jurong Formation residual soil.

6.5 Degree of Saturation

It can be seen from Figure 6.11a that the initial shear modulus is independent of degree of saturation. The initial shear modulus depends only on the properties of the soil skeleton as pore-water and pore-air have no shear resistance. However, compression wave velocity varies with degree of saturation (Figures 6.11b and 6.11c). The relationship between compression wave velocity and void ratio changes as degree of saturation changes (Table 6.7). Thus, the initial bulk modulus varies with degree of saturation. To investigate the relationship between the compression wave velocities and degree of saturation, vertical lines at various void ratios (0.35, 0.40 and 0.45) are drawn through the various trend lines to obtain the compression wave velocities at different degree of saturation. The compression wave velocities thus obtained for both Bukit Timah Granite and Jurong Formation residual soils at void ratios of 0.35, 0.40 and 0.45 are plotted with degree of saturation in Figure 6.14. The compression wave velocity increases as degree of saturation increases and approaches the compression wave velocity of water (1500 m/s) as the degree of saturation approaches 100%. This finding is similar to the findings of Allen et al. (1980) who conducted tests on sand for degree of saturation greater than 99%.

Figure 6.14 Variation of compression wave velocity with degree of saturation.

There is very little research done on the variation of very small strain damping ratio for both compression and shear wave with degree of saturation. Winkler and Nur (1979) found that very small strain damping ratio compression wave first increases and then decreases with increasing degree of saturation for Massillon sandstone. However, the very small strain damping ratio for shear wave increases with increasing degree of saturation for Massillon sandstone. Hsu (2002) investigated the effect of degree of saturation on damping ratio of sand and clay for shear wave. He found that degree of saturation does not seem to have any effect on damping ratio of sand and clay for shear wave.

From Figures 6.12 and 6.13, it can be seen that the very small strain damping ratio for both compression and shear waves depends on degree of saturation. The relationship of the damping ratio and void ratio changes as degree of saturation changes (Table 6.8). To investigate the relationship between the damping ratio and degree of saturation, vertical lines at void ratio of 0.35, 0.40 and 0.45 are drawn through the various trend lines for different ranges of degree of saturation to obtain the corresponding damping ratio. The damping ratio thus obtained for both Bukit Timah Granite and Jurong Formation residual soils are plotted with degree of saturation in Figure 6.15. In general, the very small strain damping ratio increases as degree of saturation increases. However, there are cases in which the very small strain damping ratio first decreases and then increases with increasing degree of saturation. There is insufficient data to give an exact conclusion. However, this series of tests served as preliminary test results for further comprehensive study on the effect of degree of saturation on damping ratio.

(a) Compression wave

(b) Shear wave

Figure 6.15 Variation of damping ratio with degree of saturation.

6.6 Relationship of Compression Wave Velocity with G_{max}

Soil is multi-phase consisting of an assemblage of solid particles (soil skeleton), water and air. In the literature, it was found that compression wave velocity for soil

is not only a function of G_{max} but also a function of density, porosity, degree of saturation, size of pores, solubility of pore air, shape of pores, shape of solid particles, grain size distribution, orientation of solid particles, composition of disperse and continuous phase and their bonding. In order to model the propagation of compression wave in soil, various assumptions are made to simplify the model. Biot (1956a, b) developed a theory to describe the propagation of compression wave in a porous solid. According to Biot (1956a, b), the propagation of compression wave is governed by the characteristic frequency, f' :

$$f' = \frac{ng\rho_w}{2\pi k\rho_f} \quad (6.4)$$

where n is the porosity, g is the acceleration of gravity, k is the permeability, ρ_w is the density of water and ρ_f is the density of the fluid which is the combined density of water and air (i.e. $\rho_f = S\rho_w + (1-S)\rho_a$) and ρ_a is the density of air. Using typical values of n and k for gravel, sand, silt and clay, and assuming S ranges from 0 to 100%, the typical values for the characteristic frequency, f' , of various soil types can be obtained using Equation 6.4 as summarized in Table 6.9.

Below the characteristic frequency, the relative motion of the water in the pores is of the Poiseuille type and the flow velocity profile is linear. Above the characteristic frequency, Poiseuille type flow breaks down and the flow velocity profile is no longer linear. Biot (1956a, b) suggested that there are two kinds of compression waves and one shear wave. The compression wave with a higher velocity is termed first kind compression wave while the slower compression wave is termed second kind compression wave. In general, the first kind compression wave velocity and shear wave velocity are usually measured experimentally as the frequency at which the measurement made is generally below the characteristic frequency, f' , for pulse transmission tests. Magnitude of the second kind compression wave is insignificant for frequency below f' (Chen 1991 and Miura et al. 2001) and is masked by the stronger first kind compression wave. The author will concentrate only on the

propagation of the first kind compression wave at frequency below the characteristic frequency, f' , in soil.

Table 6.9 Range of characteristic frequency for different types of soil

Soil	n	k (m/s)	ρ_f (kg/m ³)	f' (Hz)
Gravel	0.12-0.46	$1-1 \times 10^2$	0.1-1000	$2 \times 10^{-3}-7 \times 10^3$
Sand	0.33-0.44	$1 \times 10^{-4}-1$	0.1-1000	$5 \times 10^{-1}-7 \times 10^7$
Silt	0.29-0.52	$1 \times 10^{-7}-1 \times 10^{-4}$	0.1-1000	$5 \times 10^3-9 \times 10^9$
Clay	0.29-0.67	$1 \times 10^{-9}-1 \times 10^{-4}$	0.1-1000	$5 \times 10^3-1 \times 10^{11}$

Biot's theory has been confirmed and is well accepted, however, practical application is not easy since the physical interpretation of the mathematics and the constants are difficult. Various compression wave velocity models have been put forward since Biot published his paper in 1956. These models attempt to simplify or modify Biot's compression wave equations by making various assumptions to account for the various factors that influence the propagation of the first kind compression wave. In general, the models are based on the following assumptions:

- i) Soil is statically isotropic and all voids are continuous.
- ii) Soil skeleton is elastic and solid particles are incompressible.
- iii) Water is incompressible and Darcy's law is valid.

The approach adopted by different researchers varies from one to another. The most popular approaches used are meso-structural (i.e. material engineering) and macro-structural (i.e. structural engineering) levels. Meso-structural level involves the pores, cracks and inclusions to the soil. Macro-structural level involves the use of structural element. In the meso-structural and macro-structural approaches, the complexities in the particles arrangement and displacement, and characteristics of pores are avoided (Yaman et al. 2000). However, this has formed the main deficiency of these models in describing the propagation of compression wave in soil or similar porous material. In this section, the various models are discussed in terms of their deficiencies. One of the models is modified to address the micro-structural aspect to describe the propagation of compression wave.

6.6.1 Review of Existing Compression Wave Velocity Models

6.6.1.1 Soil is Treated as Single Phase Consisting of Solid Particles Only

In this case, the soil is assumed to be homogenous, infinite and the interaction between the soil skeleton with water and air is neglected. Thus, the propagation of compression wave in soil is governed by the solid particles only. This condition is generally found only in dry soil. The expression for compression wave velocity is obtained by considering the stress equilibrium of a soil element. This is a macro-structural level approach and a detailed derivation can be found in any soil dynamics text books (Das 1993). The equation of motion for wave propagation in soil treated as single phase is given by

$$\rho \frac{\partial^2 \bar{\varepsilon}}{\partial t^2} = \left(K_s + \frac{4G_{\max}}{3} \right) \nabla^2 \bar{\varepsilon} \quad (6.5)$$

where $\bar{\varepsilon}$ is the sum of the axial strains in x , y and z directions (i.e. $\varepsilon_x + \varepsilon_y + \varepsilon_z$),

$K_s = \frac{2G_{\max}(1+\nu)}{3(1-2\nu)}$ is the bulk modulus of soil skeleton, ρ is the bulk density of soil

$$\text{and } \nabla^2 = \left(\frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2} \right).$$

Therefore, the compression wave, V_p , is given by

$$V_p = \sqrt{\frac{K_s + \frac{4}{3}G_{\max}}{\rho}} \quad (6.6)$$

Equation 6.6 does not vary with degree of saturation (Figure 6.16) and is only applicable to dry soil.

Figure 6.16 Variation of compression wave velocities with degree of saturation for different models.

6.6.1.2 Soil is Treated as a System of Springs Connected in Series

In this case, the soil skeleton, water and air phases are treated as three different springs connected in series (Figure 6.17a). This model is known as the Reuss model. This is one of the meso-structural level approaches. The strain experienced by the soil is the total strain experienced by the soil skeleton, water and air. This condition is generally found in highly porous materials.

According to this model, the total strain experienced by the soil, ϵ_{soil} , is

$$\epsilon_{soil} = \epsilon_{soil \text{ skeleton}} + \epsilon_{water} + \epsilon_{air} \quad (6.7)$$

where $\epsilon_{soil \text{ skeleton}}$ is the strain experienced by soil skeleton, ϵ_{water} is the strain experienced by water and ϵ_{air} is the strain experienced by air. Using theory of elasticity, the strains can be related to their respective bulk modulus to give

$$\frac{\sigma_{total}}{E_{bulk \text{ soil}}} = \frac{\sigma_{total}}{K_s + \frac{4}{3}G_{max}} + \frac{nS\sigma_{total}}{K_w} + \frac{n(1-S)\sigma_{total}}{K_a} \quad (6.8)$$

where σ_{total} is the total stress experienced by soil, n is the porosity, S is the degree of saturation, $E_{bulk\ soil}$ is the bulk modulus of soil, K_w is the bulk modulus of water and K_a is the bulk modulus of air.

