Use of seismic waves for the characterization of soil properties

Mohammad Harunur Rashid Meer

2008

Mohammad Harunur Rashid Meer. (2008). Use of seismic waves for the characterization of soil properties. Doctoral thesis, Nanyang Technological University, Singapore.

<https://hdl.handle.net/10356/41753>

<https://doi.org/10.32657/10356/41753>

USE OF SEISMIC WAVES FOR THE CHARACTERIZATION OF SOIL PROPERTIES

Mohammad Harunur Rashid Meer

SCHOOL OF CIVIL & ENVIRONMENTAL ENGINEERING

A thesis submitted to the Nanyang Technological University in
fulfillment of the requirements for the Degree of
Doctor of Philosophy
July 2008

ACKNOWLEDGEMENTS

First and foremost, I would gratefully acknowledge the kind and wise supervision provided by Assoc. Prof. Leong Eng Choon, who taught me the ways to do research in a comprehensive way by his valuable guidance, moral support and ever present enthusiasm during the entire period of this research.

I wish to extend my deep appreciation to NTU for providing the scholarship for my study. I am also grateful to all the staff of CEE Geotechnics Laboratory, especially to Mr. Heng Hiang Kim Vincent, Mr. Phua Kok Soon, Mr. Tay Soon Leong and Mr. Tan Hiap Guan for their assistance.

In fact, it is quite a difficult task to acknowledge all the contributions received from various individuals during the course of this research. I wish to express my gratitude to all of my friends. I acknowledge the support provided by my family members, without which, it is impossible for me to reach to the state where I am now. Their sacrifices and strong moral support are not only a source of motivation and encouragement but also mental stimulation all the way. I wish to express the deepest affection to my beloved daughter 'Roha'.

Finally, I acknowledge the mercy so blessed upon me by The Almighty.

TABLE OF CONTENTS

Acknowledgements.....	i
Table of Contents.....	ii
Abstract.....	vii
List of Figures.....	ix
List of Tables.....	xv
List of Symbols.....	xvi
Abbreviations.....	xix

CHAPTER 1 INTRODUCTION

1.1 Background.....	1
1.2 Objectives and Scope.....	3
1.3 Organization of the Thesis.....	4

CHAPTER 2 LITERATURE REVIEW

2.1 Introduction.....	6
2.2 Seismic Waves.....	7
2.3 Propagation Modes of Rayleigh Wave.....	10
2.4 Variation of Shear Modulus with Strain.....	11
2.4.1 Field Geophysical Methods for Stiffness Measurements.....	12
2.5 Previous Research on Surface Waves.....	14
2.5.1 Surface Wave Method.....	14
2.5.2 Microtremor Survey Method.....	19
2.5.2.1 Method of Analysis.....	21
2.5.2.2 Spatial Auto-Correlation (SPAC) Method.....	22
2.6 Array for Microtremor Survey.....	27
2.7 Number of Stations in the Array.....	27
2.8 Sampling Frequency and Duration of Data Collection.....	29
2.9 Inversion of the dispersion curve.....	29
2.9.1 Wavelength-Depth Method.....	30
2.9.2 Linear Method.....	35

2.10	Seismic Waves in Laboratory Tests.....	39
2.10.1	Small-Strain Stiffness	40
2.11	Stiffness-Strain Relationship of Soil.....	44
2.12	Summary.....	46

CHAPTER 3 CSW AND MICROTREMOR SURVEYS

3.1	Introduction.....	47
3.2	Geology of Singapore	47
3.2.1	Jurong Formation	48
3.2.2	Old Alluvium	49
3.2.3	Bukit Timah Granite	49
3.2.4	Kallang Formation	49
3.3	Field Measurements	51
3.3.1	Continuous Surface Wave System.....	51
3.3.2	Microtremor Survey Method	53
3.4	Description of Test Sites.....	54
3.4.1	Site 1: NTU Site 1	54
3.4.2	Site 2: NTU Site 2.....	55
3.4.3	Site 3: Pioneer Road.....	56
3.4.4	Site 4: Buangkok Drive.....	57
3.4.5	Site 5: Sungei Changi.....	58
3.4.6	Site 6: Orchard	59
3.4.7	Site 7: Chinatown.....	60
3.5	Summary	61

CHAPTER 4 ANALYSIS OF CONTINUOUS SURFACE WAVE TEST RESULTS

4.1	Introduction	62
4.2	General Procedure to Obtain the Dispersion Curve.....	63
4.3	Dispersion Curves at the Seven Test Sites.....	65
4.4	Inversion of the Dispersion Curves.....	69

4.4.1	Development of a Basis for λ/z	70
4.4.2	Evaluation of modified wavelength-depth method.....	82
4.5	Summary	93

CHAPTER 5 ANALYSIS OF MICROTREMOR SURVEY RESULTS USING SPAC METHOD

5.1	Introduction	94
5.2	Estimating the Dispersion Curve	95
5.3	Case History 1: Estrella and González (2003)	96
5.3.1	Description	96
5.3.2	Estimating the Dispersion Curve	97
5.3.3	Estimating the Shear Wave Velocity Profile	99
5.4	Case History 2: Matsuoka et al. (2003)	100
5.4.1	Description.....	100
5.4.2	Estimating the Dispersion Curve	101
5.4.3	Estimating the Shear Wave Velocity Profile	103
5.5	Case History 3: Bettig et al. (2001)	106
5.5.1	Description	106
5.5.2	Estimating the Dispersion Curve	107
5.5.3	Estimating the Shear Wave Velocity Profile	108
5.6	Case History 4: Saccorotti et al. (1999)	109
5.6.1	Description	109
5.6.2	Estimating the Dispersion Curve	110
5.7	Summary	111
5.8	Dispersion Curves of Microtremor Survey Conducted in Singapore	112
5.9	Comparison of Field Dispersion Curves from CSW and Microtremor Survey Method	116
5.10	Summary	126

CHAPTER 6 ULTRASONIC VELOCITY TEST AND TRIAXIAL TEST

6.1	Introduction	127
-----	--------------------	-----

6.2	Ultrasonic Velocity Test System	128
6.3	Acoustic Coupling	135
6.4	Test Programme for Compacted Soil Samples	136
6.5	Triaxial Apparatus with local Strain Measurement	139
6.6	Test Programme for Undisturbed Soil Samples.....	140
6.7	Summary	141

CHAPTER 7 ANALYSIS OF LABORATORY TEST RESULTS

7.1	Introduction	142
7.2	Effect of Confining Pressure, Degree of Saturation and Soil Type on V_P and V_S	142
7.2.1	Effect of Void Ratio and Confining Pressure on V_P and V_S	145
7.2.2	Relationship of V_P with V_S and Degree of Saturation	151
7.2.3	Evaluation n of Equation 7.15	159
7.3	Ultrasonic Velocity test and Triaxial Tests on Undisturbed Soil Specimen.....	162
7.3.1	Ultrasonic Velocity Tests.....	163
7.3.2	Triaxial Compression Test.....	168
7.3.2.1	CU Test Results for Site 3	169
7.3.2.2	CU Test Results for Site 4	172
7.3.2.3	CU Test Results for Site 5	176
7.3.2.4	CU Test Results for Site 6	181
7.3.2.5	CU Test Results for Site 7	182
7.4	Comparison of Laboratory and Field Measured V_S	188
7.5	Comparison of Laboratory and Field Measured V_S with V_S from Empirical Relationships.....	191
7.6	Summary	196

CHAPTER 8 CONCLUSION AND RECOMMENDATIONS

8.1	Introduction	197
8.2	Conclusion	197

8.2.1	Field Tests.....	197
8.2.2	Laboratory Tests:	199
8.3	Recommendations.....	200
	References.....	202
	Appendix A-Borehole Log	
	Appendix B- Matlab Script File for SPAC method	
	Appendix C-Laboratory Calibration Curves	
	Appendix D-CU test results Shear modulus versus shear strain (Enlarged)	

ABSTRACT

Seismic waves can be employed in the field and in the laboratory for characterizing soil. Seismic waves propagating through soils induce very low level of strains. Using the framework of linear elasticity, the wave propagation velocity can be linked to the elastic parameters of the soil.

This study investigates soil properties using elastic waves in the field as well as in the laboratory. Continuous Surface Wave (CSW) and microtremor surveys were conducted at seven sites in Singapore. Borehole logs were available for all the sites. Pulse transmission tests using ultrasonic velocity test system were conducted in the laboratory. The Spatial Auto-Correlation (SPAC) method was used to analyze the data from microtremor survey to compute the dispersion curve. The dispersion curves obtained from the CSW and microtremor surveys for the same site were not coincidental but the discrepancy did not have any significant effect on the shear wave velocity profile. To obtain the shear wave velocity profile from the dispersion curve, wavelength-depth factor and linear inversion methods were used. The depth of investigation obtained from microtremor surveys was limited by array size. Because of site constraint, the array size of the microtremor is limited to 10 m with the maximum depth of investigation limited to 16 m. A modified wavelength-depth method was developed in this research and the method was verified using data from published literature. The modified wavelength-depth method was found to provide a better estimate of the shear wave velocity profile which can be used as the initial model for more accurate inversion schemes.

Ultrasonic velocity tests were conducted on compacted unsaturated soil samples to investigate the effect of degree of saturation and confining pressure on compression wave and shear wave velocities. It was found that degree of saturation did not have significant effect on shear wave velocity but compression wave velocity was found to be dependent on degree of saturation. A relationship between compression wave velocity with shear wave velocity and degree of saturation was developed. The relationship could be used to estimate compression wave velocity of an unsaturated soil/rock/concrete within $\pm 20\%$ error.

Ultrasonic velocity tests and triaxial compression tests were conducted on undisturbed soil samples to measure the small-strain stiffness and stiffness with strain relationship of soil. The compression wave velocity and shear wave velocity were measured by ultrasonic velocity test. The measured shear wave velocity by ultrasonic velocity test was compared with the shear wave velocity obtained from CSW and microtremor survey methods. The comparison showed good agreement with a discrepancy of $\pm 30\%$. The small-strain stiffness of soil was measured by triaxial compression test using local strain measurement transducers. The effects of void ratio, confining pressure and plasticity index on the stiffness-strain relationship of soil were investigated. It was found that increase in void ratio reduces the stiffness of soil, however, increase in confining pressure and plasticity index increase the shear stiffness of soil. It was also found that the effects of void ratio and confining pressure are more significant than the effect of plasticity index on stiffness-strain relationship of soil. Some empirical relationships were used in this study to compare with the shear wave velocities obtained by CSW survey method. It was found that the discrepancy between shear wave velocities from empirical relationships and CSW survey method is from $\pm 17\%$ to $\pm 40\%$.

LIST OF FIGURES

- Figure 2.1 Distribution of elastic waves in the earth surface (from Woods, 1968)
- Figure 2.2 (a) Soil before wave propagation, (b) P-wave, and (c) S-wave
- Figure 2.3 Surface waves (a) Love wave, and (b) Rayleigh wave (from Bolt, 1988)
- Figure 2.4 Particle motion in Rayleigh waves (from Richart et al., 1970)
- Figure 2.5 Variation of the elastic wave velocity with Poisson's ratio
- Figure 2.6 Phase velocity from different modes of Rayleigh wave propagation
- Figure 2.7 Variation of shear modulus of soil with experimental methods (from GDS Instruments Ltd., 1998)
- Figure 2.8 Surface and subsurface field geophysical methods
- Figure 2.9 Schematic diagram showing the steps followed in the determination of dispersion curve using the SASW method (from Matthews et al., 1996)
- Figure 2.10 Schematic diagram of obtaining dispersion curve from CSW
- Figure 2.11 Illustration of phase difference
- Figure 2.13 Variation of the vertical and horizontal motion of Rayleigh wave
- Figure 2.14 Geometric dispersion of Rayleigh wave (from Rix, 2005)
- Figure 2.15 Variation of λ/z with frequency for four heterogeneous deposits: $m = 1, 2/3, 1/2$ and $1/4$ (modified from Gazetas, 1982)
- Figure 2.16 Range of λ/z for $0.25 \leq \nu \leq 0.90$ and $0.25 \leq H_0 \leq 0.98$ and for fundamental wave mode (modified from Vrettos, 1990)
- Figure 2.17 Different transmission mode in ultrasonic velocity test
- Figure 2.18 Velocity determination in ultrasonic test
- Figure 2.19 First arrival time detection from time series data
- Figure 2.20 Typical stiffness-strain behaviour of soils for laboratory tests and structures (from Baranski, 2003)
- Figure 3.1 Location of experimental sites (modified from PWD, 1976)
- Figure 3.2 Schematic of CSWS from GDS instruments Ltd. (1998)
- Figure 3.3 Geophone arrangement in microtremor survey method
- Figure 3.4 Location of Site 1 and Site 2
- Figure 3.5 Field experimental arrangement for Site 1
- Figure 3.6 Field experimental arrangement for Site 2
- Figure 3.7 (a) Location of Site 3 (b) Field experimental arrangement for Site 3

- Figure 3.8 (a) Location of Site 4 (b) Field experimental arrangement for Site 4
- Figure 3.9 (a) Location of Site 5 (b) Field experimental arrangement for Site 5
- Figure 3.10 (a) Location of Site 6 (b) Field experimental arrangement for Site 6
- Figure 3.11 (a) Location of Site 7 (b) Field experimental arrangement for Site 7
- Figure 4.1 Slope angle of phase angles by least square method
- Figure 4.2 Dispersion curve by CSW method
- Figure 4.3 Dispersion curves from Site 1
- Figure 4.4 Dispersion curve from Site 2
- Figure 4.5 Dispersion curve from Site 3
- Figure 4.6 Dispersion curve from Site 4
- Figure 4.7 Dispersion curve from Site 5
- Figure 4.8 Dispersion curve from Site 6
- Figure 4.9 Dispersion curve from Site 7
- Figure 4.10 (a) Site 1 - Layering of soil strata by best-fit lines
(b) Site 2 - Layering of soil strata by best-fit lines
(c) Site 3 - Layering of soil strata by best-fit lines
(d) Site 4 - Layering of soil strata by best-fit lines
(e) Site 5 - Layering of soil strata by best-fit lines
(f) Site 6 - Layering of soil strata by best-fit lines
(g) Site 7 - Layering of soil strata by best-fit lines
- Figure 4.11 Dispersion curve and shear wave velocity model from Louie (2001)
- Figure 4.12 Dispersion curve and shear wave velocity model from Menzies (2001)
- Figure 4.13 Dispersion curve and shear wave velocity model from Luke and Calderón-Macías (2007)
- Figure 4.14 Dispersion curve and shear wave velocity model from Ganji et al. (1998)
- Figure 4.15 Dispersion curve and shear wave velocity model from Anand (2000)
- Figure 4.16 Wavelength-depth ratio for maximum wavelength of investigation
- Figure 4.17 Relationship of n with frequency f
- Figure 4.18 Chart for modified wavelength-depth method
- Figure 4.19 Evaluation of modified wavelength-depth method for data from Tokimatsu et al. (1992)
- Figure 4.20 Evaluation of modified wavelength-depth method for data from Tokimatsu and Miyadera (1992)

- Figure 4.21 Evaluation of modified wavelength-depth method for data from Tokimatsu et al. (1995)
- Figure 4.22 Evaluation of modified wavelength-depth method for data from Tokimatsu and Arai (1995)
- Figure 4.23 Evaluation of modified wavelength-depth method for Kusakabe site from Matsuoka et al. (1996)
- Figure 4.24 Evaluation of modified wavelength-depth method for Omiya site from Matsuoka et al. (1996)
- Figure 4.25 Evaluation of modified wavelength-depth method for Nishi site from Matsuoka et al. (1996)
- Figure 4.26 Evaluation of modified wavelength-depth method for Ambarli station site from Rosenblad et al. (2002)
- Figure 4.27 Evaluation of modified wavelength-depth method for Ballica station site from Rosenblad et al. (2002)
- Figure 4.28 Evaluation of modified wavelength-depth method for Cekmece station site from Rosenblad et al. (2002)
- Figure 4.29 Evaluation of modified wavelength-depth method for Duzce station site from Rosenblad et al. (2002)
- Figure 4.30 Evaluation of modified wavelength-depth method for LD-10 station site from Rosenblad et al. (2002)
- Figure 5.1 Fitted Bessel function J_0 with spatial auto-correlation coefficients
- Figure 5.2 Dispersion curve calculated using SPAC method
- Figure 5.3 Arrangement of the geophones for Case History 1
- Figure 5.4 Spatial auto-correlation coefficients for Case History 1 (from Estrella and González, 2003)
- Figure 5.5 Fitted Bessel functions J_0 for Case History 1
- Figure 5.6 Comparison of dispersion curves for Case History 1
- Figure 5.7 Comparison of shear wave velocity profiles for Case History 1
- Figure 5.8 Arrangement of geophones for Case History 2 (from Matsuoka et al., 2003)
- Figure 5.9 Fitted Bessel function J_0 for Case History 2
- Figure 5.10 Comparison of estimated dispersion curve for Case History 2
- Figure 5.11 Comparison of spatial auto-correlation coefficients and back-calculated spatial auto-correlation coefficients for Case History 2.

- Figure 5.12 Comparison of refitted Bessel function J_0 for Case History 2
- Figure 5.13 Comparison of dispersion curves for Case History 2
- Figure 5.14 Dispersion curves for different sites (from Matsuoka et al., 2003)
- Figure 5.15 Comparison of shear wave velocity profile for Case History 2
- Figure 5.16 Microtremor survey array for Case History 3. The triangles show the location of the stations (large array); the small circle in the upper part of the figure corresponds to the small array location (from Bettig et al., 2001).
- Figure 5.17 Comparison of spatial auto correlation coefficients for Case History 3
- Figure 5.18 Comparison of dispersion curves for Case History 3
- Figure 5.19 Comparison of shear wave velocity profiles for Case History 3
- Figure 5.20 Layout of array for Case History 4 (from Saccorotti et al., 1999)
- Figure 5.21 Fitted Bessel function J_0 for Case History 4
- Figure 5.22 Comparison of dispersion curve for Case History 4
- Figure 5.23 Dispersion curve for Site 1
- Figure 5.24 Dispersion curves for Site 2
- Figure 5.25 Dispersion curve for Site 3
- Figure 5.26 Dispersion curve for Site 4
- Figure 5.27 Dispersion curve for Site 5
- Figure 5.28 Dispersion curve for Site 6
- Figure 5.29 Dispersion curve for Site 7
- Figure 5.30 Comparison of field dispersion curves for Site 1
- Figure 5.31 Comparison of field dispersion curves for Site 2
- Figure 5.32 Comparison of field dispersion curves for Site 3
- Figure 5.33 Comparison of field dispersion curves for Site 4
- Figure 5.34 Comparison of field dispersion curves for Site 5
- Figure 5.35 Comparison of field dispersion curves for Site 6
- Figure 5.36 Comparison of field dispersion curves for Site 7
- Figure 5.37 Dispersion curve from Site 2 for different array sizes ($r = 5$ m and $r = 10$ m)
- Figure 5.38 Shear wave velocity profiles for Site 1
- Figure 5.39 Shear wave velocity profiles for Site 2
- Figure 5.40 Shear wave velocity profiles for Site 3
- Figure 5.41 Shear wave velocity profiles for Site 4

- Figure 5.42 Shear wave velocity profiles for Site 5
- Figure 5.43 Shear wave velocity profiles for Site 6
- Figure 5.44 Shear wave velocity profiles for Site 7
- Figure 6.1 Ultrasonic velocity test system (from Leong et al., 2004)
- Figure 6.2 Arrangement of compression and shear wave transducers
- Figure 6.3 Frequency spectra of compression waves (face-to-face)
- Figure 6.4 Frequency spectra of shear waves (face-to-face)
- Figure 6.5 Variation of travel time with length of aluminium
- Figure 6.6 Variation of travel time with length of mild steel
- Figure 6.7 Variation of travel time with length of nylon
- Figure 6.8 Receiver signal for P-wave (face-to-face).
- Figure 6.9 Receiver signal for S-wave (face-to-face).
- Figure 6.10 Frequency spectrum for compression wave with and without couplant
- Figure 6.11 Frequency spectrum for shear wave with and without couplant
- Figure 6.12 Grain size distributions of sand and kaolin
- Figure 6.13 Compaction curves of soil samples
- Figure 7.1 Effect of degree of saturation on V_p
- Figure 7.2 Effect of degree of saturation on V_s
- Figure 7.3 V_p/V_s with degree of saturation
- Figure 7.4 V_p^* with degree of saturation
- Figure 7.5 V_s^* with degree of saturation
- Figure 7.6 V_s^* with void ratio
- Figure 7.7 Bulk modulus of a fluid mixture of air ($K_a = 142$ kPa) and
- Figure 7.8 C_s versus S plot (from Yeo, 2005)
- Figure 7.9 Variation for V_p with S (from Wyllie et al., 1956)
- Figure 7.10 Variation for sound speed with gas volume fraction (from Lorenz, 2002)
- Figure 7.11 C_s from experimental data and fitted curve
- Figure 7.12 Evaluation of Equation 7.15 for different soils
- Figure 7.13 P- and S-waves for soil specimen S3D10.15
- Figure 7.14 P- and S-wave for soil specimen S3D16.6
- Figure 7.15 P- and S-waves for soil specimen S4D12.3
- Figure 7.16 P- and S-waves for soil specimen S4D21.2

- Figure 7.17 P- and S-waves for soil specimen S4D22.5
- Figure 7.18 P- and S-waves for soil specimen S5D8.9
- Figure 7.19 P- and S-waves for soil specimen S5D14.9
- Figure 7.20 P- and S-waves for soil specimen S5D17.85
- Figure 7.21 P- and S-waves for soil specimen S5D20.85
- Figure 7.22 P- and S-waves for soil specimen S6D12.35
- Figure 7.23 P- and S-waves for soil specimen S7D12.5
- Figure 7.24 P- and S-waves for soil specimen S7D21.5
- Figure 7.25 CU test results for soil specimen S3D10.15
- Figure 7.26 CU test results for soil specimen S3D16.6
- Figure 7.27 CU test results for soil specimen S4D12.3
- Figure 7.28 CU test results for soil specimen S4D21.2
- Figure 7.29 CU test results for soil specimen S4D22.5
- Figure 7.30 CU test results for soil specimen S5D8.9
- Figure 7.31 CU test results for soil specimen S5D14.9
- Figure 7.32 CU test results for soil specimen S5D17.85
- Figure 7.33 CU test results for soil specimen S5D20.85
- Figure 7.34 CU test results for soil specimen S6D12.35
- Figure 7.35 CU test results for soil specimen S7D12.5
- Figure 7.36 CU test results for soil specimen S7D21.5
- Figure 7.37 Effect of void ratio
- Figure 7.38 Effect of confining pressure
- Figure 7.39 Effect of PI
- Figure 7.40 Comparison of shear wave velocity profile for Site 3
- Figure 7.41 Comparison of shear wave velocity profile for Site 4
- Figure 7.42 Comparison of shear wave velocity profile for Site 5
- Figure 7.43 Comparison of shear wave velocity profile for Site 6
- Figure 7.44 Comparison of shear wave velocity profile for Site 7
- Figure 7.45 Comparison between field and laboratory measured V_S
- Figure 7.46 Comparison of V_S for Site 3
- Figure 7.47 Comparison of V_S for Site 4
- Figure 7.48 Comparison of V_S for Site 5
- Figure 7.49 Comparison of V_S for Site 6
- Figure 7.50 Comparison of V_S for Site 7

LIST OF TABLES

Table 2.1	Array size for microtremor survey
Table 2.2	Sampling frequency for microtremor survey
Table 2.3	Factors affect $G_{\max}$ (from Kramer, 1996)
Table 3.1	Summary of dynamic soil properties (Veijayaratnam et al., 1993)
Table 4.1	Phase angles for different frequencies
Table 4.2	Wavelength-depth factor for the seven sites
Table 4.3	Four-layer model for Case 1 (Tokimatsu et al. 1992)
Table 6.1	Face delays for P- and S-waves
Table 6.2	Comparison of V_P
Table 6.3	Comparison of V_S
Table 6.4	Physical properties of compacted soil samples
Table 6.5	Conditions of compacted soil specimens for ultrasonic test
Table 6.6	Properties of undisturbed soil sample
Table 6.7	Initial conditions of triaxial test
Table 7.1	Constants n and A
Table 7.2	Ultrasonic test results and soil properties
Table 7.3	Representative values of Poisson's ratio (from Das, 1998, Bowles, 1997 and BS EN 1992-1-1:2004)
Table 7.4	Ultrasonic test results and properties concrete specimen
Table 7.5	Comparison of V_P (measured) and V_P (calculated)
Table 7.6	Properties of Undisturbed soil sample
Table 7.7	Initial and final conditions of triaxial test specimens
Table 7.8	Elastic moduli E_{initial} , E_{ur} and E_{50}
Table 7.9	Grouping of undisturbed soil specimens to study the effects of e , σ_3 and PI
Table 7.10	Comparison of field and laboratory measured V_S
Table 7.11	Comparison of field and laboratory measured V_S with V_S computed by empirical relationships

LIST OF SYMBOLS

A	Amplitude of the wave motion
B	Pore pressure parameter
a	Constant
$c(f)$	Phase velocity to linear frequency
$c(\omega)$	Phase velocity with respect to angular frequency
CH	Inorganic clays of high plasticity
CL	Inorganic clays of low to medium plasticity
δ	Dirac function
e	Void ratio
E	Modulus of elasticity
ε_a	Axial strain (%)
ε_r	Radial strain (%)
f	Frequency
Fe	Void ratio homogenization factor
ϕ	Phase angle
G	Shear modulus
G_0	Shear modulus at the surface
G_α	Shear modulus at infinite depth
G_{max}	Initial shear modulus
G_s	Specific gravity
γ_{th}	Threshold stain
γ	Shear strain
h	Layer thickness
H	Thickness of the soil deposit
j	Imaginary number ($j = \sqrt{-1}$)
J_0	Bessel function of the first kind of zero order
k	Wave number
L	Length of the specimen
LL	Liquid Limit
λ	Wavelength
λ/z	Wavelength-depth factor

m	Type of heterogeneity
M	Constraint modulus
μ	Rate of heterogeneity
μs	Micro second
N	Standard Penetration Test (SPT) number
ν	Poisson's ration
ω	Angular frequency
Ω	Dimensionless frequency
P	Pressure
p	Elastic constant
p'	Effective means stress
P	Elastic constant
P_{air}	Air pressure
PL	Plastic Limit
PI	Plasticity Index
P_a	Atmospheric pressure
q	Deviator stress ($\sigma_1 - \sigma_3$)
r	Radius of the array
ρ	Bulk density
ρ_a	Bulk density of air
ρ_w	Bulk density of water
s	Second
S	Degree of saturation
σ	Stress
σ_1	Axial stress
σ_3	Confining pressure
t	Duration of the wave
T	Period of the wave
u_w	Excess pore-water pressure
v	Volume
V	Velocity
V_P	P-wave velocity or Primary/compression wave velocity

V_R	Rayleigh wave velocity
V_S	S-wave velocity or Secondary/shear wave velocity
v_a	P-wave velocity of air
v_w	P-wave velocity of water
w	Water content
W_{water}	Weight of dissolved water
z	Depth

ABBREVIATIONS

BKTF	Bukit Timah Formation
CSWS	Continuous Surface Wave System
CU	Consolidated Undrained
F-K	Frequency-Wavenumber
GCTS	Geotechnical Consulting and Testing System
H/V	Horizontal/Vertical
JF	Jurong Formation
KF	Kallang Formation
LDT	Local Deformation Transducers
LVDT	Linear Voltage Differential Transducer
MASW	Multi-Channel Analysis of Surface Waves
NTU	Nanyang Technological University
OA	Old Alluvium
PSD	Power Spectral Density
PWD	Public Works Department
RMS	Root Mean Square
SASW	Spectral Analysis of Surface Wave
SCPT	Seismic Cone Penetration Test
SPAC	Spatial Auto-correlation Coefficient
SPT	Standard Penetration Test
USCS	Unified Soil Classification System

CHAPTER 1

INTRODUCTION

1.1 Background

Seismic waves are caused by natural events such as earthquake, wind action on topography or vegetation, oceanic waves or cyclone or caused by human activities such as explosions, rotating or hammering machinery and traffic. Seismic waves are mainly of two types: body waves (Primary waves or P-waves and Secondary waves or S-waves), and surface waves (Rayleigh waves and Love waves). Body waves propagate outward in all directions from a source and travel inside the earth. Surface waves propagate approximately parallel to the earth's surface and do not penetrate directly into the earth. Seismic waves are very useful to geophysicists as well as geotechnical engineers since it carries very useful information about soil properties. Geophysicists have long used seismic wave to characterize the earth's mantle. In recent decades, geotechnical engineers have started to use seismic waves for site characterization. Seismic waves have been employed in the field for site investigation and in the laboratory for soil properties determination.

Site characterization involves evaluation of soil stratification and soil properties. Increasingly geophysical methods are being used in site investigation. Most geophysical methods make use of seismic waves. The need for accurate, fast and cost-effective site characterization is becoming more important as development is now occurring at a faster pace. The great advantage of geophysical techniques is that it offers a rapid, noninvasive, and cost-effective way to determine soil properties and covers large area of investigation. In geotechnical engineering, geophysical tests have the advantage of testing soil in the in-situ condition. Generally there are two kinds of field geophysical measurements: surface methods (e.g. refraction, reflection, surface wave) and subsurface methods (e.g. up-hole, down-hole, cross-hole, seismic cone). Surface methods have the advantage of being non-invasive i.e. do not require any borehole. Surface wave propagates along the surface of

the earth and is dispersive in nature. Surface waves of different frequencies have different penetration depths and propagate with different phase velocities (Bath, 1973).

Geophysical tests using surface wave can be classified into two broad categories based on source of the waves: active source and passive source. An active source uses an artificial source of excitation, e.g. hammer or vibrator. Passive source uses ambient seismic vibrations. Active source of surface wave method can be performed in different ways: Spectral Analysis of Surface Wave (SASW) method uses an impact source like a sledge hammer as an energy source and usually only two receivers (geophones). It is therefore usually necessary to repeat the test many times to ensure surface waves of different frequencies are collected and to reduce random disturbances during the measurements; Multi-Channel Analysis of Surface Waves (MASW) method has been used to overcome some of the disadvantages of the SASW method. The main difference between MASW and SASW is the number of geophones used, usually twelve or more in MASW method. In MASW, the surface waves generated by an impact source is simultaneously recorded by several geophones and enables the signals to be analyzed in both time and space domain using a double Fourier Transform. The evaluation of the dispersion curve becomes more straightforward and the procedure can be automated (Foti, 2000). Using MASW, the need for multiple test configuration required in SASW is avoided with the result that duration of testing and interpretation can be greatly reduced. Continuous Surface Wave (CSW) method uses a steady-state vibrator which can create surface waves of different frequencies as an energy source. The CSW method has the major advantage of frequency control plus good frequency resolution. However, the depth of investigation using surface wave is limited by the strength of the energy source. In recent years, the use of passive source surface wave method, such as microtremor survey, has gained considerable attention. The use of microtremors i.e. low frequency background seismic noise for characterizing site appears promising in overcoming the penetration depth limitation of surface wave method using active source.

Seismic waves propagating through soils induce very low level of strains. Using the framework of linear elastic laws, wave propagation velocity can be linked to the elastic

parameters of the medium. Over the past two decades, there have been significant developments in the understanding of small-strain stiffness of soils. Small-strain stiffness can be determined in the laboratory via resonant column test or pulse transmission test. Pulse transmission method has become more popular in recent years. In pulse transmission test, both P-wave velocity, V_P , and S-wave velocity, V_S , can be determined. The small-strain stiffness obtained from pulse transmission test can be compared with the results from geophysical methods.

This thesis investigates the application of seismic waves for soil properties determination. The thesis is divided into two major parts: (1) Application of seismic waves for site characterization using CSW method and microtremor survey, and (2) Application of seismic waves for determination of small-strain stiffness in the laboratory. Comparison between field and laboratory measured shear wave velocity profile or stiffness profile is made. The relationship of compression wave velocity with shear wave velocity and degree of saturation is investigated. The role of small-strain stiffness in the stiffness-strain relationship of soil is also investigated.

1.2 Objectives

The main objective of this research is to investigate the use of seismic waves for the characterization of soil properties. This is accomplished through the following:

- (1) To characterize soil profile using CSW and microtremor surveys at a number of sites with different geological formation in Singapore.
- (2) Obtain the shear wave velocity structure of a soil profile using different inversion methods and hence to develop a simpler method of inversion.
- (3) Investigate small-strain stiffness using laboratory pulse transmission method
- (4) Investigate the variation of wave velocity with different degree of saturation.
- (5) Investigate the relationship between shear wave velocity from field geophysical tests and laboratory ultrasonic velocity test.
- (6) Investigate the relationship between small-strain stiffness from seismic velocities and stiffness from triaxial compression tests using local strain measurement.

1.3 Organization of the Thesis

This thesis is organized into eight chapters:

Chapter 1 provides the background of the research. In Chapter 1, the objectives of the study as well as overview of the other chapters of the thesis are described.

Chapter 2 presents the literature review related to this research. It discusses the properties of seismic waves, previous research on CSW and microtremor survey methods as well as their theoretical background and analytical procedures. The dispersion curve from CSW and microtremor surveys has to be inverted and therefore, different inversion methods and their advantages and disadvantages are reviewed. Chapter 2 also reviews laboratory tests using pulse transmission tests and triaxial compression tests to obtain small-strain stiffness and stiffness-strain relationship of soil. The factors affecting the stiffness-strain relationship of soil are also briefly reviewed.

Chapter 3 presents a brief discussion of Singapore geology especially of the field sites where CSW and microtremor surveys were conducted and undisturbed soil samples were collected. This chapter also introduces the Continuous Surface Wave System (CSWS) used in the CSW and microtremor survey. The detailed procedures of CSW and microtremor survey methods are presented in Chapter 3.

Chapter 4 discusses the general procedure to generate a dispersion curves followed by the presentation of the dispersion curves from the seven sites where CSW and microtremor surveys were conducted for this study. In Chapter 4, a modified wavelength-depth method is developed. Finally, the modified wavelength-depth method is evaluated using case histories.

In Chapter 5, the calculation procedures of SPAC method and the linear inversion method used in this study are evaluated. The evaluation uses data from the literature. The shear

wave velocity profile obtained from seven sites in Singapore, using CSW and microtremor survey methods, are compared with borehole log in this chapter.