By rearranging Equation 6.8,

$$E_{bulk\ soil} = \frac{1}{\frac{1}{K_s + \frac{4}{3}G_{max}} + \frac{nS}{K_w} + \frac{n(1-S)}{K_a}} \quad (6.9)$$

The compression wave velocity is given by

$$V_p = \sqrt{\frac{E_{bulk\ soil}}{\rho}} \quad (6.10)$$

Therefore,

$$V_p = \sqrt{\frac{\frac{1}{\frac{1}{K_s + \frac{4}{3}G_{max}} + \frac{nS}{K_w} + \frac{n(1-S)}{K_a}}}{\rho}} \quad (6.11)$$

The variation of compression wave velocity given by Equation 6.11 with degree of saturation is plotted in Figure 6.16. The compression wave velocity obtained from Equation 6.11 increases with degree of saturation, however, the compression wave velocity at degree of saturation less than 99% is very low, well below the compression wave velocity of air (=331 m/s). The compression wave velocity shown in Figure 6.16 is for a material that is softer than air. However, it is impossible as in general soil consists of more than 50% of solid particles. The compression wave velocity for soil should not be less than compression wave velocity of air. In addition, the compression wave velocity at degree of saturation greater than 99% is not reasonable as it is well below the compression wave

velocity of water at full saturation (i.e. 770 m/s). This contradicts the experimental data and observation made by Nakagawa et al. (1996, 1997) which showed that compression wave velocity for both saturated sand and clay approaches compression wave velocity of water (i.e. 1500 m/s). Thus, Equation 6.11 fails to describe propagation of compression wave in soil properly as the soil skeleton, water and air does not arrange themselves layer by layer, the actual arrangement is far more complicated as shown in Figure 6.17b. The soil skeleton, water and air is randomly distributed which results in non-uniform pore size and different shape of pores, thus, the soil will not behave like springs connected in series.

(a) Reuss model - springs connected in series

(b) Arrangement of solid particles, water and air in soil

Figure 6.17 Schematic representation of Reuss model and soil element.

6.6.1.3 Soil is Treated as a System of Springs Connected in Parallel

In this case, the soil skeleton, water and air phases are combined together to act as three springs connected in parallel as shown in Figure 6.18 (Chang et al. 1995). This model is commonly known as the Voigt model. This is also a meso-structural level approach. The strains experienced by the soil skeleton, water and air are equal to each other.

$$\varepsilon_{soil} = \varepsilon_{soil \text{ skeleton}} = \varepsilon_{water} = \varepsilon_{air} \quad (6.12)$$

Using theory of elasticity,

$$\begin{aligned} \sigma_{total} &= \varepsilon_{soil} E_{bulk \text{ soil}} \\ &= \left[K_s + \frac{4}{3} G_{max} \right] \varepsilon_{soil \text{ skeleton}} + nSK_w \varepsilon_{water} + n(1-S)K_a \varepsilon_{air} \end{aligned} \quad (6.13)$$

Thus, the expression for compression wave velocity is:

$$V_p = \sqrt{\frac{K_s + \frac{4}{3} G_{max} + nSK_w + (1-S)nK_a}{\rho}} \quad (6.14)$$

The compression wave velocity obtained from Equation 6.14 is plotted with degree of saturation in Figure 6.16. The compression wave velocity increases with degree of saturation in a linear manner. This contradicts the trend obtained by Allen et al. (1980) which shows that the compression wave velocity increases non-linearly with degree of saturation and approaches compression wave velocity of water when the soil is fully saturated. Furthermore, the compression wave velocity (900 m/s) when the soil is fully saturated is also well below the compression wave velocity of water. Thus, Equation 6.14 fails to describe the propagation of compression wave in soil as

the assumption that soil skeleton, water and air are grouped together in different portions as shown in Figure 6.18 is too simplistic.

Figure 6.18 Voigt model – springs connected in parallel

6.6.1.4 Soil is Treated as a System of Spring Connected in Parallel where Air Phase is not Fully Surrounded by Water

In this case, the air phase is continuous but the water phase is not continuous, i.e. this kind of condition is normally found in unsaturated soil with degree of saturation less than 80% (Fredlund and Rahardjo 1993). The air phase can be in direct contact with solid particles or water. The strain experienced individually by each of the phases is the same.

$$\varepsilon_{soil} = \varepsilon_{soil \text{ skeleton}} = \varepsilon_{water} = \varepsilon_{air} \quad (6.15)$$

The strain experienced by the soil, ε_{soil} , is related to the total stress, σ_{total} , according to Bishop's equation (1959).

$$\begin{aligned}\sigma_{total} &= \sigma' + \chi u_w + (1 - \chi) u_a \\ &= E_{bulk\ soil} \varepsilon_{soil} \\ &= \left(K_s + \frac{4}{3} G_{max} \right) \varepsilon_{soil\ skeleton} + \chi \frac{K_w}{nS} \varepsilon_{water} + (1 - \chi) \frac{K_a}{n(1 - S)} \varepsilon_{air}\end{aligned}\quad (6.16)$$

where χ is the parameter related to the degree of saturation, σ' is the effective stress experienced by the soil skeleton, u_w is the pore-water pressure and u_a is the pore-air pressure. The magnitude of χ is unity for saturated soil and zero for dry soil. The relationship between χ and degree of saturation was obtained experimentally and shown in Figure 6.19.

The compression wave velocity is given by

$$V_p = \sqrt{\frac{K_s + \frac{4}{3} G_{max} + \frac{\left[\chi K_w / S + (1 - \chi) K_a / (1 - S) \right]}{n}}{\rho}} \quad (6.17)$$

A detail derivation of Equation 6.17 can be found in Xu (1997). The compression wave velocity obtained using Equation 6.17 increases with degree of saturation as shown in Figure 6.16. However, the compression wave velocity exceeds the compression wave velocity of water (1500 m/s) when the degree of saturation is high ($S > 80\%$). The compression wave velocity at full saturation is about 2000 m/s which is very much higher than the compression wave velocity of water. In addition, the χ function in Equation 6.17 is not unique and depends on soil types. The χ function used to obtain the results in Figure 6.16 is curve 4 of Figure 6.19. Thus, Equation 6.17 does not describe the propagation of compression wave velocity adequately.

Figure 6.19 Relationship of χ with degree of saturation (from Blight 1961).

6.6.1.5 Soil is Treated as a System of Spring Connected in Parallel with An Occluded Air Phase

In this case, the air phase is fully surrounded by water which means the air phase is discontinuous and water phase is continuous. Thus, the strains experienced by the soil skeleton and the combined water and air phases are equal, i.e.

$$\varepsilon_{\text{soil}} = \varepsilon_{\text{soil skeleton}} = \varepsilon_{\text{water}} + \varepsilon_{\text{air}} = \varepsilon_f \quad (6.18)$$

where ε_f is the strain experienced by fluid phases.

Occluded air phase commonly occur in unsaturated soil having a degree of saturation greater than 80% to 90% (Fredlund and Rahardjo 1993). This is a macro-structural level approach. However, it is also a partial meso-structural level approach in which the water and air can be assumed as springs connected in series. Koning (1963) obtained a similar relationship between the bulk modulus of water and air using Boyle's law. A detail derivation is given in Xu (1997) and in Miura et al. (2001).

By treating the soil skeleton as one phase and the combined water and air phases as a single phase (fluid phase), Equation 6.18 can be written in terms of stress and bulk modulus as

$$\frac{\sigma_f}{K_f} = \frac{S\sigma_f}{K_w} + \frac{(1-S)\sigma_f}{K_a} \quad (6.18)$$

The total stress, σ_{total} , can be written in terms of an effective stress, σ' , and fluid stress, σ_f .

$$\begin{aligned} \sigma_{total} &= \sigma' + \sigma_f \\ &= E_{bulk \ soil} \varepsilon_{soil} \\ &= \left(K_s + \frac{4}{3} G_{max} \right) \varepsilon_{soil \ skeleton} + \frac{K_f \varepsilon_f}{n} \end{aligned} \quad (6.19)$$

where ε_f is the fluid strain for the combined water and air phases and K_f is the combined bulk modulus of water and air phases. The compression wave velocity can be expressed as follows:

$$V_p = \sqrt{\frac{K_s + \frac{4}{3} G_{max} + \frac{K_f}{n}}{\rho}} \quad (6.20)$$

The compression wave velocity obtained using Equation 6.20 shows reasonable trend with observation found by other researchers (Stephenson 1978 and Allen et al. 1980). However, Equation 6.20 gives compression wave velocity of 2000 m/s at full saturation as shown in Figure 6.16. The weakness of Equation 6.20 lies in the expression for combined bulk modulus of water and air phases, K_f . The K_f is obtained without consideration of the characteristics of the soil voids such as size, shape and solubility of air. These factors are negligible only if the pore air has an average diameter of greater than 0.06 mm. Liv and Brennen (1998) found that pore air size of less than 0.06 mm is present in the water. Researchers have suggested different expressions for K_f to account for solubility of pore air, surface tension and size of the pore air (e.g. Tamura et al. 2002).

Schuurman (1966) derived an expression for K_f based on the following assumptions:

- i) Boyle's law is obeyed in which the product of pressure and volume of air is constant.
- ii) Henry's law is obeyed in which the weight of the air dissolved in water is proportional to the air pressure.
- iii) Air exists as bubbles and all the bubbles have the same diameter.
- iv) Time necessary to dissolve the air can be disregarded.
- v) Compressibility of water is disregarded.

According to Schuurman (1966), the modified combined bulk modulus of water and air, K_f^* , is given as:

$$K_f^* = (V_a + V_w) \left\{ \frac{(V_{a0} + V_{as})P_{a0}}{(V_a + V_{as})^2} - \frac{4T_i}{3d_0V_a} \left(\frac{V_{a0}}{V_a} \right)^{1/3} \right\} \quad (6.21)$$

where V_a and V_w are the volume of the air and water, respectively, V_{a0} is the volume of air at atmospheric pressure, P_{a0} is the atmospheric pressure ($= 100$ kPa), T_i is the surface tension ($= 72.8 \times 10^{-3}$ N/m), d_0 is the diameter of the pore-air at atmospheric pressure and V_{as} is the volume of air dissolved in water in which $V_{as} = H_s V_w$ and H_s is the coefficient of solubility ($= 0.02$ at S.T.P.).

If $V_a = V_{a0}$ and $d_0 = 0.01$ mm, Equation 6.21 becomes

$$K_f^* = \frac{200000}{1 - S + HS} - \frac{9707}{1 - S} \quad (6.22)$$

Using K_f^* , the compression wave velocity can be expressed as follows:

$$V_p = \sqrt{\frac{K_s + \frac{4}{3}G_{\max} + \frac{K_f^*}{n}}{\rho}} \quad (6.23)$$

The variation of compression wave velocity obtained from Equation 6.23 with degree of saturation is shown in Figure 6.20. Equation 6.23 gives compression wave velocity very similar to Equation 6.6 but decreases when the degree of saturation is greater than 99%. Therefore, Schuurman's expression of K_f^* is not appropriate although some characteristics of pore air is incorporated. Schuurmann's expression cannot simulate the condition when the pore air is almost dissolved in the water (i.e. $S > 99.9\%$) as solubility of pore air changes with the size of pore air. If the size of the pore air is very small, the pore air takes a longer time to dissolve into the water, the coefficient of solubility, H , decreases and becomes very small. If the pore air size is very small, V_{as} is no longer proportional to V_w (Tamura et al. 2002). This implies that the solubility of pore air is a non-linear function of pore air size which has been neglected in Schuurmann's expression.