Chapter 6 introduces the ultrasonic velocity test system, the triaxial apparatus with local strain measurement transducers followed by a description of soils, test procedures and test programmes.

Chapter 7 presents the results of ultrasonic velocity tests on compacted soil samples. The relationship of V_P with V_S and degree of saturation is investigated. Chapter 7 also presents the results of ultrasonic velocity tests on undisturbed soil samples from five sites. The effects of void ratio, confining pressure and plasticity index on stiffness-strain relationships from triaxial test with local strain measurement are also investigated in this chapter.

Chapter 8 summarizes the important findings of this study followed by recommendations for future research.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Seismic waves are very common occurrences in the earth's surface and carry valuable information which can be used to characterize the properties of soil mass (Aki, 1957; Asten, 1978; Capon, 1969; Horike, 1985; Ikegami, 1964; Kudo et al., 1976; Locoss et al., 1969; Matthews and Clayton, 1993; Nakamura, 1989; Okada, 2003; Rosenblad et al., 2002; Stokoe, 1984; Stokoe et al., 1994; Tokimatsu, 1997; Toksöz, 1964; Zywicki, 1999). To date, many researches have been performed using seismic waves to characterize properties of soil. However, most of the researches were performed by geophysicists. Until the recent decades, seismic waves did not receive much attention in characterizing soil properties.

This chapter will discuss the properties of seismic waves, previous research on surface wave methods as well as their theoretical background and analytical procedures. The data obtained from CSW and microtremor survey methods are used to produce the dispersion curve, a plot of phase velocity versus wavelength or frequency. In order to obtain a shear wave velocity or stiffness profile, the dispersion curve has to be inverted. This chapter will also discuss the different inversion methods.

Seismic waves are also used in laboratory tests to determine small-strain stiffness of soil. One of the most popular laboratory tests employing seismic waves is the pulse transmission test which is also conducted in this study. Therefore a literature review on pulse transmission tests is performed. The soil stiffness determined from seismic wave velocity is usually associated with very small-strain, less than 0.0001% (Nakagawa et al., 1996). Commonly, the very small-strain stiffness is combined with other test result such as cyclic shear or triaxial tests to produce the stiffness-strain relationship of soil. The development of more accurate strain measuring devices such as the local strain transducers enables conventional triaxial compression tests to yield the stiffness-strain relationship from 0.0001% to 0.1% to be measured. Such tests are also a part of this study and thus are also reviewed in this chapter.

2.2 Seismic Waves

The propagation of seismic waves is achieved by spontaneous distortion of the soil volume without any permanent deformation. The basic features of the wave field at a relatively large distance from a vibrating circular footing are shown in Figure 2.1. The particles in the medium of propagation oscillate whereby the displacement relative to its original position is very small. Seismic waves are associated with very small-strain, less than 0.0001%. Therefore, seismic waves are elastic waves.

Figure 2.1 Distribution of elastic waves in the earth surface (from Woods, 1968)

All elastic waves contain two different kinds of motions: the particle motion and the wave motion (Figure 2.2). Elastic waves in the soil volume are usually divided into body waves and surface waves. Body waves are the waves that propagate through the entire body of the soil mass (Figure 2.2). Body waves can be further subdivided into primary waves (P-waves), also known as compression or longitudinal waves (Figure 2.2b) and secondary waves (S-waves), also known as shear or transverse waves (Figure 2.2c). The main difference between P-waves and S-waves is the particle motion in the propagation medium. In the case of P-waves, particle motions are parallel to the direction of wave propagation, whereas in the case of S-waves, particle motions are perpendicular to the direction of the wave propagation. S-waves have been studied extensively in earthquake seismology since the beginning of the twentieth century.

Figure 2.2 (a) Soil before wave propagation, (b) P-wave, and (c) S-wave (from Santamarina, 2001)

Surface waves travel with much larger amplitudes than body waves and their velocities are frequency dependent (i.e. dispersive). They are very important in seismic problems. Most building damages during earthquakes are caused by surface waves. Two common types of surface waves are Love waves and Rayleigh waves (Figure 2.3).

Figure 2.3 Surface waves (a) Love wave, and (b) Rayleigh wave (from Bolt, 1988)

Love waves are horizontally polarized surface waves. They exist only in layered earth where the shear wave velocity of the top layer is smaller than that of the substratum. It travels somewhat slower than the Rayleigh waves. Because of their complexities, Love waves are not as heavily used for geophysical analyses, as P-waves, S-waves or

Rayleigh waves. It has been found that Love wave dispersion contributes less information about the earth structure than Rayleigh wave dispersion (Der et al., 1970; Braile and Keller, 1975).

Particle motions in Rayleigh wave are a combination of compression and shear waves. That is the particle motion has two components, one is perpendicular and the other is parallel to the direction of wave propagation. From Figure 2.4, it can be seen that for a soil medium with Poisson's ratio of 0.25, the horizontal particle motion of the Rayleigh wave at the surface of the medium is 68% of the vertical particle motion. Therefore, it has relatively high amplitude compare to body waves and is easier to detect in a noisy area. The horizontal particle motion diminishes faster than the vertical particle motion. At a depth of 0.2λ where λ = wavelength, the horizontal particle motion is on the opposite side of the zero horizontal line, that is particle motion goes from retrograde to prograde. Due to its large amplitude vertical particle motion, Rayleigh waves are usually responsible for most of the earthquake damages. Ambient ground noise in urban areas is largely due to Rayleigh waves resulting from the passage of vehicles over uneven road surfaces, and coupling of tall structures with wind and air waves (Horike, 1985; Liu et al., 2000; Satoh et al., 2001a,b).

Figure 2.4 Particle motion in Rayleigh waves (from Richart et al., 1970)

The speed of wave propagation is dependent on Poisson's ratio (Figure 2.5). Rayleigh waves travel at a velocity of about 90% of the S-waves velocity. P-waves are the fastest moving wave compare to other elastic waves in any seismic event. As soil becomes saturated (i.e. Poisson's ratio tends to 0.5), there is P-wave transmission through the fluid and the soil structure. However, since pore water has no rigidity to shear, the shear wave velocity in the soil is dependent only on the properties of the soil skeleton. The relationship between the Rayleigh wave velocity (V_R) with P-wave velocity (V_P) and S-wave velocity (V_S), which is used for homogeneous half-space, can be expressed by the following equation (Achenbach, 1975):

$$\left[2 - \left(\frac{V_R}{V_S} \right)^2 \right]^2 - 4 \left[1 - \left(\frac{V_R}{V_P} \right)^2 \right]^{\frac{1}{2}} \left[1 - \left(\frac{V_R}{V_S} \right)^2 \right]^{\frac{1}{2}} = 0 \quad (2.1)$$

Figure 2.5 Variation of the elastic wave velocity with Poisson's ratio
(from Richart, 1962)

2.3 Propagation Modes of Rayleigh Wave

Rayleigh wave can propagate with different modes, such as fundamental mode, second mode, third mode and so on. The dispersion will be different based on the propagation mode of Rayleigh wave as shown in Figure 2.6.

The fundamental mode of Rayleigh waves travels with lowest phase velocity. That is the higher the mode of wave propagation will mean the lower the depth of penetration. In most surface wave analyses, Rayleigh wave is assumed to propagate in fundamental mode. Zhang and Chan (2003) found that only the fundamental mode could give an accurate soil profile. The penetration depth of Rayleigh wave and accuracy of the inversion model are particularly sensitive to phase velocities at low frequencies of the dispersion curve. Therefore, it is very important that the dispersion curve, which is used for inversion to determine the shear wave velocity profile, to be of the fundamental mode of Rayleigh wave propagation.

Figure 2.6 Phase velocity from different modes of Rayleigh wave propagation

2.4 Variation of Shear Modulus with Strain

In the above sections, the properties of elastic wave were discussed. However, the main objective of this research is to characterize the soil properties using elastic waves in the soil. Stiffness is one of the important properties of soil. Shear modulus G is a representation of soil stiffness and is dependent on the applied strain. The value of G varies with strain level as shown in Figure 2.7. It can be observed from Figure 2.7 that different experimental methods measured G within a limited strain range. Field geophysical methods can measure G at very low strains, less than 0.0001%, i.e. G_{max} as indicated in Figure 2.7.

Figure 2.7 Variation of shear modulus of soil with experimental methods
(from GDS Ltd., 1998)

Field geophysical tests can be conducted in-situ and can cover a large area. On the other hand, laboratory methods require soil samples that may be disturbed due to sampling and can measure soil properties only at a particular depth. The values of G_{max} measured in field geophysical tests and laboratory tests may not always agree. Few studies have been performed to compare G_{max} measured in the field geophysical tests and laboratory tests. In order to understand the differences, this research will carry out field geophysical tests as well as laboratory tests to determine G_{max} .

2.4.1 Field Geophysical Methods for Stiffness Measurements

Geophysical methods employed to determine shear wave velocity profile of soil can be divided into surface and subsurface or borehole methods (Figure 2.8). The surface methods include refraction, reflection, surface wave and microtremor survey methods. The subsurface methods include up-hole, down-hole, cross hole and seismic cone methods.

Figure 2.8 Surface and subsurface field geophysical methods

The subsurface methods are invasive and usually require boreholes to be bored into the ground for the test. Subsurface methods are preferred over surface methods as the extracted soil can be observed physically. However, subsurface methods are generally of high cost, cannot be applied to deeper strata of soils and sometimes are not suitable in urban areas.

Surface methods are non invasive and are easier to perform in the field as these methods do not require any boreholes in the ground. Moreover, these methods are attractive for speed, low cost and do not have any problem associated with penetration due to obstructions such as boulders, rock etc. The surface methods only need receivers (seismometers or geophones) to be placed on the ground surface to collect data.

From the above discussions, surface methods have certain advantages over subsurface methods. This study will involve geophysical tests using surface waves. However, surface wave methods using active source have limited depth of investigation. Microtremor survey method using surface waves from passive source have larger depth of investigation. In order to overcome the limited depth of investigation

associated with surface method using active source, CSW and microtremor surveys are investigated in this research.

2.5 Previous Research on Surface Waves

This section reviews the previous researches conducted on surface waves. This section consists of two parts: Part 1 reviews surface wave analyses and Part 2 reviews microtremor analyses.

2.5.1 Surface Wave Method

Surface waves can be generated easily and have high amplitudes compare to body waves. Surface wave method can also detect relatively soft layer underlying a stiff layer (Tokimatsu, 1997). Surface wave method has been widely used in many geotechnical engineering projects for profiling of soil mass (Stokoe et al., 1994). Surface wave method consists of recording of waves generated from a vertical impact source using two or more receivers placed in a linear array and calculating the phase velocities from the phase spectra of the recorded data. The wave source can be either an impact source as in Spectral Analysis of Surface Waves (SASW) or a continuous vibrating source as in Continuous Surface Wave (CSW).

SASW method uses an impact source of seismic energy, recorded repeatedly by a pair of receivers (seismometers or geophones) at small (1 m) to large (500 m) distances. The SASW method assumes that the most energetic arrivals recorded are Rayleigh waves. The procedure for determining the dispersion curve using the SASW is shown schematically in Figure 2.9. The noise in the surface wave measurements can be controlled by using a set of empirical criteria tailored for each site investigated (Stokoe et al., 1994; Gucunski and Woods, 1991). SASW will not yield reliable results where noise overwhelms the power of the artificial source as is common in urban areas, or where body waves are more energetic than Rayleigh waves (Brown, 1998; Sutherland and Logan, 1998). The data in the SASW method are generally analyzed in the frequency domain to produce a dispersion curve by calculating the

phase difference between each receiver (Nazarian and Stokoe, 1984). The receivers used are usually vertical particle-velocity sensors, so shear wave velocity profiles were analyzed on the basis of Rayleigh-wave phase velocities interpreted from the recordings (Louie, 2001). Phase velocities are derived purely from a comparison of amplitude and differential phase spectra computed from each receiver pair for each source excitation, within an FFT (Fast Fourier Transform) oscilloscope (Gucunski and Woods, 1991).

Rosenblad et al. (2002) measured shear wave velocity profiles using the SASW method to depths ranging from 6 m to 45 m, depending on spatial and source considerations. The velocities of Rayleigh waves cannot be separated from those of other wave types in the frequency domain. Boore and Brown (1998) found that SASW consistently under-predicts shear wave velocities for six southern California sites. The major limitation of the SASW technique is a lack of frequency content, control and resolution of the source. The Continuous Surface Wave (CSW) method, on the other hand, uses a steady-state vibrator as an energy source. A sinusoidal input signal generated by a vertical vibrator that produces a series of amplitude modulated pulses at various frequencies is used in CSW method. Such sources have the major advantage of frequency control and hence frequency content with good frequency resolution. The arrangement of the geophones in CSW method is similar to the SASW. The procedure for determining the dispersion curve using the CSW method is shown schematically in Figure 2.10.

Matthews and Clayton (1993) used CSW method to estimate ground deformation. Gordon et al. (1995) and Anand (2001) used CSW method for shear modulus profiling. Lai and Rix (1998) used CSW method to determine the damping ratio of soils. Butcher and Tam (1994) used CSW method to determine landfill thickness. Although the CSW method overcomes the energy source limitation of the SASW method, the maximum depth of investigation is limited as Rayleigh waves have depths of penetration depending on their wavelengths. As the amplitude of Rayleigh waves diminishes exponentially with depth, there is a limitation on the maximum depth of measurement using the CSW method. The CSW method is thus suited to near surface investigation only. The CSW method can provide stiffness profiles in soft

ground up to about 10 m and a maximum depth of 30 m in hard soils and rocks (Matthews et al., 1996).

Figure 2.9 Schematic diagram showing the steps followed in the determination of dispersion curve using the SASW method (from Matthews et al., 1996)

Figure 2.10 Schematic diagram of obtaining dispersion curve from CSW

The basis for the continuous surface wave analysis can be illustrated using Figure 2.11. The wavelength, λ can be expressed as,

$$\lambda = \frac{2\pi d}{\phi} \quad \text{for } d < \lambda \quad (2.2)$$

$$\lambda = \frac{2\pi d}{2\pi n + \phi} \quad \text{for } d > \lambda \quad (2.3)$$

where d is the centre to centre spacing of the geophones in a linear array, ϕ is the phase difference for two consecutive geophones, and n is the number of waves. For most surface wave surveys, Equation 2.3 will not be required as the geophone spacing typically ranged from 0.5 m to 1.0 m.

Figure 2.11 Illustration of phase difference

The Rayleigh wave velocity V_R can be related to frequency f and wavelength λ by

$$V_R = f\lambda \quad (2.4)$$

Plotting the data for V_R versus f , the field dispersion curve can be obtained. However, the shear wave velocity V_S is related to Rayleigh wave, which is used for homogeneous half-space, by the following equation (White, 1955):

$$V_S = pV_R \quad (2.5)$$

where p is a function of Poisson's ratio ν . Typical value of p ranges from 1.088 to 1.047 when ν changes from 0.25 to 0.5. In this study, p is assumed to be 1.07, corresponding to $\nu = 0.33$, when ν is unknown. When shear wave velocity is known, the shear modulus, G_{max} , of the soil can be obtained using the following equation:

$$G_{max} = \rho V_s^2 \quad (2.6)$$

where ρ is the bulk density of the soil.

2.5.2 Microtremor Survey Method

The surface of the earth is always in motion due to natural phenomena (e.g. traffic, machinery, human walking, rain, water flow, local wind, storms and the action of rough surf against an extended steep coast) on the earth surface. These constant vibrations of the earth's surface are called microseisms. Microseisms travel very efficiently along the surface layers of the Earth. In geophysics, these passive waves are known as microtremors. Okada (2003) defined microtremors as ubiquitous, weak and low amplitude of vibration. However, the characteristics of the ground are reflected more or less in its vibration whatever the origin of the vibration (Aki, 1957). Therefore, the ambient vibration in the ground can be used to characterize the soil properties of the ground. Microtremor survey is a method that used microtremors of the ground to determine the soil properties. The amplitudes of microtremors are very small; displacements are in the order of 10^{-4} to 10^{-2} mm (Okada, 2003). Microseisms in the frequency band 1 to 15 Hz propagate principally as multi-mode Rayleigh waves (Asten, 1978). There are four types of microseisms at frequency 0.5 Hz to 3 Hz (Asten, 1978). These are Ocean cyclones that generate P-wave, atmospheric loading and wind action on vegetation that generates undefined modes of wave, coastal effect and cultural sources both of them generates fundamental mode Rayleigh waves.

Microtremor survey consists of recording of ambient noise in the ground by using a number of receivers (geophones) placed in an array. This technique involves the measurement of phase velocities of the fundamental-mode Rayleigh waves and the inversion of these phase velocities to shear wave velocities. Microtremor surveys have been in use for decades. Soil profiling with microtremor survey has been applied to a

number of sites (e.g., Asten et al., 2003; Bettig et al., 2001; Estrella and González, 2003; ; Kudo et al., 2003; Matsuoka et al., 2003; Matsushima and Okada, 1990; Miyakoshi et al., 1996; Okada et al., 1990; Roberts and Asten, 2004; Scherbaum et al., 2002; Tatsuya and Ryohei, 2002; Yamanaka et al., 1995).

The microtremor survey method is based on several assumptions and conditions in its theoretical development that impose a limitation to its applicability (Okada, 2003). Microtremor phenomenon varies both temporally and spatially. However, microtremor survey method assumes both temporal and spatial stationarity of microtremors. Okada (2003) investigated temporal and spatial stationarity characteristic of microtremors. To examine the temporal variation, Okada (2003) recorded data for two different days at different times on each day. The recorded data were then divided into different blocks and plotted against time series to check the temporal variation. It was found that the trend of the data did not vary; only the amplitudes were different for the different time blocks. The spatial stationarity of microtremors was examined by Okada (2003) by plotting Power Spectral Density (PSD) of the recorded data against frequency. Spatial variation of the recorded data was absent when the PSD of the recorded data did not vary. At present, the microtremor survey method is regarded as unsuitable for detailed surveying. However, there are many advantages in microtremor survey method. The advantages of microtremors are (Horike, 1985): (i) Microtremors can be observed at any time and location, (ii) Observation is much easier compared with other exploration methods, (iii) It causes no environmental problems, and (iv) Geological conditions down to a depth of more than 100 m can be investigated if microtremors of required frequency range can be observed.

The basic objective of microtremor survey method is to determine the dispersion curve. The longer the period of microtremors used in the analysis, the deeper the depth of investigation (Okada, 2003). The basic variables used to determine the nature of the soil structure by surface waves, phase velocity c and frequency f , can be related using the following equation (Okada, 2003):

$$F(c, f; V_{P1}, V_{S1}, \rho_1, h_1; V_{P2}, V_{S2}, \rho_2, h_2; \dots, V_{PN}, V_{SN}, \rho_N, h_N) = 0 \quad (2.7)$$

where V_{Pj} , V_{Sj} , ρ_j and h_j are the P-wave velocity, S-wave velocity, density, and thickness of the j -th layer of the soil structure consisting of N layers, respectively. Equation 2.7 cannot be explicitly solved for c as a function of f . However, the solution of Equation 2.7 can be expressed simply as:

$$c = c(f; V_{P1}, V_{S1}, \rho_1, h_1; V_{P2}, V_{S2}, \rho_2, h_2; \dots; V_{PN}, V_{SN}, \rho_N, h_N) \quad (2.8)$$

As surface waves consist of different modes of wave propagation, the solution for the above equation can be for more than one frequency above a certain limit. However, the fundamental mode in most surface waves is dominant. Therefore, in practice, higher mode components greater than the second order is regarded as negligible (Okada, 2003). Hence, Equation 2.8 represents the relationship between fundamental mode phase velocity and frequency, and the velocity c can be regarded as a single-valued function of frequency f .

2.5.2.1 Methods of Analysis

Microtremors contain a multitude of waves, becoming a very complex assemblage of elastic waves comprising not only body waves and surface waves (Toksöz and Lacoss, 1968), but also scattered and diffracted waves. Different methods of analysis have been used to detect the surface waves from microtremors:

- Microtremors were recorded at three points forming a triangular array, and wavelets resembling each other were identified. By reading the wavelets, it was possible to derive apparent periods and velocities (Ikegami, 1964).
- More quantitatively, a band-pass filter was applied to microtremors recorded by a triangular array, to derive the dispersion curve (phase velocity versus frequency) (Kudo et al., 1976).

Using digital signal processing, the autocorrelation of similar wavelets in the triangular array records can be used to derive periods and phase velocities quantitatively. At present, the more popular methods used to analyse surface waves are:

- Spatial Auto-correlation (SPAC) method (Aki, 1957)

- Frequency-Wavenumber (F-K) method (Capon, 1969 and Locoss et al., 1969)
- Horizontal/Vertical (H/V) method (Nakamura, 1989)

In the H/V method, data have to be collected by three-component geophones, which is beyond the scope of this research. So, H/V method will not be discussed further in this thesis. The properties common to both SPAC and F-K methods are that the microtremors are regarded as a stochastic process, and their spectra form the basis of analysis. Both methods observe the vertical component of microtremors in order to extract the Rayleigh wave. However, SPAC method requires lesser number of stations and smaller array sizes compare to F-K method. Okada (2004) suggested that the effective wavelength and the maximum explorative depth for the SPAC method is about 1.5 to 2 times those for the F-K method. Asten (2003) suggested that SPAC method can be used more efficiently when signal arrived from different directions. Furthermore, SPAC method needs smaller array than F-K method to achieve similar result (Okada, 2003). Larger arrays will also affect the assumption of spatial homogeneity and the soil layer might be parallel under the array. In this study, SPAC method was used to analyse microtremor survey data. In the following two subsections, SPAC method and its procedures will be discussed in detail.

2.5.2.2 Spatial Auto-correlation (SPAC) Method

Pioneering works for the determination of velocity structure using microtremors or microseisms were done by Aki (1957), Toksöz (1964) and Locoss et al. (1969). Aki (1957) developed a special array design for microtremor survey consisting of seismographs equally spaced on a circle with one seismograph in the centre. This array overcomes the problem of wave coming from different directions. Henstridge (1979) provided the mathematical background to the signal processing method for circular arrays and explained the spectral representation of the stationary process to which earlier results of Aki (1957) could be analyzed. Matsushima and Okada (1990) and Yamamoto (2000) developed a new algorithm to analyze microtremors data so that Rayleigh wave data can be separated from the Love wave data. Their results showed that S-wave velocity from both Rayleigh waves and Love waves in SPAC analysis gave similar results. More recently, Cho et al. (2004) made further theoretical

improvement to the SPAC method. However, the practical merit of the improvement is hard to prove (Okada, 2006).

The assumptions of the SPAC methods are (Okada, 2003):

- The complex wave motion of microtremors is a stochastic process in time,
- A spatial auto-correlation coefficient for microtremor data as observed with a circular array, can be defined when the waves composing the microtremors are dispersive like surface waves, and
- The spatial auto-correlation coefficient is a function of phase velocity and frequency.

The SPAC method is shown schematically in Figure 2.12. Practical applications of SPAC method can be found in Hidaka (1985), Okada (1983), Okada et al. (1990), Ferrazzini et al. (1991), Ling (1994), Matsuoka et al. (1996 and 2003), Konno and Kataoka (1999), Bettig et al. (2001), Sasatani et al., (2001), Asten et al. (2002, 2003), Asten (2004), Estrella and González (2003), Kudo et al. (2003) and Nguyen et al. (2004).

The theory of SPAC method (Aki, 1957) is described below:

For a single plane wave propagating along the x direction, the spatial auto-correlation function of the signal $u(x,t)$ for the region $x \in [0, X]$ and the time domain $t \in [0, T]$ is defined as:

$$\langle \phi(\xi, t) \rangle_x = u(x, t)u(x + \xi, t)dx \quad (2.9)$$

If the signal is assumed to be stationary in both space and time, Equation 2.9, can be rewritten as:

$$\phi(\xi) = \langle \phi(\xi, t) \rangle_x = \langle \phi(\xi, t) \rangle_t = \frac{1}{T} \int_0^T v_0(t)v_\xi(t)dt \quad (2.10)$$

In the above equation, $v_0(t) = u(x_0, t)$ and $v_\xi(t) = u^*(x_0 + \xi, t)$ are the signals obtained from any two stations separated by a distance ξ .

For the case of a single dispersive wave, Aki (1957) developed an auto-correlation function $\phi(\xi)$ which express the relationship between spectra in time and space:

$$\phi(\xi) = \frac{1}{\pi} \int_0^{\infty} \phi(\omega) \cos\left[\frac{\omega}{c(\omega)} \xi\right] d\omega \quad (2.11)$$

where $\phi(\omega)$ is the auto-correlation frequency spectrum. When the wave is filtered by a narrow-band filter around the frequency ω_0 , $\phi(\omega)$ can be assumed to be

$$\phi(\omega) = P(\omega_0) \delta(\omega - \omega_0) \quad \text{for } \omega > 0 \quad (2.12)$$

where $P(\omega_0)$ is the spectral power density and $\delta(\omega - \omega_0)$ is the Dirac δ function, So Equation 2.12 can be rewritten as:

$$\phi(\xi, \omega_0) = \frac{1}{\pi} P(\omega_0) \cos\left[\frac{\omega_0}{c(\omega_0)} \xi\right] \quad (2.13)$$

Now, if spatial auto-correlation coefficients are defined as $\rho(\xi, \omega_0) = \frac{\phi(\xi, \omega_0)}{\phi(0, \omega_0)}$ then:

$$\rho(\xi, \omega_0) = \cos\left(\frac{\omega_0}{c(\omega_0)} \xi\right) \quad (2.14)$$

This relationship may then be inverted in order to obtain the dispersion curve $c(\omega_0)$. When the direction of propagation of the considered wave is not known, Equation 2.14 should be expressed in a different way. For a circular array of stations, if r is the distance from the centre of the array, ϕ is azimuth of the station and θ is the azimuth of propagation of the wave, Equation 2.14 becomes:

$$\rho(\xi, \omega_0) = \cos\left[\frac{\omega_0}{c(\omega_0)} \cos(\theta - \phi)\right] \quad (2.15)$$

In Equation 2.15, it is seen that $\rho(\xi, \omega_0)$ decreases more rapidly with increasing frequency along the propagation direction ($\phi = \theta$), and that, in an ideal case, it should

be constant along the wave front ($\phi = \theta \pm \pi/2$). Although the graphical representation of $\rho(\xi, \omega_0)$ can give an estimation of the direction of propagation (provided simultaneous measurements are available along different azimuths), in general, θ is not known. It is thus necessary to introduce an azimuthally averaged of the spatial auto-correlation coefficients $\bar{\rho}(\xi, \omega_0)$:

$$\bar{\rho}(\xi, \omega_0) = \frac{1}{\pi} \int_0^\pi \rho(r, \phi, \omega_0) d\phi \quad (2.16)$$

Then for a wave filtered around ω_0 , using Equation 2.15, $\bar{\rho}(\xi, \omega_0)$ is obtained as

$$\bar{\rho}(\xi, \omega_0) = J_0\left[\cos\left(\frac{\omega_0}{c(\omega_0)}\right)\right] \quad (2.17)$$

where J_0 is Bessel function of first kind of zero order given as:

$$J_0(x) = \frac{1}{\pi \int_0^\pi \cos(x \cos(\phi)) d\phi} \quad (2.18)$$

For Rayleigh waves recorded by vertical geophones and dropping the index of frequency ω_0 , Equation 2.15 gives

$$\bar{\rho}(\xi, \omega_0) = J_0\left(\frac{\omega_0 r}{c_R(\omega_0)}\right) \quad (2.19)$$

In the above equation, $c_R(\omega)$ is the phase velocity of the dispersive Rayleigh waves.

The spatial auto-correlation coefficients, $\bar{\rho}(\xi, \omega_0)$, can be measured for several stations located at a distance r with respect to a central station. Knowing the values of $\bar{\rho}(\xi, \omega_0)$ at different frequency ω the phase velocity $c_R(\omega)$ can be calculated from Equation 2.19. The dispersion curve is then obtained as a plot of c_R versus f .

Figure 2.12 Schematic diagram showing the steps followed in the determination of dispersion curve using SPAC method in microtremor survey (from Okada, 2003)

2.6 Array for Microtremor Survey

Array size selection is one of the vital issues to be considered in microtremor survey methods. Array size should be adjusted according to the wavelength range of interest (Arai and Tokimatsu, 2005). The types of array used by other researchers are summarized in Table 2.1. The larger the array size, the deeper the shear wave profile of the soil that can be drawn (Okada 2003). Matsuoka et al. (2003) used different sizes of arrays at the same site and obtained lower depth of investigation for small radius and higher depth of investigation for large radius. From Table 2.1, the size of the array that has been used varies from 50 m to 786 m in radius. The depth of investigation ranges from 80 m to 4000 m.

2.7 Number of Stations in the Array

Yamamoto et al. (1997) investigated extensively on the optimum number of stations for SPAC method of analysis. Different number of stations (three, four, five, six, seven, and eight) at equally spaced circumferential distance with one central station were investigated. They found the error in calculating phase velocity for 3-station circular array with one central station was only 5%. Therefore, they recommended the 3-station circular array with one central station as most practical for microtremor survey when data is analyzed using SPAC method. Cho (2004) also found the 3-station circular array with one central station to be useful in most situations. However, phase velocity estimation becomes more accurate and more reliable with an increasing number of stations. Okada (2006) investigated the number of stations required for a circular array for efficient data collection in terms of analytical efficacy and field effort when SPAC method was used for data analysis. He found that the 3-station circular array with one central station was the most efficient and favourable for observation of microtremors when compared with 4-, 5-, and 9-station circular arrays with one central station. Three-station circular array with one central station was also used by Kudo et al., (2002); Matsuoka et al., (1996); Okada, (2003); Okada et al., (1990). Asten (2004) and Roberts and Asten (2004) used 6-station circular array with one central station. The great advantage this arrangement is that it can give three different radial distances. Therefore, data can easily be checked. In this study, a 3-station circular array with one central station was used.

Table 2.1 Array Size for Microtremor Survey

References	Radii of circular arrays (m)	Depth of investigation (m)	Geophone type	No. of Geophones	Central geophone	Method of analysis	Remarks
Asten et al. (1976)	50	Not Available	Not Available	7	Yes	Single and Double mode SPAC and F-K (BFM)	S-wave velocity profile
Okada and Matsushima (1990)	3, 10 and 28	103	Geophone	Unknown	Yes	SPAC	S-wave velocity profile
Gilberto et al. (1999)	60 to 160	Not Available	At centre 3-component others vertical geophone (4.5 Hz)	12 or 14	Yes	SPAC	
Kudo et al. (1999)	50, 100 and 200	1000	3 Hz Seismograph	5 or 7	Yes	SPAC	S-wave velocity profile and site effect in Turkey
Bettig et al. (2001)	53 to 600	800	5s-3-components Lennartz Le3D sensor	13	Yes	Modified SPAC	P- and S-wave Velocity profile
Asten et al. (2003)	50, 85 and 100	80	Vertical geophone	7	Yes	SPAC	23 m clay, 53m sand
Estrella et al. (2003)	786	4000	Guralp broadband sensors	4	Yes	SPAC	S-wave velocity profile
Matsuoka et al. (2003)	100, 300 and 600	2500 to 3500	Seismometer (GPS receiver)	4	Yes	SPAC	S-wave velocity structure (Bedrock 700 to 3400m)
Yasuia, and Noguchib (2004)	500	150	Geophone	Linear arrangement	No	H/V	S-wave velocity structure (Bedrock at 3400 m)

2.8 Sampling Frequency and Duration of Data Collection

The sampling frequency is related to the maximum frequency to be estimated, and the duration is related to the number of temporal averages used to reduce the variance of the spectral quantities. The sampling frequency and duration of microtremor data collection used by other researchers are summarized in Table 2.2. The sampling frequency used ranged from 50 to 200 Hz and the sampling duration ranged from 2.5 to 180 minutes. In the analysis, the microtremor data are divided into different time windows in order to examine the temporal and spatial stationarity. The length of the windows was different for different researchers. If there is no temporal or spatial variation then the microtremor data will give the same autocorrelation coefficients for each window. No recommendation on sampling frequency and duration can be found in the microtremor survey literature.

Table 2.2 Sampling frequency for microtremor survey

References	Sampling frequency (Hz)	Duration (min.)	No. of windows
Saccorotti et al. (1999)	200	2.5	11 (8192 and 4096 points window)
Kudo et al. (1976)	Not Available	30 for < 100 m array 45 to 60 for >100 m array	20 to 40
Bettig et al. (2001)	Not Available	180	100 (8192 points window)
Estrella and González. (2003)	100	40	6
Matsuoka et al. (2003)	50	50	Not Available

2.9 Inversion of the dispersion curve

The dispersion curve obtained either from CSW method or microtremor survey method needs to be inverted to obtain the shear wave velocity profile of the site. The shear wave profile is the key parameter to geotechnical engineers in evaluating

dynamic characteristics of soils (Tokimatsu, 1997). The shear wave velocity profile can be evaluated from the dispersion curve using three principal inversion methods: factored wavelength method (also called wavelength-depth method), linear method and finite element method. Finite element method is beyond the scope of this study and will not be discussed further. In this study, wavelength-depth method and linear inversion method will be used to determine shear wave velocity profile of soil and are reviewed in the following sections.

2.9.1 Wavelength-Depth Method

The wavelength-depth method is the simplest method of inversion. The phase velocity is used to determine the shear wave velocity profile and is assumed to vary with depth z where z is a factor of wavelength λ . Assuming a value of λ/z , wavelength in the dispersion curve is converted to depth in the shear wave velocity profile.

A value of $\lambda/z = 2$ was first used by Jones (1958). Abbiss (1981) used $\lambda/z = 2$, as he noted that surface wave amplitude decays with depth and its velocity is characteristic of a depth of approximately one half wavelengths. However, he did not provide any theoretical basis for $\lambda/z = 2$. To distinguish separate layers of macadam, sand and gravel and underlying clay, Heukelom and Foster (1962) used $\lambda/z = 2$. Maxwell and Fry (1967) and Ballard and Mclean (1975) used $\lambda/z = 2$ to determine shear wave velocity profile by surface wave method.