Figure 6.20 Variation of compression wave velocities with degree of saturation for different models incorporating the characteristics of pore-air.

Akagawa (1974) derived another expression for K_f with the aid of the gas-liquid two phase flow theory using the following assumptions:

- i) The momentum of water and air are conserved at the wavefront of compression wave.

- ii) The mass of water and air is conserved at the wave front of compression wave.
- iii) The change in unit weight with pressure is assumed to be isotropic.
- iv) The air does not dissolve in the water with compression wave propagation.
- v) The air exists as bubbles and all the air bubbles have the same diameter.

The modified combined bulk modulus of water and air, K_f^{**} , by Akagawa (1974) is given by

$$K_f^{**} = \frac{\rho_f v_a^2}{\left\{ (1-S)^2 + (1-S)S \frac{\rho_w}{\rho_a} + \frac{v_a^2}{v_w^2} S \left[1 - (1-S) \left(1 - \frac{\rho_a}{\rho_w} \right) \right] \right\}} \quad (6.24)$$

where v_a and v_w are the compression wave velocity of air and water, respectively, ρ_a and ρ_w are density of air and water, respectively and ρ_f is the density of the combined water and air phase (i.e. $\rho_f = S\rho_w + (1-S)\rho_a$).

Using K_f^{**} and poroelastic theory, Tamura et al. (2002) derived an expression for compression wave velocity:

$$V_p = \sqrt{\frac{n\rho_d \left(V_{pd}^2 - \frac{4}{3} V_{sd}^2 \right) \left(1 - \frac{K_f^{**}}{K_{sp}} \right) + \left[1 - \frac{\rho_d \left(V_{pd}^2 - \frac{4}{3} V_{sd}^2 \right)}{K_{sp}} \right] K_f^{**}}{n \left(1 - \frac{K_f^{**}}{K_{sp}} \right) + \frac{K_f^{**}}{K_{sp}} \left(1 - \frac{\rho_d \left(V_{pd}^2 - \frac{4}{3} V_{sd}^2 \right)}{K_{sp}} \right)} + \frac{4}{3} G_{\max}} \quad (6.25)$$

where ρ_d is the density of the soil skeleton at dry state, V_{pd} and V_{sd} are the compression and shear wave velocities of the soil skeleton at dry state, and K_{sp} is the bulk modulus of solid particles.

The weakness of Equation 6.25 is it needs more input parameters namely, bulk modulus of solid particles, K_{sp} , and compression wave velocity of soil skeleton at dry state, V_{pd} . Thus, Equation 6.25 is not applicable as compression wave velocity of soil skeleton at dry state is not known.

Comparing Equations 6.20, 6.23 and 6.25, a more practical form of compression wave velocity equation should treat the soil as a system of spring connected in parallel with an occluded air phase and a rigorous expression for the combined bulk modulus of water and air. The modified combined bulk modulus of water and air, K_f^{**} (Equation 6.24) is more rigorous than K_f (Equation 6.18) and K_f^* (Equation 6.22) as it involves degree of saturation to order of two. Thus, the following form of compression wave velocity equation is suggested:

$$V_p = \sqrt{\frac{K_s + \frac{4}{3}G_{\max} + \frac{K_f^{**}}{n}}{\rho}} \quad (6.26)$$

However, Tamura et al. (2002) had shown that porosity of soil, n , has little effect on compression wave velocity compared to the characteristics of pore-air and bulk modulus of soil skeleton. Therefore Equation 6.26 is simplified to

$$V_p = \sqrt{\frac{K_s + \frac{4}{3}G_{\max} + K_f^{**}}{\rho}} \quad (6.27)$$

The variation of compression wave velocity obtained using Equation 6.27 with degree of saturation is shown in Figure 6.20. The compression wave velocity increases with degree of saturation and approaches compression wave velocity of water. The compression wave velocity from Equation 6.27 is about 1300 m/s at full saturation. Equation 6.27 appears promising and will be investigated using the experimental data obtained in Section 6.4.

6.6.2 Verification of Equation 6.27

The compression wave velocities obtained using Equation 6.27 and the compression wave velocities obtained from ultrasonic test (Figure 6.14) are plotted together in Figure 6.21. There is discrepancy in the calculated and experimental compression wave velocity especially for degree of saturation in the range of 70% to 90%. Equation 6.27 assumed that the pore air is spherical in shape and has only one size which is not true. According to Yaman et al. (2000), the compression wave velocity for concrete is not the same under dry and wet condition at the same porosity. This is because the pores are interconnected, creating needle shaped long narrow channel under dry condition as compared to the circular or nearly spherical shaped pores under wet condition. Modification is made to K_f^{**} (Equation 6.24) to account for the shape of pore air and the pore air size distribution to provide a more reasonable estimation of the combined bulk modulus of water and air phases. A correction for the condition of pores, C_s , as a function of degree of saturation is introduced into Equation 6.24 as follows:

$$K_f^{***} = \frac{\rho_f v_a^2}{C_s \left\{ (1-S)^2 + (1-S)S \frac{\rho_w}{\rho_a} + \frac{v_a^2}{v_w^2} S \left[1 - (1-S) \left(1 - \frac{\rho_a}{\rho_w} \right) \right] \right\}} \quad (6.28)$$

where C_s is greater than zero but less than or equal to one.

The relationship between C_s and S unlike χ should be independent of soil type. Therefore, the relationship between C_s and S is established using the experimental data of different soil types from Bukit Timah Granite residual soil, Jurong Formation residual soil, kaolin (Nakagawa et al. 1996), Monterey sand and fine crystal silica (Nakagawa et al. 1997) and Ticino and Kenya sands (Fioravante 2000) as shown in Figure 6.22. The relationship of C_s with S can be approximated using the following equation:

$$\log C_s = -1.6(100 - S)^{0.29} e^{-0.006(100-S)^{1.29}} \quad (6.29)$$

where e is natural number and S is degree of saturation in percent.

The variation of C_s can be divided into three zones:

- i) Zone 1 ($S < 40\%$): Air phase is continuous but water phase is discontinuous (Blight 1997). In this zone, the soil structure is flocculated. The pore air generally has longer pore passages.
- ii) Zone 2 ($40\% < S < 80\%$): Transitional zone in which air phase becomes discontinuous and water phase becomes continuous (i.e. continuous air and water phase). In this zone, the soil structure is also flocculated. There is a wide ranges of pore air size both spherical and needle shaped pore air as the air phase changes from continuous to discontinuous and the water phase changes from discontinuous to continuous.
- iii) Zone 3 ($S > 80\%$): Air phase is discontinuous and water phase is continuous (Fredlund and Rahardjo 1993). In this zone, the soil structure is more oriented or dispersed. The size of the pore air is generally uniform and the shape changes from needle like to spherical.

The proposed model for compression wave velocity is given by:

$$V_p = \sqrt{\frac{K_s + \frac{4}{3}G_{\max} + K_f^{***}}{\rho}} \quad (6.30)$$

Figure 6.21 Variation of measured and calculated compression wave velocities for residual soils.

Figure 6.22 Variation of C_s with degree of saturation.

6.6.3 Comparison of Measured and Calculated Compression Wave Velocity

The variation of compression wave velocity obtained from Equation 6.30 with degree of saturation is plotted in Figure 6.14 for the Bukit Timah Granite and Jurong Formation residual soils. The compression wave velocity obtained from Equation 6.30 increases with degree of saturation and approaches the compression wave velocity of water at full saturation. This agrees with the findings in the literature (Stephenson 1978 and Allen et al. 1980).

The validity of Equation 6.30 is also verified using other experimental data from the literature for various types of soil, rock and concrete (Figure 6.23). Figure 6.23 shows that Equation 6.30 not only provides a good match with measured compression wave velocity for soils such as compacted silty clay (Stephenson 1978), compacted clayey soil S1, S2 and S3 (Inci et al. 2003) and Ottawa sand (Hardin and Richart 1963 and Allen et al. 1980), but also concrete under dry and wet condition (Yaman et al. 2000) and limestone under dry and wet condition (Baechle et al. 2003) with an error of less than $\pm 20\%$. The ability to estimate the compression wave velocity with higher accuracy using a less rigorous equation, provided by Equation 6.30 facilitated the estimation of compression wave velocity for different degree of saturation. The discrepancy of $\pm 20\%$ is due to C_s not accounting for the exact pore air shape and distribution especially with degree of

saturation in the range of 0% to 60%. The model can be further improved if more data on different types of soil at different degree of saturation especially in the range of 0% to 50% is available for analysis. The study in this thesis is limited to degree of saturation in the range of 60% to 100% because it is difficult to dynamically compact soil specimens at degree of saturation less than 50%.

Compacted soils which are unsaturated are commonly used in construction of embankments, backfill and subgrades of pavement. The shear wave and compression wave velocities are needed to evaluate the soil-structure response due to the vibration induced by people, vehicle and construction activity. The ability to estimate compression wave velocity from initial shear modulus has facilitated the use of dynamic analysis to investigate the soil-structure interaction due to vibration. For example, an analytical solution for an oscillator resting on a semi infinite elastic body can be used to determine the dynamic characteristics of foundation subjected to vibration if the shear wave velocity, compression wave velocity and damping ratio are known. Thus, Equation 6.30 is very useful as a single test to obtain initial shear modulus or shear wave velocity can be used to obtain compression wave velocity for dynamic analysis.

Figure 6.23 Comparison of measured and calculated compression wave velocities for various materials.

Chapter 7 Small Strain Shear Modulus and Damping Ratio

In the past, the small strain cyclic characteristics ($0.0005\% < \gamma < 1\%$) of soils were measured in the laboratory mainly by resonant column test (Woods 1991 and 1994). Nowadays, measurement technique for small strain characteristics such as shear modulus, G , and damping ratio has been extended to cyclic simple shear and cyclic torsional shear tests. The cyclic simple shear apparatus described in Chapter 5 was employed to determine the shear modulus and damping ratio at small shear strain ($0.0005\% < \gamma < 1\%$). Hardin and Drnevich (1972) reported that shear strain, effective confining pressure, frequency of loading and number of loading cycles are important parameters for determination of shear modulus and damping ratio for both sand and clay. Thus, the effect of effective confining pressure, frequency of loading and number of loading cycles on shear modulus and damping ratio for shear strain less than 1% ($0.005\% < \gamma < 1\%$) on residual soils from Bukit Timah Granite site I and Jurong Formation site III in Singapore were studied. In order to obtain a complete stiffness-strain curve for residual soils in Singapore, the conventional triaxial compression test was employed to measure the shear modulus at shear strains greater than 1% and the ultrasonic test was employed to measure the initial shear modulus. The major parameters such as shear modulus, initial shear modulus and damping ratio in the shear strain range of 0.0001% to 5% and the non-linearity of the stiffness-strain curve for residual soil for Bukit Timah Granite site I and II and Jurong Formation site III and IV are discussed in this chapter.