Matthews and Clayton (1993) successfully applied the wavelength-depth method with $\lambda/z = 3$ to compressible soils in UK. Asten and Henstridge (1984) found that phase velocity is strongly affected by V_S at a depth of one-third of its wavelength. Butcher and Tam (1994) explained the basis of wavelength-depth method by considering the theory of Rayleigh wave propagation. The depth of Rayleigh wave penetration was taken to a depth of about one wavelength. This assumption gives rise to a triangular Rayleigh wave amplitude distribution over an effective depth of one wavelength. The centroid of the triangle is at one-third depth of the wavelength. Therefore, $\lambda/z = 3$ is more reasonable to be used in the wavelength-depth method. However, the shape of the displacement amplitude of the vertical component of Rayleigh wave with depth is not perfectly triangular. Moreover, the triangular shape assumed is only valid for single homogenous layer.

The variations of the amplitude of particle motions for different Poisson's ratio are shown in Figure 2.13. Poisson's ratio can be taken to imply a different soil type. It was found that for a single homogeneous layer the curves are very similar for Poisson's ratio ranging from 0.25 to 0.50. That means the centroid of the curve will always be at about one-third depth of the wavelength according to Butcher and Tam (1994). Therefore, λ/z will always be 3 regardless of soil type or ground condition which does not agree with observations.

Figure 2.13 Variation of the vertical and horizontal motion of Rayleigh wave at different Poisson's ratio.

Sezawa and Kanai (1935) suggested $\lambda/z = 4.54$ when they investigated the equation for the propagation of Rayleigh waves through a single layer overlaid on a more rigid substratum. Many researchers suggested upper value of $\lambda/z = 4$ and lower value of $\lambda/z = 2$ and used average value of $\lambda/z = 3$ as recommended by Gazetas (1982) (Matthews and Clayton, 1993; Matthews et al., 1996, 2000; Moxhay et al., 2001; Menzies, 2001).

The problems of the wavelength-depth method are its arbitrary nature, its inability to address modal propagation of Rayleigh waves, and its uncertainty in the assignment of shear modulus when used in vertically heterogeneous grounds (Ballard and McLean, 1975). By comparing the shear modulus profile obtained from the wavelength-depth method with the shear modulus profile obtained from the FE method, Clayton et al. (1995) found that the wavelength-depth method underestimated the shear modulus when stiffness of the soil changes greatly with depth.

Clayton et al. (1995) and Lo Presti et al. (2004) have pointed out that simple inversion methods may yield erroneous interpretation of the dispersion curve leading to a different stiffness-depth profile. However simple inversion methods such as the wavelength-depth method is still a useful tool to estimate the initial shear wave velocity profile in forward modeling (Addo and Robertson 1992, Ganji et al. 1998, Luke and Calderón-Macías 2007). The basis for the wavelength-depth method was explained by considering the theory of Rayleigh wave propagation (Addo and Robertson 1992, Butcher and Tam 1994). The dispersive nature of Rayleigh waves propagating in a layered earth model is shown in Figure 2.14. The particle motion of the Rayleigh wave is confined within a depth of about one wavelength. At high frequency, the particle motion occurs almost entirely in Layer 1 and the material properties of Layer 1 controls the Rayleigh wave velocity. At low frequency, the particle motion extends to Layer 3 and the velocity of this Rayleigh wave is governed by the combined material properties of all the three layers.

The wavelength-depth method has been investigated analytically by Gazetas (1982), Vrettos (1990) and Vrettos and Prange (1990). Gazetas (1982) investigated the variation of λ/z assuming that the shear modulus G increases with depth z according to the following equation:

$$G(z) = G_0(1 + \mu z)^m \quad (2.20)$$

where G_0 is the shear modulus at the surface, m is type of heterogeneity and μ is rate of heterogeneity.

Figure 2.14 Geometric dispersion of Rayleigh wave (from Rix, 2005)

Figure 2.15 Variation of λ/z with frequency for four heterogeneous deposits: $m = 1$, $2/3$, $1/2$ and $1/4$ (modified from Gazetas, 1982)

Figure 2.15 shows the variation of λ/z with frequency for four heterogeneous deposits ($m = 1$, $2/3$, $1/2$ and $1/4$) having $bH = 3$ where b is the rate of homogeneity of the deposit with linear variation in velocity and H is the thickness of the soil deposit. The approximate average value for λ/z is 4. Gazetas (1982) concluded that $\lambda/z = 4$ for a heterogeneous site and $\lambda/z = 2$ for a homogeneous site. As an average, a value of $\lambda/z = 3$ can be used.

Vrettos (1990) and Vrettos and Prange (1990) considered the following variation of shear modulus with depth for a homogeneous site:

$$G(z) = G_0 + (G_\infty - G_0)[1 - e^{-az}] \quad (2.21)$$

$$G(z) = G_0 + G_\infty H_0 [1 - e^{-az}] \quad (2.22)$$

where G_∞ is the shear modulus at infinite depth, a is a constant and $H_0 = (1 - G_0/G_\infty)$ is the inhomogeneity ratio. They found that $\lambda/z = 3.33$ more appropriate. Vrettos (1990) showed that λ/z varies with Poisson's ratio ν and dimensionless frequency Ω . The dimensionless frequency Ω is defined as follows:

$$\Omega = \frac{2\pi f}{a \sqrt{\frac{G_0}{\rho}}} \quad (2.23)$$

The dimensionless frequencies Ω can also be rewritten as:

$$\Omega = \frac{2\pi}{a\lambda^*} \quad (2.24)$$

The range values λ/z values for Poisson's ratio ν from 0.25 to 0.5 and inhomogeneity ratio H_0 from 0.90 to 0.98 for various dimensionless frequencies Ω and for the fundamental wave mode is reproduced here as Figure 2.16. Both Figure 2.15 and Figure 2.16 show that the range of λ/z is from 2 to 8 depending on degree of heterogeneity of the deposit, Poisson's ratio, frequency and wavelength. However most site stiffness profiles cannot be approximated using either Equation 2.20 or Equation 2.21, and therefore the value of λ/z will be difficult to estimate.

It is found from the above discussion that λ/z used in the literature is arbitrary. Use of an arbitrary value for λ/z sometimes leads to inaccurate shear wave velocity profile.

Nevertheless, wavelength-depth factor method is very simple and easy to use. If the wavelength-depth method can be improved such that λ/z is estimated more accurately, it can be provide practitioners an expedient tool to obtain a more reliable shear velocity profile which can be used as the initial model for more accurate inversion schemes.

Figure 2.16 Range of λ/z for $0.25 \leq v \leq 0.90$ and $0.25 \leq H_0 \leq 0.98$ and for fundamental wave mode (modified from Vrettos, 1990)

2.9.2 Linear Method

The assumption of the linear inversion method is that dispersion characteristics of a surface wave depends on P-wave velocity, V_P , S-wave velocity, V_S , bulk density, ρ and layer thickness, h . Tokimatsu et al. (1991) showed that change in V_P and ρ on the dispersion curve is negligible and therefore these two parameters are normally assumed as known during linear inversion (Orozco, 2003). Nazarian (1984) numerically showed the effects of ρ and v are less than 5%. The strongest dependence of dispersion characteristics of a surface wave is on V_S (Ewing et al., 1957; Fowler, 1990; Yuan and Nazarian, 1993; Xia et al., 1999).

Inversion of the dispersion curve can be performed in two ways: forward modeling and backward modeling. In forward modeling, Rayleigh wave velocity is determined from known values of V_P , V_S and ρ whereas in backward modeling or in inverse process, the parameter V_P , V_S and ρ is determined from the known dispersion curve. In the inversion process, different model parameters are iteratively adjusted until a best match between theoretical and experimental dispersion curves (Parker, 1994). Searching of the model parameters can be done in two ways: local search procedures (Tarantola, 1987; Menke, 1989) and global search procedures (Tarantola, 1987; Sen and Stoffa, 1995). In local search procedures, the error between theoretical and experimental dispersion curves is minimized. In global search procedures, the entire solution spaces are searched to find the global minimum by successive forward simulations. The global search procedure is advantageous because problem can be solved without any type of linearization of the forward algorithm (Orozco, 2003). However, it is time consuming and needs lots of calculation. Since inversion is an iterative process, the solution of the dispersion curve inversion is generally not unique (Knopoff, 1961; Tatham, 1975). The best inversion solution depends on the ability to experimentally measure the field variables and the ability to solve the corresponding inverse problem (Lai and Rix, 1998).

Linear inversion method based on wave propagation theory cannot be solved directly (Orozco, 2003). Different methods have been developed to find the theoretical dispersion curve. The methods commonly used in the inversion process are: (a) the transfer matrix method (Thomson, 1950; Haskell, 1953; Harkrider, 1964), (b) the stiffness matrix method (Kausel and Roësset, 1981; Wolf, 1985) (c) linearized stiffness matrix method (Lysmer and Waas, 1972; Waas, 1972), and (d) the method of reflection and transmission coefficients (Kennett, 1983). All the methods assume that soils are homogeneous layered material and there is no variation laterally. Some of the inversion methods are reviewed below.

Yuan and Nazarian (1993) developed an automated algorithm for determining theoretical dispersion curve based on transfer matrix method. The difference between the theoretical and experimental dispersion curve is expressed as an RMS value given by the following equation:

$$RMS = \sqrt{\frac{1}{N} \sum_{i=1}^N \Delta c_i} \quad (2.25)$$

The output parameters of the inversion are V_S and h . Yuan and Nazarian (1993) used data from SASW method and assumed that the dispersion curve was from fundamental mode Rayleigh wave. The soil profile was divided into many layers to find the layer thickness, and many iterations are needed to arrive at a good match of the theoretical and experimental dispersion curves.

Horike (1985) used an inversion method developed by Wiggins (1972) and Jackson (1972) to develop V_S structure from long period (0.5 Hz to 3.0 Hz) microtremor data. They solved the following eigenvalue problem:

$$\mathbf{Y} = \mathbf{A}\mathbf{X} \quad (2.26)$$

where $\mathbf{Y}$ is the differences between measured and initial phase velocities, $\mathbf{X}$ is the solution to be sought and $\mathbf{A}$ is the coefficient matrix derived from partial derivatives of phase velocities with respect to V_P , V_S , and ρ . In the method, a few initial models are assumed based on the geological and geophysical information and then all these initial models are inverted. The final inverted model selected from all those inverted models is based on the minimal discrepancy between initial information of the site and inverted V_S profile. However, this inversion method requires initial geological and geophysical information which are not always available. This method cannot detect extremely-low-velocity layer within the soil profile.

Tokimatsu (1997) presented an inversion method that is capable of accounting for higher modes for both active and passive surface wave data. The eigenvalue problem given by Equation 2.26 is solved using the method of Dorman and Ewing (1962) and Wiggins (1972). In the search process, Tokimatsu (1997) used the following least-squares equation:

$$RMS = \sum_{i=1}^N \Delta c_i^2 \quad (2.27)$$

To reduce the ambiguity of the inversion, Tokimatsu (1997) suggested that both vertical and horizontal motion dispersion curve should be used. Fundamental mode is dominant in normally dispersive soil profile (stiffness increases with depth) and inversely dispersive soil profile has multiple mode of Rayleigh wave propagation. His method can invert both normally and inversely dispersive soil profiles and was verified by comparing with data from down-hole method which showed good agreement. However, the numerical calculation of Tokimatsu (1997) inversion process is too time consuming.

Ganji et al. (1998) developed an automated inversion procedure for the spectral analysis of surface wave (SASW) which can be used for all types of surface waves (Rayleigh and Love) and multiple modes. In the algorithm they used a two-step calculation. Firstly, the theoretical dispersion curve was calculated based on the properties of the assumed profile using Kausel's algorithm. Secondly, the error between experimental and theoretical dispersion curve is minimised by using both linear minimization technique, developed by Wiggins (1972) and nonlinear minimization technique, developed by David-Fletcher-Powell method (McCormick, 1983). In order to verify the inversion procedure, Ganji et al. (1998) used six theoretical cases, both regular (normally dispersive) and irregular (inversely dispersive) soil profiles and showed that the inverted V_S profile matched well with the theoretical V_S profile. However, all the six cases were created using finite element methods.

Lai and Rix (1998) developed a linear inversion program using Matlab to invert the fundamental mode Rayleigh wave dispersion curve for an assumed layered earth model. The assumed initial values for inversion are V_S , ρ , h and ν . The inversion algorithm is based on the algorithm presented by Constable et al. (1987). The strategy of the algorithm is to find the smoothest shear wave velocity profile subject to the constraint of a specified misfit between experimental and theoretical dispersion curves and associated uncertainties. Convergence is achieved if the root-mean-square (RMS) error is 1.0 or less (i.e., the specified misfit) or there is a negligible change in the shear wave velocity profile from one iteration to the next. The RMS error is as follows:

$$RMS = \frac{norm[W\Delta c]}{\sqrt{N}} \quad (2.28)$$

where W is a diagonal matrix with elements equal to the inverse of the standard deviations of the experimental dispersion curve. The output parameters are vector of final shear wave velocities (empty if no convergence), vector of RMS errors for each iteration, matrix of theoretical phase velocities, vector of standard deviations of final shear wave velocities.

Xia et al. (1999) used an inversion process for fundamental mode Rayleigh-wave dispersion curve of a layered-earth model. Xia et al. (1999) solved the eigenvalue problem given in Equation 2.26 using Schwab and Knopoff (1972). The RMS error criterion was not given. Xia et al. (1999) investigated the sensitivity of input parameters: V_P , V_S , ρ and h . They found that V_S is the most dominant parameter in the high-frequency range (> 5 Hz) followed by layer thickness. However, Xia et al. (1999) used only V_S as the unknown parameter in the inversion process.

In summary, there are a number of inversion methods available. However, as in all inverse problems, uniqueness of solution is not guaranteed. Lai and Rix (1998) method of inversion needs four physical properties of the soil layer (h , v , ρ and initial V_S) as input parameters. In this study, the linear inversion method developed by Lai and Rix (1998) was adopted due to its simplicity for use.

2.10 Seismic Waves in Laboratory Tests

Seismic wave velocity can provide information on small-strain stiffness which is an important input parameter in soil dynamic analyses (Pezeshk and Zarrabi, 2005). Seismic wave velocities of soil can be measured in the laboratory (Nakagawa et al., 1996). Even though laboratory tests may not give more accurate wave propagation velocity of soil compared to field geophysical methods due to sampling disturbance and small sample size, laboratory tests can provide a check of the field test results. There is a need to understand the cause of the discrepancy between shear wave velocities determined from field geophysical methods and those determined in the laboratory. It is also well known that the stiffness of soil is strain dependent and

laboratory tests can provide the stiffness of soil at various strains whereas in the field, the stiffness-strain relationship cannot be measured easily. In the following sections, previous research on determination of small-strain stiffness and the stiffness-strain relationship of soils in the laboratory will be reviewed.

2.10.1 Small-Strain Stiffness

Cyclic loading (Seed and Chan 1966; Matsui et al. 1980; Yeo, 2005), resonant column (Hardin and Richart, 1963; Schneider et al., 1999) and pulse transmission (Stephenson, 1978; Dyvik and Madshus, 1985; Nakagawa et al., 1996; Arulnathan et al., 1998; Inci et al., 2003; Leong et al., 2004; Lee and Santamarina, 2005; Yeo, 2005) are three methods that have been used in the laboratory for the determination of small-strain elastic constants of soils. Cyclic loading and resonant column methods are the most commonly used methods (Nakagawa et al., 1996). However, the small-strain shear moduli determined by field geophysical measurements are higher than that obtained from cyclic loading tests or resonant column tests, even at very small strain levels (Anderson and Woods, 1976). Pulse transmission method is useful for the direct observation of propagating elastic waves at low-strain level (Nakagawa et al., 1996).

Pulse transmission test is becoming popular in the laboratory due to its simplicity and cost effectiveness (Jivanuwong, 1998; Leong et al., 2004). The bender element test and ultrasonic velocity test. The principle of the pulse transmission test is simple: travel time for the wave Δt to pass through the sample of the length L is measured and the wave velocity is computed as:

$$V = \frac{L}{\Delta t} \quad (2.29)$$

In bender element test, two piezoelectric ceramic elements (bender element) are used as transmitter and receiver. It has the advantage of direct contact of the bender element with the soil specimen to create a stronger signal. However, it is very vulnerable to damage. In bender element test, the S-wave arrival is sometimes affected by near field effects due to P-wave. In ultrasonic velocity test, the

piezoelectric elements are not in direct contact with the soil specimen but are bonded inside a pair of platens and therefore it is less vulnerable to damage. However, there is a gap between piezoelectric element and soil specimen due to the platens. In the ultrasonic velocity test, signal can be transmitted in three different: (a) direct transmission (b) Semi-direct transmission and (c) indirect or surface transmission as shown in Figure 2.17. The direct transmission arrangement is the most satisfactory since the longitudinal pulses leaving the transmitter are propagated mainly in the direction normal to the receiver hence the receiver will receive maximum energy from the transmitter. Furthermore, in case of semi-direct transmission and surface transmission, the length of the path is not well defined and it is assumed that the travel distance of the transmitter signal should be taken as centre to centre of the transducers.

Figure 2.17 Different transmission mode in ultrasonic velocity test

In this study, the ultrasonic velocity test system developed by Geotechnical Consulting and Testing System (GCTS, 2001) will be used to determine the dynamic properties of soil. A schematic of the ultrasonic velocity test is shown in Figure 2.18. The velocity, either P-wave or S-wave, of the soil specimen is given by the length of the sample, L , divided by the arrival time of the signal from transmitter to receiver, Δt , given by Equation 2.29.

Figure 2.18 Velocity determination in ultrasonic test

The very small-strain shear modulus G_{max} can be expressed as:

$$G_{max} = \rho V_s^2 \quad (2.30)$$

where ρ is the bulk density.

The ultrasonic velocity test system (GCTS, 2001) which was used in this research for laboratory experiment could measure travel time to a resolution 0.1 μs . For a specimen length of 140 mm and a wave velocity of 1500 m/s (close to V_P of water), the error in velocity measurement is about ± 2 m/s. However, the greater error in the ultrasonic velocity test is the determination of first arrival time (Lee and Santamarina, 2005). There are different methods of determining first arrival time for wave velocity determination. It can be determined from time series data or from cross correlations of the time series data by separating them into different windows. First arrival time selection from time series data can be effected by near field effect (Brignoli, 1996). The near field effect will create an extra bump just before the arrival of original wave velocity signal to the receiver as shown by segment ABC (Figure 2.19). Therefore, the

selection of first arrival from time series data has different options (Figure 2.19): (A) first deflection (B) first reversal (C) zero after first reversal, and (D) major first peak (Lee and Santamarina, 2005).

Figure 2.19 First arrival time detection from time series data
(modified from Lee and Santamarina, 2005)

Lee and Santamarina, (2005) mentioned that the determination of the travel time from A, B, C and D is controversial, therefore, they suggested that selection of criteria depends on installation, application and input signal. However, when there is near-field effect, the use of point A is not suggested since that point might represent the near-field component signal (Mancuso and Vinale, 1988). The measurement of travel time by cross-correlation method is determined by plotting cross correlation of the receiver signal by separating 'same nature' data (Lee and Santamarina, 2005) into different windows. The first arrival time is given by the major first peak point. Unlike time series method, cross correlation method has only one point to select. Viggiani and Atkinson (1995) found the cross-correlation method to be the most accurate method to determine the first arrival time. However, ambiguity remains here with the separation of data into different windows as there is a possibility of mixing up or overlapping same 'nature data'. Gajo et al. (1997) found that cross-correlation method underestimates the arrival time. Jovicic et al. (1996) recommended time series method to find first arrival time when input signal is properly specified. In this study, the time series method was used to determine first arrival time. It was observed from time series data that there is no near field effect. Therefore, the first deflecting point (A) was used consistently in first arrival time determination.

2.11 Stiffness-Strain Relationship of Soil

Strain mobilization is dependent on the geotechnical structures as shown in Figure 2.20. Therefore, it is important for the geotechnical engineer to understand the behaviour of soil stiffness with strain. A number of researches have been performed to investigate the stress-strain relationship of soils (Huoo-Ni, 1987; Hardin and Black, 1968; Borden et al., 1996; Burland, 1989; Hoyos and Macari, 1999; Schneider et al., 1999; Atkinson, 2000; Baranski, 2003).

Figure 2.20 Typical stiffness-strain behaviour of soils for laboratory tests and structures (from Baranski, 2003)

Stiffness of soil, G , at a strain below threshold strain, γ_{th} , is called initial shear modulus or very small strain shear modulus, G_{max} . Soil exhibits modulus degradation when it experiences strains higher than γ_{th} and the stiffness-strain relationships of soil are non-linear (Huoo-Ni, 1987; Hardin and Black, 1968; Borden et al., 1996; Burland, 1989; Hoyos and Macari, 1999; Schneider et al., 1999; Atkinson, 2000; Baranski, 2003). G_{max} depends on effective confining pressure σ_3 , void ratio e , geologic age, cementation, overconsolidation ratio (OCR), plasticity index (PI), cyclic strain and number of loading cycles as illustrated in Table 2.3. The relationship of G_{max} with shear wave velocity is given by Equation 2.30. Hardin and Black (1968) showed

from laboratory experiments that shear wave velocity is a function of test conditions: e.g., stress condition, OCR, e , and strain of the measurement.

Table 2.3 Factors affect $G_{\max}$ (from Kramer, 1996)

Increasing Factor	$G_{\max}$
Effective confining pressure, σ'_m	Increases with σ'_m
Void ratio, e	Decreases with e
Geologic age, t_g	Increases with t_g
Cementation, c	Increases with c
Overconsolidation ratio, OCR	Increases with OCR
Plasticity index, PI	Increases with PI if OCR > 1; stays about constant if OCR = 1
Strain rate, $\dot{\gamma}$	No effect for non-plastic soils; increases with $\dot{\gamma}$ for plastic soils (up to ~10% increase per log cycle increase in $\dot{\gamma}$)
Number of loading cycles, N	Decreases after N cycles of large γ_c , but recovers later with time in clays; increases with N for sand

Source: Modified from Dobry and Vucetic (1987).

Borden et al. (1996) found that difference in effective confining pressure σ_m from 25 to 100 kPa would increase threshold strains about 2.5 times. Schneider et al. (1999) found the effect of effective confining pressure on threshold strain could be even ten times. That means, increasing in σ_m will increase G (Dorby and Vucetic, 1987). Void ratio, e , will affect G , that is, G will increase with decrease in e and vice versa (Dorby and Vucetic, 1987). The soil samples used in this study were of the same geologic age and did not have any cementation. Therefore, effects of geologic age and cementation are not considered in this study. Since, all the soil samples used in this study were normally consolidated; therefore, OCR effect can be neglected. Furthermore, Schneider et al. (1999) found that OCR did not have noticeable effect on G . Schneider et al. (1999) showed that increase in PI will increase G . Only quazi-static triaxial compression tests with local strain measurement were performed in this study. Therefore, effects of strain rate and number of loading cycles are not investigated. In this study, only effects of σ_3 , e and PI on G will be investigated.

2.12 Summary

This chapter discusses the properties of seismic waves with some basic theories of seismic waves and their propagation modes. Different geophysical methods have also been presented in this chapter followed by the literature review of surface wave methods. This chapter discusses previous researches, theoretical background as well as the analytical procedure of some surface wave methods and microtremor survey method. Different parameter that can influence the results of CSW and microtremor survey method were also discussed in this chapter. Extensive literature review has been presented in this chapter about the inversion of dispersion curves obtained by geophysical methods. This chapter also discusses seismic wave in the laboratory tests. Determination of small strain stiffness is one of the objectives in this research. Therefore this chapter also discusses the effects of some parameters on small-strain stiffness with experimental results.

CHAPTER 3

CSW AND MICROTREMOR SURVEYS

3.1 Introduction

Field experiments, both CSW and microtremor surveys, were conducted at seven sites of different geological formations in Singapore. In this chapter, the geology of Singapore is briefly described, followed by the test procedures of the CSW and microtremor survey methods and the description of the seven test sites.

3.2 Geology of Singapore

The Republic of Singapore, consisting of one large island, area of 540 km², and several small islands, area of 44 km², lies between latitude 1° 09' N to 1° 28' N and longitude 103° 38' E to 104° 06'. About two thirds of the land area consists of residual soils, the result of weathering over the years. Studies had been done to classify and understand the geological formation of the Singapore (Leow, 1962; Burton, 1964; Hutchison, 1964; Wong, 1960; Chin, 1965; Tai, 1972; Seet, 1974; Lim, 1974; PWD, 1976). Singapore geology can be divided into nine geological formations (PWD, 1976): Sajahat Formation, Gombak Norite, Paleozoic Volcanic, Bukit Timah Granite, Jurong Formation, Old Alluvium, Huat Choe Formation, Tekong Formation and Kallang Formation. The major geological formations are Bukit Timah Granite, Jurong Formation, Old Alluvium and Kallang Formation.

In this research, site characterization by CSW and microtremor surveys were carried out at seven sites in four geological formations, i.e., Jurong Formation (JF), Old Alluvium (OA), Bukit Timah Granite (BKG) and Kallang Formation (KF). The locations of the sites are given in Figure 3.1.

Figure 3.1 Location of experimental sites (modified from PWD, 1976)

3.2.1 Jurong Formation

Jurong Formation located in the southern and western part of Singapore consists of sedimentary rocks like conglomerate, limestone, mudstone, sandstone, siltstone which have been subjected to various degree of weathering. Based on the grain size, composition and hardness, six sedimentary facies are identified: Queenstown facies, Jong facies, Ayer Chawan facies, Rimau facies, St John facies and Tengah facies. None of the facies is continuous, tectonic activity has been responsible for this dynamic metamorphism (PWD, 1976)

Site 1 and Site 2 at the NTU campus and Site 3 along the pioneer road are in the Jurong Formation. By examining the borehole logs, these sites can be said to be of the Queenstown facies.

3.2.2 Old Alluvium

Old Alluvium is found in the north and north-east of the Kallang River Basin and in the north-western part of the Singapore main island in the Buloh Besar area. It consisted mostly of clayey coarse angular sand with stringers of subrounded pebbles up to 4.0 cm in diameter and the beds are often cross-bedded with coset thickness up to 1.5 m (PWD, 1976). The pebbles within the Old Alluvium are dominantly quartz and no granite pebbles have yet to be found. The formation of Old Alluvium is usually uncemented but quite dense and with low permeability, 10^{-8} to 10^{-10} m/s (Pfeiffer, 1972). Site 4 is an Old Alluvium site.

3.2.3 Bukit Timah Granite

Bukit Timah Granite covers one third of the area of Singapore. The formation consisting of igneous granite is found in two different areas of Singapore, one is in the central part and the other is in the north eastern part, on Pulau Ubin island. The upper zone of the Bukit Timah Granite has been subjected to severe weathering and became residual soil. The depth of weathering ranges from 0 to 25 m. Usually three distinct layers can be observed (Thum and Yong, 1987), the top layer is about 6 m thick consisting of sandy silty clay and the middle layer is about 10 to 25 m thick consisting of sandy clayey silt and the bottom layer consists of sandy silt or silty sand. Sites 5 and 6 are in the Bukit Timah Granite.

3.2.4 Kallang Formation

Kallang Formation is mostly found around Sungei Kallang, Sungei Jurong, Sungei Serangoon and Sungei Seletar. The deposits are generally low lying, and are seldom recognized more than 4 m above sea-level except in more inland areas (PWD, 1976). The Kallang Formation consists of sedimentary deposits. Pockets of peat, shells and lithic fragments can also be found in this formation. Based on the mode of sedimentation, five members are recognized in the Kallang Formation: marine member, alluvial member, littoral member, and transitional member and reef member (PWD 1976).

The marine clays of the Kallang Formation usually occurred in two layers, the upper marine clay and the lower marine clay. These two layers of marine clay are separated by a weathered crust of the lower marine clay layer. The separation layer between the upper and lower marine clays represents a significant boundary and can be 1 to 2.5 m thick of stiff reddish brown or grey silty clay (PWD, 1976). The lower marine clay was deposited unconformably and was in turn overlain by unconsolidated peat, beach and young fluvial deposits. Although deposited at different times, the upper and lower marine clays share similarities in appearance and touch. Both members are essentially a kaolinite-rich clay deposit.

There is very little information on the dynamic soil properties of the geological formations in Singapore. Veijayaratnam et al. (1993) collated the dynamic soil properties for different formations of Singapore soils from tests performed during the site investigation for the construction of the mass rapid transit system in Singapore. The dynamic soil properties are summarized in Table 3.1.

Table 3.1 Summary of dynamic soil properties (Veijayaratnam et al., 1993)

Soil Type	C_u (kN/m ²)	ρ (Mg/m ³)	V_p (m/s)	V_s (m/s)	G_{max} (kPa)	ν	E_0 (kPa)
JF (Queenstown facies)	162	2.10	1750	210	93	0.49	276
OA	148	2.00	2400	500	500	0.48	1477
BTG	38	1.80	1650	200	72	0.49	215
KF (Peaty clay)	20	1.42	1350	60	5	0.50	15
KF (Marine clay)	13	1.51	1450	70	7	0.50	22

3.3 Field Measurements

In this research, seismic surveys were conducted at sites where borehole information was available. Two types of seismic surveys were conducted: continuous surface wave test and microtremor survey. Both of these surveys were conducted using the Continuous Surface Wave System (CSWS) developed by GDS (1998). The characteristics of the CSWS are described in the following sections.

3.3.1 Continuous Surface Wave System

The CSWS developed by GDS (GDS Instruments Ltd., 1998) shown in Figure 3.2 was used for this research. CSWS is a portable system that can be easily transported to the site. The test can be set up by two persons. The CSWS consists of (a) control unit (b) personal computer (c) drive unit (d) portable power generator (e) vibrator, and (f) geophones.

The control unit synchronizes the data acquisition of the geophones and controls the drive unit. The drive unit drives the vibrator according to signals from the control unit. The CSWS has a vibrator of 64 kg, which can produce surface waves with frequencies ranging from 5 Hz to 600 Hz by moving vertically up and down during test. The vibration frequency of the vibrator is controlled by the control unit. The vibrator had its own cooling fan, which prevents the vibrator from overheating.

Vertical geophones in CSWS detect the surface waves. Six 4.5 Hz geophones were used in the test. The consistencies of the geophones were checked using huddle test (Lennartz -electronic, 2004). In the huddle test, all the geophones are placed in close proximity. All the geophone signals should be almost similar if they are in good working condition. Each of the geophones has a spike for anchoring into the ground so that it can be fixed during test. The whole CSWS is powered by a portable power generator that supplies 110 Vac.

Figure 3.2 Schematic of CSWS from GDS instruments Ltd. (1998)

The CSWS comes with a computer program that controls the test. The computer program requires the input of some parameters before start of field test. The parameters required are the position of the geophones with respect to the vibrator, Poisson's ratios and unit weight of soils and the sampling frequency of the data. In the field, the positions of the geophones and the vibrator were marked. The top soil was removed to place the vibrator firmly and to plant the geophones properly in place. The spacing of the geophones was 1.0 m. The level of the vibrator was confirmed using a spirit level. The resolution of the V_s profile is related to geophone spacing. Geophone spacing should be less than one tenth of the depth of investigation for good resolution (Park, 1995). The drive unit, the control unit, the laptop PC and the vibrator were then connected. In order to reduce ambient noise, the power generator was placed as far as possible from the test set-up. The computer program was then started and all the necessary parameters for the test were input. The vibrator created vertically polarized disturbance to the ground that generated surface waves. These surface waves were detected by six geophones. The frequency of the vibrator can be varied from 5 Hz to 600 Hz with an increment of 0.25 Hz to 5 Hz during the test. During the test, the dispersion curve was inverted using the wavelength-depth method which can be observed on the PC display. When there is a need, the test can be repeated for smaller frequency range and at a smaller frequency increment to

obtain a more detailed dispersion curve. Finally, the collected data can be extracted from the data acquisition system for further analysis.

3.3.2 Microtremor Survey Method

The main different between CSW and microtremor survey methods is the energy source. In microtremor survey, passive waves due to ambient noise are monitored. A circular array is used to eliminate directionality of the surface waves (Okada, 2003). Therefore, most of the CSWS can be used for microtremor survey.

The CSWS computer program can be made to collect data in a continuous mode. For the microtremor survey method, the CSWS vibrator is not needed. However, microtremor survey method used much wider spacing of the geophones compared to CSW method. To achieve an array of 40 m diameter, additional 20 m long extension cables were added to each of the geophones of the existing CSWS. The test procedure of microtremor survey is similar to the CSW method except for the orientation of the geophones and duration of the data collection. In microtremor surveys, three geophones are placed at the vertices of an equilateral triangle and the fourth geophone is placed at the centre (Figure 3.3). The radii of the array using the CSWS can be varied from 1 m to 20 m and data are collected for 10 minutes with a sampling frequency of 204.8 Hz.

Figure 3.3 Geophone arrangement in microtremor survey method

3.4 Description of Test Sites

The site conditions and the test layout for CSW and microtremor surveys at the seven test sites are presented in this section.

Figure 3.4 Location of Site 1 and Site 2

3.4.1 Site 1: NTU Site 1

NTU Site 1 is at Nanyang Technological University (NTU) campus, located behind the School of Civil and Environmental Engineering as shown in Figure 3.4. The site is a narrow strip located on the crest of the slope. Therefore, CSW investigation was

conducted in two opposite directions (A and B) with the vibrator at the centre. Past site investigation records consisting of borehole logs and SPT are available which shows the geological formation of this site is Jurong Formation. CSW and microtremor surveys were conducted at the site as shown in Figure 3.5. The microtremor survey consisted of three geophones in a triangular pattern and one central geophone (Figure 3.5). The array size was 2 m in diameter.

Figure 3.5 Field experimental arrangement for Site 1

3.4.2 Site 2: NTU Site 2

NTU Site 2 is also at NTU campus, near the Jalan Bahar exit as shown in the Figure 3.4. This site is at the corner of an open land with very gentle slope. Past site investigation records from research project (Leong et al., 1996) are also available for this site. Eight Cone Penetration Test (CPT) and four boreholes at various locations of the site are available. The geology of this site is Jurong Formation.

CSW survey was conducted in a straight line of 50 m length. The vibrator position was at 6 m, 18 m, 30 m and 42 m along the survey line; i.e. each position of the vibrator covers 12 m along the survey line (Figure 3.6). Microtremor surveys consisting of four geophones were conducted in five plots; each plot size was 10 m square. The array size was 10 m and 20 m in diameter. The arrangements of CSW and microtremor surveys are shown in Figure 3.6.