7.1 Testing Program

A series of ultrasonic tests, cyclic simple shear tests and triaxial compression tests were conducted on compacted Bukit Timah Granite residual soil from sites I and II and Jurong Formation residual soil from sites III and IV. The Bukit Timah Granite site I residual soil and Jurong Formation site III residual soil were compacted to conditions as shown in Table 6.1. The compaction curves for both Bukit Timah

Granite site I residual soil and Jurong Formation site III residual soil are shown in Figures 6.1 and 6.2, respectively.

The compacted soil specimen of size 100 mm (diameter) x 160 mm (height) was trimmed to the required diameter of 70 mm using a soil lathe. After trimming, the resulting soil specimen was placed into a two part split mould and trimmed to the final height. The surface of soil specimen was trimmed flat using a straight edge. The height to diameter ratio of the soil specimen was about 0.5 for cyclic simple shear test while the height to diameter ratio of the soil specimen for ultrasonic test and triaxial compression test was 2. All the tests were conducted under consolidated undrained condition (CU). The saturation and consolidation process were conducted in accordance with ASTM D4767-95 (1997). The specimens were kept undrained throughout the cyclic loading stages while increasing the shear strain levels as the excess pore water pressure developed during the test is insignificant to affect the effective stress condition. A typical pore-water pressure response curve is shown in Figure 7.1. The shearing rate used for triaxial compression test was 0.04 mm/min based on ASTM D4767-95 (1997).

A total of 24 soil specimens were tested under consolidated undrained condition using ultrasonic test, cyclic simple shear test and triaxial compression test. Twelve out of 24 soil specimens were tested using cyclic simple shear test while another 12 soil specimens were tested using ultrasonic test and triaxial compression test. In addition, undisturbed soil specimens were obtained from both Bukit Timah Granite site II and Jurong Formation site IV to determine their shear modulus and damping ratio. Details of the cyclic simple shear tests and conventional triaxial tests are shown in Appendices A and B, respectively. Comparison between the shear modulus and damping ratio of the compacted and undisturbed soil specimens are made. The properties of the compacted and undisturbed residual soil specimens tested using ultrasonic tests, cyclic simple shear tests and triaxial compression tests are summarized in Table 7.1.

Figure 7.1 Typical pore-water pressure response during cyclic loading stages for specimen T11(M).

Table 7.1 Properties of soil specimens for ultrasonic test, cyclic simple shear test and triaxial compression test

Specimen Number	e	ρ (Mg/m ³)	w^* (%)	B	Effective confining pressure (kPa)	Site	Remarks
T11 (M) [#]	0.54	2.09	20.11	0.98	100	III	Standard 3
T21 (M) [#]	0.54	2.09	20.11	0.98	200	III	Standard 3
T41 (M) [#]	0.54	2.09	20.11	0.98	400	III	Standard 3
T11 (B) [#]	0.52	2.02	20.11	0.96	100	I	Standard 3
T21 (B) [#]	0.52	2.02	20.11	0.96	200	I	Standard 3
T41 (B) [#]	0.52	2.02	20.11	0.96	400	I	Standard 3
T12 (M) [#]	0.46	2.05	12.25	0.99	100	III	Standard 3
T22 (M) [#]	0.46	2.05	12.25	0.99	200	III	Standard 3
T42 (M) [#]	0.46	2.05	12.25	0.99	400	III	Standard 3
T12 (B) [#]	0.44	2.01	12.92	0.98	100	I	Standard 3
T22 (B) [#]	0.44	2.01	12.92	0.98	200	I	Standard 3
T42 (B) [#]	0.44	2.01	12.92	0.98	400	I	Standard 3
D11 (M) [#]	0.21	2.42	7.62	0.99	100	IV	Undisturbed
D12 (M) [#]	0.21	2.40	6.86	0.96	100	IV	Undisturbed
D11 (B) [#]	1.01	1.72	40.90	0.98	100	II	Undisturbed
D12 (B) [#]	0.96	1.75	40.30	0.96	100	II	Undisturbed

[#] B represents Bukit Timah Granite residual soil while M represents Jurong Formation residual soil

* w shown is the initial water content before saturation

7.2 Effective Confining Pressure

According to Macari and Hoyos (1996), Lanzo and Vucetic (1998), D'Elia et al. (2003) and Yashuhara (2003), shear modulus increases as effective confining pressure increases. The shear modulus-shear strain curve shifts upwards as the effective confining pressure increases. This is because the application of effective confining pressure on the saturated residual soil decreases its void ratio as the excess pore-water pressure was allowed to dissipate since drainage was allowed. The soil particles rearrange into a more compact and stable structure and subsequently, shear resistance increases and therefore, shear modulus increases. The variation of shear modulus-shear strain relationship with effective confining pressure for both Bukit Timah Granite site I residual soil and Jurong Formation site III residual soil are shown in Figure 7.2. The shear modulus and damping ratio were determined from the first loading cycle in the cyclic simple shear test. The effect of number of loading cycles on shear modulus and damping ratio discussed in Section 7.4 showed negligible effect of number of loading cycle on shear modulus and damping ratio. Shear modulus increases as the effective confining pressure increases. In addition, the effect of the effective confining pressure on shear modulus diminishes as shear strain increases from 0.005% to more than 1%.

The effect of effective confining pressure on damping ratio is also shown in Figure 7.2. Damping ratio decreases as the effective confining pressure increases for a given shear strain. The damping ratio-shear strain curve shifts downwards as the application of effective confining pressure on the saturated residual soil decreases its void ratio as the excess pore-water pressure was allowed to dissipate since drainage was allowed. Frictional loss due to energy dissipation decreases and therefore, damping ratio decreases. A similar trend was obtained by Tatsuoka et al. (1979), Macari and Hoyos (1996), Vucetic et al. (1998) and Stokoe et al. (1999) and Hsu (2002).

(a) Bukit Timah Granite site I ($e=0.52$)

(b) Bukit Timah Granite site I ($e=0.44$)

(c) Jurong Formation site III ($e=0.54$)

(d) Jurong Formation site III ($e=0.46$)

* shaded symbols are the corresponding damping ratio for the residual soils

Figure 7.2 Variation of shear modulus-strain and damping ratio-strain curves with effective confining pressure for saturated residual soils.

7.3 Strain Rate or Frequency of Loading

It is well established that the stress at a given strain is affected by the strain rate (Matešić and Vucetic 2003). As strain rate increases, the stress at a given strain increases, meaning that the larger the strain rate the higher will be the stress-strain curve. Thus, the shear modulus increases with increasing strain rate or frequency of loading for cyclic test (Whitman 1957, Yong and Japp 1967, Olson and Parola 1967, Akai et al. 1975, Lacasse 1979, Isenhower and Stokoe 1981, Dobry and Vucetic 1987, Lo Presti et al. 1996 and Matešić and Vucetic 2003). The effect of strain rate or frequency of loading on soil is conceptually simple. If the rate of loading is low, more time is allowed for the soil to relax and to creep and a smaller stress is developed at a given strain, thus, a lower stiffness.

In order to investigate the effect of strain rate or frequency of loading on shear modulus of residual soils in Singapore under cyclic strain-controlled loading, the cyclic simple shear tests were conducted on Bukit Timah Granite site I residual soil and Jurong Formation site III residual soil in the frequency range of 0.1 Hz to 1 Hz

at shear strains ranging from 0.14% to 0.17%. The variation of shear modulus with frequency is plotted in Figure 7.3. Based on Figure 7.3, the shear modulus increases linearly with the logarithm of frequency. Some researchers prefer to express the relationship in terms of shear modulus and strain rate instead of frequency (Hsu 2002 and Matešić and Vucetic 2003). The strain rate, $\dot{\gamma}$, can be expressed as an average strain rate for a given cycle:

$$\dot{\gamma} = 4f\gamma \quad (7.1)$$

where f is the frequency in Hz and γ is the applied shear strain in percent.

The variation of shear modulus for residual soils is plotted with strain rate calculated using Equation 7.1 in Figure 7.4. Shear modulus increases linearly with logarithm of strain rate. A similar trend is obtained for the variation of shear modulus with frequency since strain rate is related to frequency (Equation 7.1). The result obtained is consistent with the findings of other researchers (Whitman 1957, Yong and Japp 1967, Olson and Parola 1967, Akai et al. 1975, Lacasse 1979, Isenhower and Stokoe 1981, Dobry and Vucetic 1987, Lo Presti et al. 1996 and Matešić and Vucetic 2003). The slope of shear modulus versus logarithm of strain rate, α_G , measures the influence of strain rate on shear modulus (Hsu 2002). The strain rate shear modulus parameter α_G for Bukit Timah Granite site I residual soil and Jurong Formation site III residual soil are summarized in Table 7.2. The value of α_G ranges from 0.46 MPa to 1.24 MPa for both Bukit Timah Granite and Jurong Formation residual soils. Hsu (2002) found that α_G for sand and clay ranges from 0.8 MPa to 7 MPa for shear strain in the range 0.0005% to 0.02% at strain rates of 0.001%/s to 0.05%/s. Matešić and Vucetic (2003) found that α_G for sand and clay ranges from 0.2 MPa to 6 MPa for shear strain in the range from 0.0005% to 0.015% at strain rates of 0.0005%/s to 0.1%/s. The values of α_G found in this study are within the range of the values for α_G found in the literature.

Damping ratio is also a function of strain rate or frequency of loading. The effects of frequency of loading and strain rate on damping ratio are also shown in Figures 7.3 and 7.4 (shaded symbols), respectively. Based on Figures 7.3 and 7.4, the damping ratio generally decreases linearly with logarithm of strain rate and frequency except for Jurong Formation site III residual soil at void ratio of 0.54. The trend obtained seems to contradict with Hsu (2002) who found that damping ratio to either increase with strain rate or hardly change with strain rate. In fact, the relationship between damping ratio and strain rate is more complicated. The damping ratio first decreases and then increases with strain rate and frequency as shown in Figure 7.5 (D'onofrio 1996, Cavallaro 1997, Shibuya et al. 1997, Lanzo and Vucetic 1998, Lanzo et al. 1999 and Kramer 2000). However, in this study damping ratio was found to decrease with increasing strain rate and frequency in general.