Figure 3.6 Field experimental arrangement for Site 2

3.4.3 Site 3: Pioneer Road

This site is at the corner of the Pioneer Road and Liu Fang Road as shown in Figure 3.1(a).

Figure 3.7 (a) Location of Site 3 (b) Field experimental arrangement for Site 3

Heavy vehicles pass by frequently at the site. The available borehole was 8 m from position of the vibrator of the CSW surveys. The borehole log of this site showed that the site consists of Jurong Formation.

The experiments were performed beside Pioneer Road at a distance of 2.5 m from the edge of the road. CSW investigation was conducted in two opposite directions (A and B) with the vibrator at the centre. The microtremor survey was conducted at the same location as the CSW survey. The array size of the microtremor survey was 8 m in diameter. The arrangements of CSW and microtremor surveys are shown in Figure 3.7(b).

3.4.4 Site 4: Buangkok Drive

This site is near a river, Sungei Serangoon, as shown in Figure 3.8(a). The available borehole was at 5 m from the vibrator position of the CSW surveys. The borehole log of this site shows that the site consists of a backfill of 11.4 m followed by Old Alluvium.

Figure 3.8 (a) Location of Site 4 (b) Field experimental arrangement for Site 4

CSW surveys were conducted in four orthogonal directions (A, B, C and D) with the vibrator at the centre. The microtremor survey was conducted at the same location with the central geophone at the CSW vibrator position. The array size of the microtremor survey was 10 m in diameter. The arrangements of CSW and microtremor surveys are shown in Figure 3.8(b).

3.4.5 Site 5: Sungei Changi

Site 5 is at the corner of the Paku Road and a river, Sungei Changi. It is an open space beside the Singapore Aviation Academy as shown in Figure 3.9(a). The available borehole was 8 m from the vibrator position of the CSW tests. The borehole log of this site shows that the site consists of a 5.0 m thick backfill followed by Bukit Timah Formation.

(a)

(b)

Figure 3.9 (a) Location of Site 5 (b) Field experimental arrangement for Site 5

CSW tests were conducted in four orthogonal directions (A, B, C and D). The microtremor survey was conducted with the central geophone at the CSW vibrator position. The array size of the microtremor survey was 10 m in diameter. The arrangements of CSW and microtremor surveys are shown in Figure 3.9(b).

3.4.6 Site 6: Orchard

Site 6 is beside a car park in the Orchard road area opposite Youth Park as shown in Figure 3.10(a). The available borehole was 7 m from the vibrator position of CSW surveys. The borehole log of this site shows that the site consists of Bukit Timah Granite.

Figure 3.10 (a) Location of Site 6 (b) Field experimental arrangement for Site 6

CSW investigation was conducted in four orthogonal directions (A, B, C and D) with the vibrator at centre. Microtremor survey was conducted with the central geophone at the

CSW vibrator position. The array size of the microtremor survey was 10 m in diameter. The arrangements of CSW and microtremor surveys are shown in Figure 3.10(b).

3.4.7 Site 7: Chinatown

Site 7 was located near the junction of Telok Ayer Street and Cross Street in downtown, Singapore, as shown in Figure 3.11(a). The available borehole was 5 m from the position of the vibrator of CSW surveys. The borehole log of this site shows that the site consists of Kallang Formation.

Figure 3.11 (a) Location of Site 7 (b) Field experimental arrangement for Site 7

CSW investigation was conducted in four orthogonal directions (A, B, C and D) with vibrator at the centre. Microtremor survey was conducted with the central geophone at the vibrator position of CSW surveys. The array size of the microtremor survey was 10 m in diameter. The arrangements of CSW and microtremor surveys are shown in Figure 3.11(b).

3.5 Summary

Singapore geology was briefly reviewed to provide a background to the geology of the seven sites. CSW and microtremor test set-ups were described. CSW and microtremor surveys were conducted at seven sites in Singapore with different geological conditions. The CSW and microtremor survey results are analyzed and discussed in Chapters 4 and 5, respectively.

CHAPTER 4

ANALYSIS OF CONTINUOUS SURFACE WAVE TEST RESULTS

4.1 Introduction

Site characterisation using seismic methods has become a common site investigation tool in many engineering projects (Addo and Robertson, 1992; Butcher and Powell, 1996; Tokimatsu, 1997; Luna and Jardi, 2000; Matthews et al., 2000; Foti, 2005). Methods employing surface waves have seen increased usage in the last two decades due to advancement in equipment, data acquisition and numerical tools to interpret the survey. Both active and passive source systems produce a dispersion curve which is then inverted to obtain a shear velocity-depth profile. A dispersion curve is a plot of wave velocity versus wavelength or frequency. Most of the recent research works are concentrated on the inversion of the dispersion curve (Ganji et al., 1998; Lai and Rix, 1998; Xia et al., 2000; Beaty et al., 2002; Lebedev et al., 2005; Storme et al., 2005). The inversion methods can be broadly divided into (1) Wavelength-depth method, and (2) Elastic models.

The wavelength-depth method is an empirical method and involves a direct scaling of the dispersion curve to convert phase velocity, V_ϕ , to shear wave velocity, V_s , and wavelength, λ , to an equivalent depth, z . Phase velocity is related to shear wave velocity through the Poisson's ratio ν . For Poisson's ratios ranging from 0.1 to 0.4, the ratio of phase velocity to shear wave velocity (V_ϕ/V_s) varies from 0.89 to 0.95 (Lay and Wallace, 1995). To convert wavelength to depth, it is assumed that the ratio of wavelength and depth (λ/z) is a constant.

In the elastic models, the ground is modeled as a layered earth model. Each soil layer has an assumed shear wave velocity, compression wave velocity (or Poisson's ratio), mass density and thickness. The inversion of the dispersion curve, similar to many inverse problems, is ill-posed and may have non-unique solution. In order to obtain a good solution, a high quality starting model is required (Orozco, 1996; Luke and Calderón-Macías, 2007). Addo and Robertson (1992), Ganji et al. (1998) and Luke and Calderón-Macías (2007) used the wavelength-depth method to obtain a starting

model for inversion. The intent of this chapter is to present an improved wavelength-depth method to provide a more reliable and quick estimate of the stiffness profile which can be used as the starting model for more accurate inversion schemes.

This chapter described the general procedure to calculate phase velocities from CSW data followed by the development of a modified wavelength-depth inversion procedure. This modified wavelength-depth inversion procedure will be evaluated using case histories.

4.2 General Procedure to Obtain the Dispersion Curve

The theory for the CSW method was described in Chapter 2. It is seen from the theory that the wavelength λ of the propagating waves can be estimated from the relationship $\lambda = 2\pi d/\phi$, where d/ϕ for each frequency can be directly calculated by plotting phase angles of geophones against distance of the geophones from the vibrator. A sample calculation of phase angles for each geophone by this method is tabulated in Table 4.1. The phase angles are plotted against distance of the geophones from the vibrator in Figure 4.1. The slopes, ϕ/d , (variation of phase angle of the geophones with spacing) were then obtained by least square method. It was assumed in the analysis of CSW that there was no near-field effect. All the data were collected at a known frequency which was confirmed by the PSD (Power Spectral Density) plot, where dominant frequency was always the same as the prescribed frequency during CSW test.

Table 4.1 Phase angles for different frequencies

Frequency (Hz)	Phase angle (degree)					
	GP-1	GP-2	GP-3	GP-4	GP-5	GP-6
10	33.29	33.66	36.03	38.4	40.77	43.14
12	287.41	290.89	294.37	297.85	301.33	304.81
13	33.16	37.32	44.48	45.64	49.8	53.96
14	-15.18	-10.36	-5.54	-0.72	4.1	11.92
15	4.56	10.05	15.54	23.03	26.52	32.01
16	26.683	32.973	39.263	45.55	51.843	58.13
Note: GP-1= Geophone 1, GP-2= Geophone 2, GP-3= Geophone 3, GP-4= Geophone 4, GP-5= Geophone 5 and GP-6= Geophone 6.

(a) 10 Hz

(b) 12 Hz

(c) 13 Hz

(d) 14 Hz

(e) 15 Hz

(f) 16 Hz

(g) 17 Hz

(h) 18 Hz

Figure 4.1 Slope angle of phase angles by least square method

The wavelength for each frequency is then calculated from $\lambda = 2\pi d/\phi$. Using this calculated wavelength, the corresponding phase velocity V_ϕ is calculated. The calculated V_ϕ are then plotted with wavelength to give the dispersion curve (Figure 4.2).

Figure 4.2 Dispersion curve by CSW method

4.3 Dispersion Curves at the Seven Test Sites

The descriptions of sites as well as experimental set-up for the seven sites are presented in Chapter 3 and will not be repeated here. The dispersion curves obtained by CSW method at the seven sites are shown in Figure 4.3 to Figure 4.9. In order to determine the lateral variation of the soil at the site, the CSW survey was performed along four orthogonal directions with the vibrator at the centre. The directions of the lines were chosen based on availability of space at the sites. Different alphabets have been used to distinguish the dispersion curves according to direction. Furthermore, the CSW surveys along different directions at the same site will also provide a consistency check of the data.

Figure 4.3 Dispersion curves from Site 1

Figure 4.4 Dispersion curve from Site 2

Figure 4.5 Dispersion curve from Site 3

Figure 4.6 Dispersion curve from Site 4

Figure 4.7 Dispersion curve from Site 5

Figure 4.8 Dispersion curve from Site 6

Figure 4.9 Dispersion curve from Site 7

It can be observed from the dispersion curves for each site that the results are consistent irrespective of the survey direction which implies that the stratification at the site is horizontal. It also shows that CSW surveys along different directions increase the number of data points along the dispersion curve hence a more detailed dispersion curve could be observed. The dispersion curves are used to determine the shear wave velocity profile of the sites by inversion. However, low phase velocity was observed at the surface of Site 1. The low phase velocities at short wavelengths were due to the loose materials in the backfill as shown in the borehole log of Figure A1.

4.4 Inversion of the Dispersion Curves

The data obtained by CSW or microtremor survey method give the variation of phase velocity of the ground with wavelength, known as dispersion curve. To obtain stiffness profile of the ground, the dispersion curve needs to be inverted to obtain the shear wave velocity profile of the site. As reviewed in Chapter 2, different methods can be used to invert the dispersion curve. In the following sections, wavelength-

depth method is re-examined to provide a better basis to select the wavelength-depth ratio, λ/z .

4.4.1 Development of a Basis for λ/z

To develop a basis for the λ/z , a review of the development of wavelength-depth method is necessary. Jones (1958) observed the propagation of Rayleigh waves for a road pavement and found that the dispersion curve showed an abrupt change in phase velocity when the second layer was encountered and that the ratio of the wavelength of the vibration and the thickness of the layer was 4.55. Powell and Butcher (1991), Butcher and McElmeel (1993) and Butcher and Tam (1994) found generally $\lambda/z = 2$ or 3 works well and that $\lambda/z = 2$ for clay soils and $\lambda/z = 3$ for granular soils and fills. Comparisons were made between the dispersion curves with borehole logs, seismic cone profiles or other in situ profiling tools. Abbiss (2001) measured the Rayleigh wave velocity profile for a landfill site and found that $\lambda/z = 2$ showed good agreement with the site profile.

For a normally dispersive soil deposit, the Rayleigh wave velocity increases with depth. Where changes of soil types are encountered, the variation of Rayleigh wave is discontinuous (Zhang and Lu, 2003). This fact can be exploited to determine the thickness of the individual soil layers encountered in the surface wave test. Following from the work of Gazetas (1982), Vrettos (1990) and Vrettos and Prange (1990), the relationship between λ/z with frequency and wavelength is investigated.

The dispersion curve is approximated by several best fit lines and the change in gradient of the best fit lines identifies a change in soil type as illustrated in Figure 4.. Subsequently, the dispersion curve is matched with the borehole log by applying a suitable λ/z to obtain the best agreement with the borehole information. The best matched λ/z ratios for Sites 1 to 7 are 3.5, 3.5, 3.5, 3.48, 3.64, 3.65 and 3.0, respectively. A summary of the wavelength-depth ratio λ/z and maximum penetration depth in terms of wavelength is shown in Table 4.2.

Figure 4.10 (a) Site 1 - Layering of soil strata by best-fit lines

Figure 4.10 (b) Site 2 - Layering of soil strata by best-fit lines

Figure 4.10 (c) Site 3 - Layering of soil strata by best-fit lines

Figure 4.10 (d) Site 4 - Layering of soil strata by best-fit lines

Figure 4.10 (e) Site 5 - Layering of soil strata by best-fit lines

Figure 4.10 (f) Site 6 - Layering of soil strata by best-fit lines

Figure 4.10 (g) Site 7 - Layering of soil strata by best-fit lines

Table 4.2 Wavelength-depth factor for the seven sites

Site	Wavelength-depth factor (λ/z)
Site 1	3.50
Site 2	3.50
Site 3	3.50
Site 4	3.48
Site 5	3.64
Site 6	3.65
Site 7	3.00

Since the range of λ/z for the seven test sites are within a narrow range of 3.0 to 3.65, dispersion data from the literature where borehole or equivalent information is available are collected to provide a data set where λ/z varies from 2 to 8. The data collected from the literature are briefly described below. In those case studies the shear wave velocity V_s is directly converted from the phase velocity V_ϕ using the

elastic equation ($V_S = pV_\phi$) as described in Chapter 2. In the modified wavelength-depth method, it is suggested that V_ϕ at the interface of two layers be taken as the approximate V_ϕ for the lower layer in order to generate a layered earth model which can be used as the initial model for more accurate inversion scheme.

Louie (2001) demonstrated that microtremor surveys conducted using 200 m long lines of seismic refraction equipment could estimate shear wave velocity profile with 20% accuracy up to a depth of 100 m. The microtremor surveys were conducted at two locations: at a large accelerometer microtremor array in Reno, Nevada, and above a borehole logged for shear wave velocity in Newhall, California. Only the latter case is used as the uncertainty of the soil profile is much less for the microtremor survey conducted above the borehole logged for shear wave. The dispersion curve and the reported shear wave velocity model are shown in Figure 4.11. The best λ/z for this site is 3.9.

Figure 4.11 Dispersion curve and shear wave velocity model from Louie (2001)

Menzies (2001) reported CSW tests for four sites in UK. Only the site in northern France where borehole information is available is used. The borehole log at this site shows the top 4.5 m consists of weathered sandy clay followed by a silty clay layer of 4.5 m thick overlying a thick deposit of clay marl. The dispersion curve together with the borehole information is shown in Figure 4.12. The best fit λ/z for this site is 2.5.

Figure 4.12 Dispersion curve and shear wave velocity model from Menzies (2001)

Luke and Calderón-Macías (2007) investigated a complex layered site where thin cemented layers are found within an alluvial deposit. The borehole log showed semi-continuous cemented layers ranging to 1 m thick were found at depths of 0.6, 7.5, 16 and 29 m. The dispersion curve together with the borehole information is shown in Figure 4.13. The best fit λ/z for this site is 5. It is seen that this method could not correctly show the stiff layer. However, it is strictly incorrect to assume that the Luke and Calderon-Macias (2007) profile is correct. Figure 4.13 shows that the velocity profiles are comparable at least in layering.

Figure 4.13 Dispersion curve and shear wave velocity model from Luke and Calderón-Macías (2007)

Two cases of numerical studies (Ganji et al., 1998 and Anand, 2000), where artificial normally dispersive soil profiles were used to generate the dispersion curves were also included in this study. Ganji et al. (1998) studied the automated inversion procedure for spectral analysis of surface waves. Six cases were described but only two cases have a normally dispersive soil profile. Therefore, only two normally dispersive cases are used and are shown in Figure 4.14. The best fit λ/z for the two cases are 2 and 2.4, respectively. Anand (2007) modelled CSW test using LS-DYNA. One case with normally dispersive soil profile is shown in Figure 4.15. The best fit λ/z for this case is 3.6.

(a) Dispersion curve

(b) Shear wave velocity model

(a) Case 1

(a) Dispersion curve

(b) Shear wave velocity model

(b) Case 3

Figure 4.14 Dispersion curve and shear wave velocity model from Ganji et al. (1998)

Figure 4.15 Dispersion curve and shear wave velocity model from Anand (2000)

From each dispersion curve, λ corresponding to several frequencies can be identified. Using the above cases, the values of λ/z are plotted against λ in Figure 4.16(a). Same points are plotted in Figure 4.16(b) to show the points according to frequency. Figure 4.16(b) indicates that there is a linear relationship between λ/z and λ for each frequency f and the linear relationship can be expressed as follows:

$$\frac{\lambda}{z} = n(f)\lambda + 2 \quad (4.1)$$

where n having units of $1/\text{length}$ is the slope of the linear relationship and is a function of frequency f . The relationship of n with frequency f is shown in Figure 4.17. Figure 4.17 suggests that there are two horizontal asymptotes, one at $n = 0$ and one at $n = 0.2$. The sigmoid relationship between n and f can be approximated using the following equation:

$$n = \frac{1}{5.31 + [0.06(1 + f)]^{-4.58}} + 0.015 \quad (4.2)$$

Using Equation 4.2, a series of λ/z ratios versus λ relationships can be generated for various frequencies f as shown in Figure 4.18.