The slope of damping ratio versus logarithm of strain rate, α_D , measures the influence of strain rate on damping ratio. The values of α_D for Bukit Timah Granite site I residual soil and Jurong Formation site III residual soil are summarized in Table 7.2. The magnitude of α_D ranges from -0.034 to -0.012 and -0.028 to 0 for Bukit Timah Granite and Jurong Formation residual soils, respectively.

(a) Bukit Timah Granite site I ($e=0.52$)

(b) Jurong Formation site III ($e=0.54$)

(c) Bukit Timah Granite site I ($e=0.44$)

(d) Jurong Formation site III ($e=0.46$)

* shaded symbols are the corresponding damping ratio for the residual soils

Figure 7.3 Variation of shear modulus and damping ratio with frequency of loading.

(a) Bukit Timah Granite site I ($e=0.52$)

(b) Jurong Formation site III ($e=0.54$)

(c) Bukit Timah Granite site I ($e=0.44$)

(d) Jurong Formation site III ($e=0.46$)

* shaded symbols are the corresponding damping ratio for the residual soils

Figure 7.4 Variation of shear modulus and damping ratio with strain rate.

Table 7.2 Parameter α_G and α_D for Bukit Timah Granite and Jurong Formation residual soils

Specimen Number	α_G (MPa)	α_D *	Specimen Number	α_G (MPa)	α_D *
T11 (B) [#]	0.65	-0.029	T11 (M) [#]	0.51	0
T21 (B) [#]	1.24	-0.034	T21 (M) [#]	1.05	0
T41 (B) [#]	0.46	-0.033	T41 (M) [#]	1.13	0
T12 (B) [#]	0.64	-0.015	T12 (M) [#]	0.88	-0.023
T22 (B) [#]	0.68	-0.012	T22 (M) [#]	0.71	-0.018
T42 (B) [#]	1.00	-0.018	T42 (M) [#]	0.53	-0.028

[#] B represents Bukit Timah Granite residual soil while M represents Jurong Formation residual soil

* negative value denotes a decrement in damping ratio with frequency or strain rate

Figure 7.5 Variation of damping ratio with frequency for clay at $\gamma = \pm 0.005\%$ (modified from Lanzo et al. 1999).

7.4 Number of Loading Cycles

According to Drnevich et al. (1967), shear modulus increases with increasing number of loading cycles. However, the effect of the number of loading cycles on shear modulus depends on the shear strain level (Vucetic and Dobry 1991). According to Vucetic and Dobry (1991), the effect of the number of loading cycles on shear modulus is significant if shear strain is less than a threshold (0.06%). The

shear modulus changes with number of loading cycles if the shear strain is above the cyclic volumetric threshold shear strain, γ_v , as the soil microstructure changes permanently with number of loading cycles. In addition, it is also known that the shear modulus increases with number of loading cycles for soil with low plasticity index while the shear modulus decreases with number of loading cycles for soil with high plasticity index. Thus, the effect of number of loading cycles on shear modulus depends on both the cyclic volumetric threshold shear strain and the plasticity index.

In order to study the affect of number of loading cycles on shear modulus, the cyclic volumetric threshold shear strain has to be determined. Cyclic simple shear test under undrained condition using the procedure described in Section 5.1.1 were conducted on Bukit Timah Granite site I residual soil and Jurong Formation site III residual soil to determine the volumetric threshold shear strain. A specimen was tested under several consecutive cyclic strain-controlled stages with different constant cyclic shear strain in each stage. The cyclic shear strain in the range of 0.003% to 1% was applied to the specimen in each stage. In each stage, a cyclic simple shear strain was applied and the excess pore-water pressure generated was recorded. The excess pore-water pressure change (i.e. difference of excess pore-water pressure for i^{th} cycle and first cycle for corresponding shear strain) was normalized with the effective confining pressure. The variation of normalized excess pore-water pressure change with shear strain for different number of cycles are plotted in Figures 7.6 and 7.7 for Bukit Timah Granite site I and Jurong Formation site III residual soils, respectively. The variation of normalized excess pore-water pressure change with shear strain is similar to the trend obtained by Dyvik et al. (1984), Ohara and Matsuda (1988) and Hsu (2002). An example of the variation of normalized excess pore-water pressure change with shear strain from Hsu (2002) is shown in Figure 7.8. The normalized excess pore-water pressure change increases with shear strain. The range of shear strain at which the normalized excess pore-water pressure change started to increase or a permanent excess pore-water pressure change is present is denoted as γ_v (cyclic volumetric

threshold shear strain). A range of cyclic volumetric threshold shear strain was normally reported (Figure 7.8). The cyclic volumetric threshold shear strain obtained for the residual soils in this study are reported in Table 7.3. The determination of cyclic volumetric threshold shear strain would facilitate the explanation for variation of shear modulus with number of loading cycles. In addition, Figures 7.6 and 7.7 show that the cyclic volumetric threshold shear strain increases as the effective confining pressure increases. A similar finding was obtained by Hsu (2002).

Figure 7.6 Determination of cyclic volumetric threshold shear strain for Bukit Timah Granite site I residual soil.

Figure 7.7 Determination of cyclic volumetric threshold shear strain for Jurong Formation site III residual soil.

Figure 7.8 Determination of cyclic volumetric threshold shear strain for fully saturated kaolin specimen with initial water content of 43.6% (modified from Hsu 2002).

Table 7.3 Cyclic volumetric threshold shear strain for residual soils

Specimen Number	γ_{tv} (%)	Specimen Number	γ_{tv} (%)
T11 (B)	0.02-0.1	T11 (M)	0.02-0.12
T21 (B)	0.02-0.18	T21 (M)	0.02-0.12
T41 (B)	0.02-0.18	T41 (M)	0.02-0.24
T12 (B)	0.20	T12 (M)	0.05
T22 (B)	0.20	T22 (M)	0.09
T42 (B)	0.25	T42 (M)	0.20

Typical shear stress-strain loops for the first, tenth and hundredth cycle of Jurong Formation site III residual soil at an initial effective confining pressure of 200 kPa (i.e. at the beginning of the cyclic undrained loading test) and cyclic shear strain of $\pm 0.17\%$ are shown in Figure 7.9. The variation of shear modulus for residual soils at shear strain of 0.14%-0.17% with number of loading cycles is shown in Figure 7.10. It is found that the shear modulus of both types of residual soils either increases linearly with logarithm of the number of loading cycles or remains constant with number of loading cycles. For the soil specimen with shear strain greater than the cyclic volumetric shear strain ($\gamma > \gamma_{tv}$), T11 (B), T21 (B), T41 (B), T11 (M), T21

(M), T41 (M) and T12 (M), the shear modulus increases linearly with logarithm of the number of loading cycles. This finding agrees with that of Drnevich et al. (1967), Silver and Seed (1971) and Hardin and Drnevich (1972). For the soil with shear strain lower than the cyclic volumetric shear strain ($\gamma \leq \gamma_{lv}$), the shear modulus remains almost constant with number of loading cycles because the soil does not experience significant changes in pore-water pressure, thus, no permanent soil microstructure changes occurred.

In addition, the variation of damping ratio with number of loading cycles is also shown in Figure 7.10 (shaded symbols). It is found that the damping ratio is almost unaffected by the number of loading cycles. The damping ratio remains almost constant and independent of the number of loading cycles. This result is similar to the findings obtained of Dobry and Vucetic (1987) and Borden et al. (1996) who found no significant effect of number of loading cycles on damping ratio.

Figure 7.9 Shear stress-strain loops for first, tenth and hundredth cycle for Jurong Formation site III residual soil.

Figure 7.10 Variation of shear modulus and damping ratio with number of loading cycles.

7.5 Variation of Shear Modulus and Damping Ratio with Shear Strain

The analysis of the data over a range of cyclic shear strain from 0.00005% to 5% is performed to characterize the non-linearity of Singapore residual soils by considering the relationship between shear modulus, G , and damping ratio with shear strain, γ . In this section, all the data from the tests are collated and the general trends are discussed. Empirical equations are proposed to estimate the shear modulus and the damping ratio at various strain levels.

7.5.1 Stiffness-Strain Relationships

The shear moduli of Bukit Timah Granite site I residual soil and Jurong Formation site III residual soil obtained from ultrasonic test, cyclic simple shear test and triaxial compression test plotted against shear strain are shown in Figure 7.2. The shear modulus remains constant at very small shear strain and decreases as the shear strain increases. The shear modulus becomes very small compared to G_{max} when the shear strain exceeds 1%. This finding is consistent with the findings of Thiers and Seed (1968), Park and Silver (1975), Anderson et al. (1983), Seed et al. (1986), Kagawa (1992), Viggiani and Atkinson (1995), Macari and Hoyos (1996), Guha et al. (1997), Rollins et al. (1998), Ng and Wang (2001), Wehling et al. (2003) and Yasuhara et al. (2003). As discussed earlier (Section 7.2), shear modulus is affected by effective confining pressure. Normally, the variation of the shear modulus with cyclic shear strain is represented by normalizing shear modulus, G , at a given shear strain with the initial shear modulus, G_{max} . This normalization process makes it possible to compare stiffness-strain relationships under different effective confining pressure and those obtained by other researchers. The G/G_{max} versus γ relationship for both Bukit Timah Granite site I residual soil and Jurong Formation site III residual soil is shown in Figure 7.11. Figure 7.11 shows that the G/G_{max} versus γ curves fall in a band. The shapes of all the G/G_{max} versus γ curves are very similar. It would be convenient if all the G/G_{max} versus γ curves can be collapsed into a single curve. Such attempts have been made by Santos and Gomes (2000) and Hsu (2002). Santos and Gomes (2000) and Hsu (2002) normalized the shear strain, γ , by a reference cyclic shear strain, γ_{ref} . They suggested that γ_{ref} is γ corresponding to G/G_{max} of 0.7. By plotting the G/G_{max} versus γ/γ_{ref} , they showed that the curves obtained fall within a very narrow band (Figure 7.12).

Figure 7.11 Variation of G/G_{max} with shear strain for compacted residual soils.

Figure 7.12 Variation of G/G_{max} with γ/γ_{ref} for various soil types (from Hsu 2002).

The same approach is applied in this study in which γ_{ref} corresponding to γ at G/G_{max} of 0.7 is used to normalize the shear strain. The method to determine γ_{ref} is

shown in Appendix C. The variation of G/G_{max} with γ/γ_{ref} for all the data points shown in Figure 7.11 are plotted in Figure 7.13. Figure 7.13 shows that all the G/G_{max} versus γ/γ_{ref} curves collapsed into a single curve. It shows a unique G/G_{max} versus γ/γ_{ref} curve for the Singapore residual soils. This facilitates the development of an empirical equation to estimate the shear modulus of the residual soils at a given shear strain. The γ_{ref} for the Bukit Timah Granite site I and Jurong Formation site II residual soils are shown in Table 7.4.

Table 7.4 Reference cyclic shear strain for shear modulus, γ_{ref} , for Bukit Timah Granite site I and Jurong Formation site II residual soils

Specimen Number	G_{max} (MPa)	γ_{ref} (%)	Specimen Number	G_{max} (MPa)	γ_{ref} (%)
T11 (B)	82	0.009	T11 (M)	69	0.010
T21 (B)	108	0.007	T21 (M)	98	0.009
T41 (B)	165	0.005	T41 (M)	157	0.005
T12 (B)	127	0.005	T12 (M)	149	0.005
T22 (B)	159	0.005	T22 (M)	172	0.004
T42 (B)	220	0.003	T42 (M)	263	0.003

Hardin and Drnevich (1972) suggested that the variation of shear modulus with shear strain can be modelled by the modified hyperbolic stress-strain relationship:

$$\frac{G}{G_{max}} = \frac{1}{1 + \frac{\gamma}{\gamma_{ref}} \left[1 + J e^{-q \left(\frac{\gamma}{\gamma_{ref}} \right)} \right]} \tag{7.2}$$

where e is the natural number, J and q are constants.

The best-fit curve for the data in Figure 7.13 gives $J = 0.65$ and $q = -1.05$ with a root mean square error of ± 0.02 :

$$\frac{G}{G_{\max}} = \frac{1}{1 + \frac{\gamma}{\gamma_{\text{ref}}} \left[1 + 0.65e^{1.05\left(\frac{\gamma}{\gamma_{\text{ref}}}\right)} \right]} \quad (7.3)$$

Another equation was proposed by Borden et al. (1996) for Piedmont residual soil (Equation 7.4) as residual soil is not a sedimentary soil such as sand and clay and Equation 7.2 may not be valid. The equation proposed by Borden et al. (1996) is

$$\frac{G}{G_{\max}} = \frac{1}{\left[1 + M \left(\frac{\gamma}{\gamma_{\text{ref}}} \right)^A \right]^x} \quad (7.5)$$

where A , M and x are constants.

The best-fit curve for the data in Figure 7.13 using Equation 7.5 gives $A = 5.28$, $M = 3.42$ and $x = 0.22$ with a root mean square error of ± 0.009 :

$$\frac{G}{G_{\max}} = \frac{1}{\left[1 + 5.28 \left(\frac{\gamma}{\gamma_{\text{ref}}} \right)^{3.42} \right]^{0.22}} \quad (7.6)$$

Equation 7.6 gives a better fit to the Singapore residual soil data than Equation 7.3. The value of x obtained in this study lies within the range obtained by Borden et al. (1996). For Piedmont residual soil, x varies from 0.15 to 0.55 (Borden et al. 1996).

Equation 7.6 is useful to estimate the shear modulus of compacted Singapore residual soils at any shear strain levels, if both the $G_{\max}$ and γ_{ref} are known. $G_{\max}$ can

be obtained directly from ultrasonic test. However, the determination of γ_{ref} is not as straight forward as G_{max} as it involves the determination of γ_{ref} . The variation of G_{max} with γ_{ref} is plotted in Figure 7.14. It is found that γ_{ref} varies linearly with G_{max} on a logarithmic scale for Singapore residual soils. The relationship between γ_{ref} and G_{max} is given by

$$\gamma_{ref} = -0.0058 \ln \left(\frac{G_{max}}{P_{atm}} \right) + 0.047 \quad (7.7)$$

where γ_{ref} is in percent, G_{max} is in MPa and P_{atm} is atmospheric pressure (= 0.1 MPa).

Thus, shear modulus for compacted Singapore residual soils at any shear strain levels can be estimated using Equations 7.6 and 7.7, if G_{max} is known.

Figure 7.13 Variation of G/G_{max} with γ/γ_{ref} for compacted residual soils.

Figure 7.14 Variation of G_{max} with γ_{ref} for Singapore residual soils.

7.5.2 Damping Ratio-Strain Relationships

The variation of the damping ratio of Singapore residual soil with shear strain in Figure 7.2 shows that the damping ratio increases as the shear strain increases. A similar trend is obtained by Thiers and Seed (1968), Park and Silver (1975), Anderson et al. (1983), Seed et al. (1986), Kagawa (1992), Macari and Hoyos (1996), Guha et al. (1997), Rollins et al. (1998), Wehling et al. (2003) and Yasuhara et al. (2003). The damping ratio of Singapore residual soils ranges from 7% to 9% at very small shear strain. In general, the very small strain damping ratio obtained by other researchers for sand and clay at frequencies less than 1 Hz is in the range of 0% to 10% (Vucetic 1994 and Hsu 2002). Kim et al. (1991), Shibuya et al. (1995) and Tatsuoka et al. (1995) stated that the damping ratio at strain levels less than 0.001% cannot be neglected, though it is small. The very small strain damping ratio obtained in this study lies within the typical range reported. However, it is known that damping ratio is a function of applied frequency or strain rate in which damping ratio first decreases with frequency of loading and then increases with frequency of loading (D’onofrio 1996, Cavallaro 1997, Shibuya et al. 1997, Lanzo

and Vucetic 1998, Lanzo et al. 1999, Santucci De Magistris et al. 1999 and Kramer 2000). Therefore, the very small strain damping ratio obtained in this study should be corrected as the damping ratio at very small shear strain obtained in this study is at a frequency of 76 kHz which is much higher than the loading frequency of the cyclic simple shear test (i.e. $f=0.1$ Hz).

The relationship between damping ratio and frequency for soil is complicated. There are some works done on the frequency effect on damping ratio for soil in the literature and this study, however, it is not comprehensive as the frequency range is limited. The frequency range used is generally less than 20 Hz or greater than 0.5 MHz. There is not much work done for frequency ranges from 0.1 Hz to 0.5 MHz. In addition, the theoretical models proposed in the literature were not substantiated with sufficient experimental data. Hence, the correction for the very small damping ratio for its frequency effect is not simple. However, Nakagawa et al. (1997) and Miura et al. (2001) showed that the very small strain damping ratio for shear wave for various saturated soils (sand and clay) is generally less than 10% for frequencies less than 100 kHz. Therefore, the correction for very small strain damping ratio for saturated soils can be disregarded as the very small strain damping ratio does not vary more than 10%. Thus, no correction is done for the very small strain damping ratio obtained for saturated residual soils in this section as the very small strain damping ratios obtained were less than 10%. The very small strain damping ratios obtained are summarized in Table 7.5.

Table 7.5 Very small strain damping ratio for saturated residual soils

Specimen Number	D_{amp}	Specimen Number	D_{amp}
T11 (B)	0.081	T11 (M)	0.076
T21 (B)	0.080	T21 (M)	0.073
T41 (B)	0.078	T41 (M)	0.070
T12 (B)	0.093	T12 (M)	0.081
T22 (B)	0.093	T22 (M)	0.070
T42 (B)	0.091	T42 (M)	0.070

Similar to shear modulus, damping ratio is also affected by the effective confining pressure and soil type. To compare the damping ratios for different effective confining pressure and soil type, a maximum damping ratio, D_{max} , is needed. Hardin and Drnevich (1972) suggested that D_{max} be the maximum damping ratio is when shear modulus is equal to zero (i.e. $G=0$) for saturated cohesive soil. However, in the present study, shear modulus reduces to a small value but not zero. And, the cyclic simple shear apparatus described in Chapter 5 is capable of measuring damping ratio until shear strain of 1%. Therefore, it is proposed that D_{max} be taken as D_{amp} at 1% shear strain i.e. $D_{amp@1\%}$.

The variation of the corrected damping ratio together with the damping ratios obtained from cyclic simple shear test normalized by the damping ratio at shear strain of 1% ($D_{amp}/D_{amp@1\%}$) with shear strain is plotted in Figure 7.15. Figure 7.15 shows that the $D_{amp}/D_{amp@1\%}$ versus shear strain curves have a similar trend and fall in a band. It is also found that if the shear strain is normalized by reference cyclic shear strain for damping ratio, γ_{ref}' , which is γ at $D_{amp}/D_{amp@1\%}$ of 0.7, all the $D_{amp}/D_{amp@1\%}$ versus γ/γ_{ref}' curves fall into a narrow band as shown in Figure 7.16. Table 7.6 shows the γ_{ref}' for the Bukit Timah Granite site I and Jurong Formation site III residual soils.

Table 7.6 Reference cyclic shear strain for damping ratio, γ_{ref}' , for Bukit Timah Granite site I and Jurong Formation site III residual soils

Specimen Number	$D_{amp@1\%}$	$\gamma_{ref}'(\%)$	Specimen Number	$D_{amp@1\%}$	$\gamma_{ref}'(\%)$
T11 (B)	0.27	0.0055	T11 (M)	0.34	0.0070
T21 (B)	0.28	0.0060	T21 (M)	0.33	0.0070
T41 (B)	0.25	0.0050	T41 (M)	0.25	0.0060
T12 (B)	0.34	0.0025	T12 (M)	0.33	0.0024
T22 (B)	0.33	0.0030	T22 (M)	0.32	0.0025
T42 (B)	0.32	0.0030	T42 (M)	0.31	0.0030

According to Hardin and Drnevich (1972), the relationship between the shear modulus and damping ratio for sand and clay is given by

$$\frac{D_{amp}}{D_{max}} = 1 - \frac{G}{G_{max}} \quad (7.9)$$

where D_{max} is the maximum damping ratio at $G = 0$. If D_{max} is taken as $D_{amp@1\%}$, and substituting G/G_{max} with Equation 7.6, Equation 7.9 becomes

$$\frac{D_{amp}}{D_{amp@1\%}} = 1 - \frac{1}{\left[1 + 5.28 \left(\frac{\gamma}{\gamma_{ref}} \right)^{3.42} \right]^{0.22}} \quad (7.10)$$

Equation 7.10 is plotted in Figure 7.16. Figure 7.16 shows that Equation 7.10 does not match the test results for the Singapore residual soils. However, Equation 7.10 can be retained in form but with parameters M , A and x found by curve fitting, i.e.

$$\frac{D_{amp}}{D_{amp@1\%}} = 1 - \frac{1}{\left[1 + M \left(\frac{\gamma}{\gamma_{ref}} \right)^A \right]^x} \quad (7.11)$$

The best-fit curve to represent the $D_{amp}/D_{amp@1\%}$ for Singapore residual soil in Figure 7.16 using Equation 7.12 gives $A = 0.40$, $M = 0.10$ and $x = 17$ with a root mean square error of ± 0.03 :

$$\frac{D_{amp}}{D_{amp@1\%}} = 1 - \frac{1}{\left[1 + 0.1 \left(\frac{\gamma}{\gamma_{ref}'}\right)^{0.4}\right]^{17}} \quad (7.12)$$

Figure 7.15 Variation of $D_{amp}/D_{amp@1\%}$ with shear strain for compacted residual soils.

Figure 7.16 Variation of $D_{amp}/D_{amp@1\%}$ with γ/γ_{ref}' for compacted residual soils.

Equation 7.12 is useful to estimate the damping ratio of compacted Singapore residual soils at any shear strain levels, if both the $D_{amp@1\%}$ and γ_{ref}' are known. $D_{amp@1\%}$ can be obtained directly from cyclic simple shear test. However, the determination of γ_{ref}' is not as straight forward as $D_{amp@1\%}$ as it involves the determination of shear strain corresponding to $D_{amp}/D_{amp@1\%}$ of 0.7. The variation of $D_{amp@1\%}$ with γ_{ref}' is plotted in Figure 7.17. It is found that $D_{amp@1\%}$ is not a unique function of γ_{ref}' for Singapore residual soils. The γ_{ref}' ranged from 0.0025% to 0.007% with an average value of 0.005%.

An upper bound, lower bound and average damping ratio for compacted Singapore residual soils at any shear strain levels can be estimated using Equations 7.12 with γ_{ref}' of 0.007%, 0.0025% and 0.005%, respectively.

Figure 7.17 Variation of $D_{amp@1\%}$ with γ_{ref}' for Singapore residual soils.

7.5.3 Stiffness-Strain and Damping Ratio Relationships for Undisturbed Soil Specimens

In this study, undisturbed residual soil specimens were obtained from both Bukit Timah Granite site II and Jurong Formation site IV. The basic index properties of the undisturbed residual soils are shown in Table 2.6 and Figure 2.22. The undisturbed soil specimens were obtained at a depth of about 10 m. An effective confining pressure of 100 kPa was applied to the soil specimens during the ultrasonic test, cyclic simple shear test and triaxial compression test to simulate the in-situ stress condition, assuming $K_o = 0.5$.

The measured shear modulus and damping ratio for undisturbed Bukit Timah Granite site II and Jurong Formation site IV residual soil specimens at various shear strains are plotted in Figure 7.18 and the values of G_{max} , $D_{amp@1\%}$, γ_{ref} and γ_{ref}' are shown in Table 7.7. The relationships of G_{max} with γ_{ref} and $D_{amp@1\%}$ with γ_{ref}' are also shown in Figures 7.14 and 7.17, respectively.

* shaded symbols are the corresponding damping ratio

Figure 7.18 Variation of shear modulus and damping ratio for undisturbed residual soils.

Table 7.7 Values of G_{max} , $D_{amp@1\%}$, γ_{ref} and γ_{ref}' for undisturbed residual soil specimens.

Specimen Number	G_{max} (MPa)	γ_{ref} (%)	$D_{amp@1\%}$	γ_{ref}' (%)
D11 (M)	270	0.002	0.34	0.003
D12 (M)	258	0.002	0.35	0.003
D11 (B)	166	0.004	0.39	0.003
D12 (B)	187	0.003	0.37	0.003

The variation of G/G_{max} with γ/γ_{ref} for the data points shown in Figure 7.18 is plotted in Figure 7.19. Figure 7.19 shows that all the G/G_{max} versus γ/γ_{ref} curves collapsed into a single curve which can be described using Equation 7.6 (solid line). Similar to G/G_{max} , the variation of $D_{amp}/D_{amp@1\%}$ with γ/γ_{ref}' for data points shown in Figure 7.18 is plotted in Figure 7.20. Figure 7.20 shows that all the $D_{amp}/D_{amp@1\%}$ versus γ/γ_{ref}' curves collapsed into a single curve which can be described by Equation 7.12 (solid line).

In Figures 7.19 and 7.20, the γ_{ref} and γ_{ref}' used were determined from experimental data (Table 7.7). To evaluate the quality of estimates using Equations 7.6 and 7.12, γ_{ref} should be determined from Equation 7.7 and γ_{ref}' should take on values of 0.007%, 0.0025% and 0.005%. The experimental data in Figure 7.19 are replotted using γ_{ref} obtained from Equation 7.7 and shown in Figure 7.19. The difference between the replotted data points and the original data points is minor illustrating that G/G_{max} can be reliably obtained using Equations 7.6 and 7.7 provided G_{max} is known. The experimental data in Figure 7.20 are replotted using γ_{ref}' of 0.007%, 0.0025% and 0.005% and shown in Figure 7.20 as upper bound, lower bound and average curves. The original data shown by the solid lines lies between the average and lower bound curves indicating that D_{amp}/D_{max} can be estimated using Equation 7.12 with an average value of $\gamma_{ref}' = 0.005\%$ provided $D_{amp@1\%}$ is known. However, the estimate of D_{amp}/D_{max} is not as reliable as the estimate of G/G_{max} . Upper and lower bound values of D_{amp}/D_{max} can be obtained using γ_{ref}' values of 0.007% and 0.0025%, respectively.

Figure 7.19 Variation of G/G_{max} with γ/γ_{ref} for undisturbed residual soils.

Figure 7.20 Variation of $D_{amp}/D_{amp@1\%}$ with γ/γ_{ref}' for undisturbed residual soils.

Chapter 8 Conclusion and Recommendations

8.1 Conclusion

In Singapore, residual soils cover two thirds of the land area. Characterizing the engineering properties of the local residual soil is essential for geotechnical design. However, there is very little research done on the characterization of the local residual soil with respect to its soil stiffness non-linearity and damping ratio. To date, there are only a few in-situ geophysical tests done on the local residual soils at limited locations. The determination of soil stiffness and damping ratio by laboratory tests in Singapore is even more scarce. Most of the laboratory tests concentrated on the soil stiffness at large strain. There is little or no work done to characterize damping ratio of local residual soil. Therefore, there is a need to investigate the soil stiffness non-linearity and damping ratio of the Singapore residual soils.

In summary, this study characterized the shear modulus and damping ratio at various strain for the residual soils in Singapore using a combination of pulse transmission tests, cyclic simple shear and triaxial compression tests. The effects of soil parameters such as confining pressure, void ratio and degree of saturation on soil stiffness and damping ratio were investigated. In addition, the effects of frequency and number of cycles of load application on soil stiffness and damping ratio were also investigated. This study also contributed to a better understanding of the bender element test and ultrasonic test. An equation to estimate compression wave velocity from G_{max} and degree of saturation was developed using ultrasonic test results.

Based on the results and findings in the study, the following key conclusions were made:

i) Bender element test

The travel time for bender element should be based on the first arrival of the shear wave. Sinusoidal pulse is a better choice of input signal. A clear receiver signal will be obtained if the signal to noise ratio (SNR) of the receiver signal is at least 6 dB and the wave path length to wavelength ratio (L_{tr}/λ_w) is at least 3.33. The SNR can be increased by increasing the applied voltage. Signal processing is essential to remove transients and noise in the receiver signal. The strain level at which G_{max} was measured was estimated to be in the order of 0.00001%.

ii) Ultrasonic test

Acoustic couplant is essential to obtain good signal in the ultrasonic test. Compression wave velocity measured by ultrasonic system can range from one dimensional compression wave velocity to the wave velocity in an infinite medium depending on the D/λ_p ratio. The effects of L/D and L/λ_w ratios were found to be negligible if $L/D \geq 2$ and $L/\lambda_p \geq 2$ for compression wave and $L/D \geq 1$ and $L/\lambda_s \geq 2$ for shear wave. Attenuation coefficient can be determined using the maximum amplitude of the frequency spectrum. Signal processing is essential for ultrasonic test in order to obtain a clear and clean receiver signal. The strain levels at which G_{max} and $E_{bulk,max}$ were measured were of the order of 0.00001%.

iii) Very small strain soil stiffness and damping ratio

Both G_{max} and $E_{bulk,max}$ increase with confining pressure for compacted residual soils. However, the damping ratio for both compression and shear waves decrease with confining pressure. The compression and shear wave velocities increase with decreasing void ratio. Both compression and shear wave velocities vary linearly with void ratio. The damping ratio for both compression and shear waves decrease linearly with void ratio. The shear wave velocity is independent of the degree of saturation. However, the compression wave velocity increases with degree of

saturation and it approaches compression wave velocity of water when it is fully saturated.

iv) Equation for compression wave velocity

Biot's theory to describe the propagation of compression wave in a porous solid is well accepted, however, the practical application is not easy. Various models have been put forwarded to simplify Biot's equation by different researchers to relate compression wave velocity to G_{max} . These models proposed by researchers were examined. However, the equations were found to be inadequate in estimating the compression wave velocity of the soil at various degree of saturation. The main deficiency is that the micro structural aspects such as solubility of pore air, shape of pores and size of pores is not taken into account. One of the models is modified to address the micro structural aspect. A new equation (i.e. Equation 6.30) was proposed to estimate the compression wave velocity. The validity of Equation 6.30 was verified using various experimental data in the literature for various types of soil, rock and concrete. Comparison of the compression wave velocity estimated using the new equation and the measured compression wave velocity showed good agreement for concrete, rock and soil with an error of $\pm 20\%$.

v) Small strain shear modulus and damping ratio

Shear modulus increases as effective confining pressure increases. However, damping ratio decreases with effective confining pressure. Shear modulus increases linearly with logarithm of frequency and strain rate. The effect of number of loading cycles on shear modulus depends on the cyclic volumetric threshold shear strain. Shear modulus increases linearly with logarithm of number of loading cycles if the applied shear strain is greater than the cyclic volumetric shear strain and remains almost constant with number of loading cycles if the applied shear strain is less than the cyclic volumetric threshold shear strain. Damping ratio is independent of number of loading cycles.

vi) Equations for shear modulus and damping ratio

The variation of G/G_{max} with γ for compacted and undisturbed Singapore residual soils collapsed into a single curve when γ was normalized with a reference cyclic shear strain, γ_{ref} , which is γ corresponding to G/G_{max} of 0.7. The best-fit curve for the relationship of G/G_{max} with γ/γ_{ref} for Singapore residual soils is given by Equation 7.6. A similar normalization was applied to D_{amp} using $D_{amp@1\%}$ which is the damping ratio at 1% shear strain. The $D_{amp}/D_{amp@1\%}$ versus γ curves for compacted and undisturbed Singapore residual soils fell into a very narrow band when γ was normalized with a reference cyclic shear strain, γ_{ref}' , which is γ corresponding to $D_{amp}/D_{amp@1\%}$ of 0.7. The best-fit curve for the relationship of $D_{amp}/D_{amp@1\%}$ with γ/γ_{ref}' for Singapore residual soils is given by Equation 7.12. The γ_{ref} is a unique function of G_{max} , however, γ_{ref}' is not a unique function of $D_{amp@1\%}$. The value of γ_{ref} can be estimated using Equation 7.7 if G_{max} is known. The value of γ_{ref}' ranges from 0.0025% to 0.007%, with an average value of 0.005% for Singapore residual soils. The shear modulus for Singapore residual soils at any shear strain level can be estimated using Equations 7.6 and 7.7 if G_{max} is measured. The damping ratio for Singapore residual soils at any shear strain level can be estimated using Equation 7.12 and $\gamma_{ref}' = 0.005\%$ if $D_{amp@1\%}$ is measured. The estimation for shear modulus is more reliable than the estimation for damping ratio.

8.2 Recommendations

Due to time constraint, it was not possible to investigate all aspect of the factors affecting the stiffness and damping ratio of Singapore residual soils. The following recommendations are suggested for future research:

- i) Tests were only performed on a limited number of residual soil samples. It is widely recognized that residual soils exhibit large variations from locality to locality. It is recommended that more tests be performed to obtain the relationships of stiffness and damping ratio with strain for residual soil samples from different localities to examine the variation, if any.

Furthermore, more tests should be performed on undisturbed soil specimens to determine the difference between compacted and undisturbed soil specimens and, subsequently the effect of compaction process on the soil structure.

- ii) Tests were only performed for residual soil specimens under unconsolidated undrained condition for pulse transmission test. Thus, it is recommended to extend this study to include tests under consolidated drained and undrained condition.
- iii) The effect of plasticity index on shear modulus and damping ratio should be investigated in order to determine its effect on the shear modulus-strain and damping ratio-strain curves. It is believed that the shear modulus-strain curve shifts upwards as the plasticity index increases and the damping ratio-strain curve shifts downwards as the plasticity index increases.
- iv) An exact conclusion on the effect of degree of saturation on very small strain damping ratio cannot be drawn as only limited data are available in the study. It is recommended that a more comprehensive study should be performed to investigate the effect of degree of saturation on very small strain damping ratio. The study should also focus on the air content and the composition of the soil such as size, shape, mineralogy of the soil particles and bonding of the soil particles.
- v) The effect of degree of saturation on the stiffness-strain relationship of the residual soils was not fully investigated in this study as all the cyclic simple shear test and triaxial compression tests were performed on saturated soil specimens. As most residual soils exist in the unsaturated state, the effect of

degree of saturation on the stiffness-strain relationship should be investigated.

- vi) The relationship between damping ratio and strain rate is very complicated as damping ratio first decreases and then increases with strain rate or frequency. Thus, a more comprehensive investigation should be done in order to address the effect of strain rate and frequency on damping ratio.
- vii) The model for relating compression wave velocity to G_{max} (i.e. Equation 6.29) can be further improved if more data on different soil types at different degree of saturation especially in the range of 0% to 50% are available for analysis.
- viii) The effect of stress ratio (shear stress to effective confining pressure) on soil stiffness and damping ratio should be investigated in future.
- ix) The cyclic simple shear apparatus can be improved to increase its strain measurement sensitivity so that shear modulus and damping ratio at strain level ranging from 0.0001% to 0.001% can be determined. This will help to reduce the strain interval between the pulse transmission tests and cyclic simple shear tests.
- x) Local strain measurements were not performed in the triaxial tests. Local strain measurements may give more accurate assessment of the small strain stiffness in the axial strain range of 0.25% to 3%.

- xi) In-situ measurement of soil stiffness and damping ratio of Singapore residual soils should be considered so that the comparison between the in-situ and laboratory measurements can be made.
- xii) Anisotropy in soil stiffness was not considered in this study and may be important for residual soil. Measurement of stiffness of residual soils in orthogonal planes can be conducted in future.

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Appendix A –Cyclic Simple Shear Tests

Figure A1 Stress-strain curve for specimen T11 (M) at various applied shear strain for first loading cycle only.

Figure A2 Stress-strain curve for specimen T21 (M) at various applied shear strain for first loading cycle only.

Figure A3 Stress-strain curve for specimen T41 (M) at various applied shear strain for first loading cycle only.

Figure A4 Stress-strain curve for specimen T11 (B) at various applied shear strain for first loading cycle only.

Figure A5 Stress-strain curve for specimen T21 (B) at various applied shear strain for first loading cycle only.

Figure A6 Stress-strain curve for specimen T41 (B) at various applied shear strain for first loading cycle only.

Figure A7 Stress-strain curve for specimen T12 (M) at various applied shear strain for first loading cycle only.

Figure A8 Stress-strain curve for specimen T22 (M) at various applied shear strain for first loading cycle only.

Figure A9 Stress-strain curve for specimen T42 (M) at various applied shear strain for first loading cycle only.

Figure A10 Stress-strain curve for specimen T12 (B) at various applied shear strain for first loading cycle only.

Figure A11 Stress-strain curve for specimen T22 (B) at various applied shear strain for first loading cycle only.

Figure A12 Stress-strain curve for specimen T42 (B) at various applied shear strain for first loading cycle only.

Figure A13 Stress-strain curve for specimen D11 (M) at various applied shear strain for first loading cycle only.

Figure A14 Stress-strain curve for specimen D12 (M) at various applied shear strain for first loading cycle only.

Figure A15 Stress-strain curve for specimen D11 (B) at various applied shear strain for first loading cycle only.

Figure A16 Stress-strain curve for specimen D12 (B) at various applied shear strain for first loading cycle only.

Figure A17 Stress-strain curve for specimen T11(M) at various applied frequency for first loading cycle only.

Figure A18 Stress-strain curve for specimen T21(M) at various applied frequency for first loading cycle only.

Figure A19 Stress-strain curve for specimen T41(M) at various applied frequency for first loading cycle only.

Figure A20 Stress-strain curve for specimen T11(B) at various applied frequency for first loading cycle only.

Figure A21 Stress-strain curve for specimen T21(B) at various applied frequency for first loading cycle only.

Figure A22 Stress-strain curve for specimen T41(B) at various applied frequency f first loading cycle only.

Figure A23 Stress-strain curve for specimen T12(M) at various applied frequency for first loading cycle only.

Figure A24 Stress-strain curve for specimen T22(M) at various applied frequency for first loading cycle only.

Figure A25 Stress-strain curve for specimen T42(M) at various applied frequency for first loading cycle only.

Figure A26 Stress-strain curve for specimen T12(B) at various applied frequency for first loading cycle only.

Figure A27 Stress-strain curve for specimen T22(B) at various applied frequency for first loading cycle only.

(a) $f=0.2$ Hz

(c) $f=0.8$ Hz

(b) $f=0.4$ Hz

(d) $f=1$ Hz

Figure A28 Stress-strain curve for specimen T42(B) at various applied frequency for first loading cycle only.

Figure A29 Stress-strain curves for first, tenth and hundredth cycle for specimen T11(M).

Figure A30 Stress-strain curves for first, tenth and hundredth cycle for specimen T21(M).

Figure A31 Stress-strain curves for first, tenth and hundredth cycle for specimen T41(M).

Figure A32 Stress-strain curves for first, tenth and hundredth cycle for specimen T11(B).

Figure A33 Stress-strain curves for first, tenth and hundredth cycle for specimen T21(B).

Figure A34 Stress-strain curves for first, tenth and hundredth cycle for specimen T41(B).

Figure A35 Stress-strain curves for first, tenth and hundredth cycle for specimen T12(M).

Figure A36 Stress-strain curves for first, tenth and hundredth cycle for specimen T22(M).

Figure A37 Stress-strain curves for first, tenth and hundredth cycle for specimen T42(M).

Figure A38 Stress-strain curves for first, tenth and hundredth cycle for specimen T12(B).

Figure A39 Stress-strain curves for first, tenth and hundredth cycle for specimen T22(B).

Figure A40 Stress-strain curves for first, tenth and hundredth cycle for specimen T42(B).

Appendix B – Triaxial Compression Tests

Figure B1 Stress-strain curve for specimen T11(M).

Figure B2 Stress-strain curve for specimen T21(M)

Figure B3 Stress-strain curve for specimen T41(M)

Figure B4 Stress-strain curve for specimen T11(B)

Figure B5 Stress-strain curve for specimen T21(B)

Figure B6 Stress-strain curve for specimen T41(B)

Figure B7 Stress-strain curve for specimen T12(M)

Figure B8 Stress-strain curve for specimen T22(M)

Figure B9 Stress-strain curve for specimen T42(M)

Figure B10 Stress-strain curve for specimen T12(B)

Figure B11 Stress-strain curve for specimen T22(B)

Figure B12 Stress-strain curve for specimen T42(B)

Figure B13 Stress-strain curve for specimen D11(M)

Figure B14 Stress-strain curve for specimen D12(M)

Figure B15 Stress-strain curve for specimen D11(B)

Figure B16 Stress-strain curve for specimen D12(B)

Appendix C – Determination of γ_{ref} , γ at G/G_{max} of 0.7

Determination of γ_{ref} , γ at G/G_{max} of 0.7

Figure C1 Variation of G/G_{max} with shear strain.

Figure C1 shows the variation of G/G_{max} with shear strain. A horizontal line which corresponds to G/G_{max} of 0.7 is drawn on Figure C1. The intersection of the horizontal line with the G/G_{max} versus shear strain curve is identified as shown in Figure C1. A vertical line is projected from the intersection point onto the horizontal x-axis (i.e. shear strain axis) to determine the γ_{ref} .

The same method is applicable to the determination of γ_{ref}' , γ at $D_{amp}/D_{amp@1\%}$ for the corresponding $D_{amp}/D_{amp@1\%}$ with shear strain plot.