(a) Data according to Sites

(b) Data according to frequency

Figure 4.16 Wavelength-depth ratio for maximum wavelength of investigation

Figure 4.17 Relationship of n with frequency f

Figure 4.18 Chart for modified wavelength-depth method

4.4.2 Evaluation of modified wavelength-depth method

The validity of the modified wavelength-depth method is evaluated using published cases where both dispersion curve and soil profile information are available. As the modified wavelength-depth method was developed from normally dispersive sites, only normally dispersive sites are selected for the evaluation. The case histories used are from Tokimatsu et al. (1992), Tokimatsu and Miyadera (1992), Tokimatsu (1995), Tokimatsu and Arai (1995), Matsuoka et al. (1996), and Rosenblad et al. (2002).

Tokimatsu et al. (1992) studied the effects of multiple modes on Rayleigh wave dispersion using three four-layer models. However, only one case (Case 1) was normally dispersive. The properties of the four-layer model are summarized in Table 4.3. The dispersion curve is shown in Figure 4.19(a). Using Figure 4.18 and the dispersion curve, λ/z for Case 1 is 3.4. The shear wave velocity profile from the modified wavelength-depth method is compared with the four-layer model in Figure

4.19(b). The comparison shows a good match for both layer thickness and shear wave velocity.

Table 4.3 Four-layer model for Case 1 (Tokimatsu et al. 1992)

Layer No.	Thickness, h (m)	Bulk density, ρ (Mg/m^3)	V_p (m/s)	V_s (m/s)
1	2	1.8	360	80
2	4	1.8	1000	120
3	8	1.8	1400	180
4	-	1.8	1400	360

Figure 4.19 Evaluation of modified wavelength-depth method for data from Tokimatsu et al. (1992)

Tokimatsu and Miyadera (1992) conducted microtremor surveys using a circular array of six vertical geophones at two sites: Kamidokoro in Niigata and Daiba in Tokyo, Japan. The site profile at Kamidokoro, Niigata, is normally dispersive and the dispersion curve is shown in Figure 4.20(a). From Figure 4.18 and the dispersion

curve, the λ/z for this site is 4.7. The shear wave velocity profile using the modified wavelength method and is compared the shear wave velocity profile obtained by Tokimatsu and Miyadera (1992) in Figure 4.20(b). Tokimatsu and Miyadera (1992) found that their inversion gave a good agreement with the shear wave velocity profile obtained from downhole method. Figure 4.20(b) shows the modified wavelength method was able to estimate the layer thickness accurately while the difference in shear wave velocity ranges from -35% to 26%.

Figure 4.20 Evaluation of modified wavelength-depth method for data from Tokimatsu and Miyadera (1992)

Tokimatsu et al. (1995) conducted microtremor surveys at two sites: Yumenoshima and Midorigaoka. However only the Midorigaoka site is normally dispersive where the borehole log shows a thin soft clay layer overlying a stiff layer. The dispersion curve for the Midorigaoka site is shown in Figure 4.21(a). Using Figure 4.18 and the dispersion curve, the λ/z for this site is 5.1. The shear wave velocity profile from the modified wavelength-depth method is compared with the shear wave velocity profile

from the geological logs of the Midorigaoka site in Figure 4.21(b). The comparison shows a reasonable match for layer thickness, but shear wave velocity is off by $\pm 35\%$.

Figure 4.21 Evaluation of modified wavelength-depth method for data from Tokimatsu et al. (1995)

Tokimatsu and Arai (1995) studied the use of short-period microtremors for estimating local site effects at two strong motion stations: Kushiro Japan Metrological Agency and Kushiro Harbour. However, only the site at Kushiro Japan Metrological Agency is normally dispersive. The dispersion curve at this site is shown in Figure 4.22(a). Using Figure 4.18 and the dispersion curve, the λ/z for this site is 8. The shear wave velocity profile from the modified wavelength-depth method is compared with the inverted shear wave velocity profile from Tokimatsu and Arai (1995) in Figure 4.22(b). The comparison shows a reasonable match for layer thickness, but shear wave velocity is underestimated by 67% at shallow depth and overestimated by 30% at deeper depth.

Figure 4.22 Evaluation of modified wavelength-depth method for data from Tokimatsu and Arai (1995)

Matsuoka et al. (1996) showed the results of SPAC method for microtremor surveys at four locations on the coast of Japan: Center, Kusakabe, Omiya and Nishi. The shear wave velocity profiles of these sites are available from well-log and refraction seismic surveys. Except for location Center, the other three site profiles are approximately normally dispersive. The dispersion curves of locations Kusakabe, Omiya and Nishi are shown in Figure 4.23(a), Figure 4.24(a) and Figure 4.25(a), respectively. Using Figure 4.18 and the dispersion curves, λ/z for locations Kusakabe, Omiya and Nishi are all 8. The shear wave velocity profiles using the modified wavelength-depth method are compared with the shear wave velocity profiles from the well-log and refraction seismic surveys for locations Kusakabe, Omiya and Nishi in Figure 4.23(b), Figure 4.24(b) and Figure 4.25(b), respectively. Figure 4.23(b), Figure 4.24(b) and Figure 4.25(b) show that the thicknesses of the soil layers could be estimated quite accurately but the shear wave velocity can be over- or underestimated by as much as 135%.

Figure 4.23 Evaluation of modified wavelength-depth method for Kusakabe site from Matsuoka et al. (1996)

Figure 4.24 Evaluation of modified wavelength-depth method for Omiya site from Matsuoka et al. (1996)

Figure 4.25 Evaluation of modified wavelength-depth method for Nishi site from Matsuoka et al. (1996)

Rosenblad et al. (2002) reported the shear wave velocity profiling at 16 strong motion stations and 4 aftershock stations in Turkey using spectral analysis of surface waves (SASW) and microtremors. The results are divided into three general categories: (1) Deep profile and laterally uniform site, (2) Deep profile and laterally variable site, and (3) shallow profile. The dispersion curve was inverted by forward modelling using the dynamic stiffness method developed by Kausel and Roesset (1981) implemented in a computer program WinSASW (Joh 1996). The shear wave velocities and thicknesses of the soil layers were adjusted by trial and error until a satisfactory match was obtained between the theoretical and field dispersion curve. At some sites, there were discrepancies between the dispersion curves from SASW and microtremors. This was attributed to the difference in scale of the SASW and microtremors tests and lateral variability. As the modified wavelength depth method is meant for vertically heterogeneous sites, shear wave profiles for sites under categories 2 and 3 are strictly not applicable. Therefore only category 1 sites i.e. Ambarli, Ballica, Cekmece, Duzce and LD-10 are evaluated.

Ambarli station is located on the grounds of a power generation plant in western Istanbul situated in an alluvial valley that is less than 5 km wide. Spectral Analysis of Surface Wave (SASW) survey was conducted using hammers and a bulldozer as wave source about 25 m from the strong motion station. The dispersion curve at this site is shown in Figure 4.26 (a). Using Figure 4.18 and the dispersion curve, the λ/z for this site is 3.4. The shear wave velocity profile from the modified wavelength-depth method is compared with the inverted shear wave velocity profile from SASW survey reported by Rosenblad et al. (2002) in Figure 4.26 (b). The comparison shows a good match for layer thickness and shear wave velocity.

Figure 4.26 Evaluation of modified wavelength-depth method for Ambarli station site from Rosenblad et al. (2002)

Ballica station is located in a small town of the same name in the Duzce alluvial basin, south of the city of Duzce. SASW survey was conducted using hammers and a bulldozer as wave source approximately 75 m from the Ballica station. The dispersion curve at this site is shown in Figure 4.27(a). Using Figure 4.18 and the dispersion curve, the λ/z for this site is 3.1. The shear wave velocity profile from the modified wavelength-depth method is compared with the inverted shear wave velocity profile

from SASW survey reported by Rosenblad et al. (2002) in Figure 4.27(b). The comparison shows a good match for layer thickness and shear wave velocity.

Figure 4.27 Evaluation of modified wavelength-depth method for Ballica station site from Rosenblad et al. (2002)

Cekmece station is located on the campus of a nuclear research facility in western Istanbul. SASW survey was conducted using hammers and a bulldozer as wave source approximately 60 m from the Cekmece station. The dispersion curve at this site is shown in Figure 4.28(a). Using Figure 4.18 and the dispersion curve, the λ/z for this site is 5.1. The shear wave velocity profile from the modified wavelength-depth method is compared with the inverted shear wave velocity profile from SASW survey reported by Rosenblad et al. (2002) in Figure 4.28(b). The comparison shows a good match for the upper soil layers and a reasonable match for the lower soil layers.

Figure 4.28 Evaluation of modified wavelength-depth method for Cekmece station site from Rosenblad et al. (2002)

Duzce station is located in the middle of Duzce city and lies in the large Duzce alluvial basin. SASW survey was conducted using hammers and a bulldozer as wave source approximately 60 m from the Cekmece station. The dispersion curve at this site is shown in Figure 4.29(a). Using Figure 4.18 and the dispersion curve, the λ/z for this site is 4. The shear wave velocity profile from the modified wavelength-depth method is compared with the inverted shear wave velocity profile from SASW survey reported by Rosenblad et al. (2002) in Figure 4.29(b). The comparison shows a reasonable match for layer thickness and shear wave velocity.

LD-10 station is a temporary station located in the mountainous area between Akyazi and Duzce. SASW survey was conducted using hammers and a bulldozer as wave source approximately 150 m from the LD-10 station. The dispersion curve at this site is shown in Figure 4.30(a). Using Figure 4.18 and the dispersion curve, the λ/z for this site is 8. The shear wave velocity profile from the modified wavelength-depth method is compared with the inverted shear wave velocity profile from SASW survey reported by Rosenblad et al. (2002) in Figure 4.30(b). The comparison shows a good match for layer thickness and shear wave velocity.

Figure 4.29 Evaluation of modified wavelength-depth method for Duzce station site from Rosenblad et al. (2002)

Figure 4.30 Evaluation of modified wavelength-depth method for LD-10 station site from Rosenblad et al. (2002)

The modified wavelength-depth method has been verified in this section. The literature review on the wavelength-depth method already explained the reason for using a maximum value of $\lambda_\phi/z = 8$. It was shown in Figure 2.15 and Figure 2.16, modified from Gazetas (1982) and Vrettos (1990), respectively, that λ_ϕ/z can be up to a value of 8 and therefore $\lambda_\phi/z = 8$ was used. It is also discussed in the literature that the higher the frequency the lower the wavelength or depth of investigation. The proposed method is most accurate if the largest wavelength possible is used within the bounds of Figure 4.18. Other values of λ/z will give a less accurate answer as the largest wavelength captures the average soil properties to a deeper depth.

4.5 Summary

The general procedure of obtaining the dispersion curve from CSW test was illustrated. The CSW surveys along different orthogonal lines at the seven sites showed that the sites are all horizontally stratified. In this chapter, a modified wavelength-depth method was also suggested. In the modified wavelength-depth method, a chart to determine the wavelength-depth factor λ/z using the dispersion curve was developed from CSW tests and data from the literature. From the dispersion curve, a change in gradient of the dispersion curve was identified as a change in soil type and the wavelength of the dispersion curve at the change point was converted to an equivalent depth using λ/z . The shear wave velocity of the layer was computed from the phase velocity at the top of the layer. Verification of the modified wavelength-depth method was performed using data from the literature. Reasonable shear wave velocity profiles can be obtained where good estimate of layer thickness can be obtained while shear wave velocity estimate was considered reasonable. The modified wavelength-depth method will be useful in providing an initial soil model for more accurate inversion methods.

CHAPTER 5

ANALYSIS OF MICROTREMOR SURVEY RESULTS USING SPAC METHOD

5.1 Introduction

Microtremor survey can be analyzed by different methods. SPAC method and F-K method have been used to analyze microtremor survey. SPAC method needs less number of stations, smaller array size and the depth of investigation can be 1.5 to 2 times higher than other methods (Okada, 2004). The SPAC method which produces a dispersion curve from the microtremor survey results is described in this chapter. In this study, a Matlab script file has been written for the SPAC method of analysis. The dispersion curve is then inverted using the linear inversion method of Lai and Rix (1998) to obtain the shear wave velocity profile. One of the objectives of this chapter is to verify the correctness of the implemented SPAC method in Matlab using case histories from the literature. Four case histories are selected based on completeness of essential data needed for interpretation of microtremor surveys i.e. data from receivers, spatial auto-correlation coefficients, dispersion curve and shear wave velocity profile.

CSW method uses an active energy source where the frequency range is from 5 Hz to 600 Hz and the investigation depth is limited to a depth of about 10 m for soft soil and 30 m under stiff soil conditions (Matthews et al., 1996). Microtremor survey is dependent on very low frequency passive surface waves. Hence, the depth of investigation for microtremor survey can be very deep. Investigation depth of 4000 m was achieved by Estrella and González (2003). Since there are some differences in CSW and microtremor survey methods in collecting data and in case of depth of investigation dispersion curves from both the methods, are compared in this chapter. This chapter will also compare and discuss the inverted shear wave velocity profiles from CSW and microtremor survey methods. Furthermore, the shear wave velocity profiles will also be compared with the borehole logs.

5.2 Estimating the Dispersion Curve

The SPAC method based on Aki (1957) theory, described in the literature review, was implemented in a Matlab script file (see Appendix B). For case histories where raw data from microtremor surveys are available, these are digitized to obtain the spatial auto-correlation coefficients. For cases where raw data from the microtremor surveys are not available, the spatial auto-correlation coefficients are usually available and are digitized. The spatial auto-correlation coefficients are then fitted with Bessel function of first kind of zero order J_0 given as:

$$\bar{\rho}(\xi, \omega_0) = J_0(x) \quad (5.1)$$

$$x = \frac{2\pi f r}{V_\phi} \quad (5.2)$$

where $\bar{\rho}(\xi, \omega_0)$ = spatial auto-correlation coefficients, f = frequency, r = radius of the arrays, and V_ϕ = phase velocity. An example of fitting the Bessel function J_0 to the auto-correlation coefficient for one of the microtremor surveys conducted in this study is shown Figure 5.1.

Figure 5.1 Fitted Bessel function J_0 with spatial auto-correlation coefficients

The values of x are calculated from the fitted Bessel function J_0 . Using Equation (5.2), the phase velocity V_ϕ is calculated. The dispersion curve is given by a plot of V_ϕ versus f (Figure 5.2).

Figure 5.2 Dispersion curve calculated using SPAC method

5.3 Case History 1: Estrella and González (2003)

5.3.1 Description

Estrella and González (2003) estimated deep S-wave velocity structures in Mexico City using the SPAC method proposed by Aki (1957) and compared with those obtained by Kagawa (1996) who used the F-K method. The conclusion of the research was that the velocity structure could be estimated reliably from microtremor survey using the SPAC method. The arrangement of the geophones in their survey is shown in Figure 5.3. The first geophone (geophone 1) was placed at the centre of the triangle and the other three geophones (geophones 2, 3 and 4) were deployed at the vertices of an equilateral triangle with sides of 1000 m.

Duration of the data collection was about 40 minutes with a sampling interval of 0.01 s (i.e. sampling frequency of 100 Hz). The field data were first separated into six time windows before analysis and then spatial auto correlation coefficients were calculated for all six time windows separately.

The spatial auto-correlation coefficients were computed from either a combination of the central geophone with any other geophone at the vertices (2, 3 or 4) of the triangular array (i.e. 1-2, 1-3, or 1-4 combination) denoted as internal combination, or a combination of any two geophones at the vertices of the triangular array (i.e. 2-3, 3-4, and 4-2 combination) denoted as external combination.

Figure 5.3 Arrangement of the geophones for Case History 1

The presented spatial auto-correlation coefficients were averaged, either for internal (i.e. average of spatial auto-correlation coefficients for 1-2, 1-3 and 1-4) or external (i.e. average of spatial auto-correlation coefficients for 2-3, 3-4 and 4-2) combination of the geophones for each of the six time windows. The dispersion curve was calculated for the frequency range from 0.5 Hz to 0.75 Hz for the internal combination of geophones and from 0.26 Hz to 0.67 Hz for the external combination of the geophones.

5.3.2 Estimating the Dispersion Curve

The average spatial auto-correlation coefficients presented by Estrella and González (2003) for internal and external combinations of geophones for each of the time windows are shown in Figure 5.4.

As the SPAC method assumes spatial and temporal stationarity of the microtremors, the spatial auto-correlation coefficients should be similar for the internal and external

combination of geophones as well as for the six time windows. However, from Figure 5.4 the spatial and temporal stationarity of the microtremors is only approximately satisfied for the external combination of geophones. For the frequency range from 0.2 Hz to 0.75 Hz, both internal and external combinations of geophones gave similar spatial auto-correlation coefficients. The average spatial auto-correlation coefficients for the internal and external combinations of the geophones were curve-fitted with a Bessel function J_0 shown as symbols in Figure 5.5. It can be seen from Figure 5.5 that the fitted curves are satisfactory.

Figure 5.4 Spatial auto-correlation coefficients for Case History 1 (from Estrella and González, 2003)

Figure 5.5 Fitted Bessel functions J_0 for Case History 1

After obtaining the Bessel function J_0 for the average spatial auto-correlation coefficients, the phase velocities for both internal and external combinations were calculated using Equation (5.2). The calculated phase velocities are plotted against frequency to give the dispersion curve as shown in Figure 5.6. Figure 5.6 shows that the dispersion curves for both the internal and external combinations of geophones do not coincide. The differences indicate the uncertainty in the microtremor survey method in estimating the phase velocity. However, individual comparison of dispersion curves, either for external combination or for internal combination of the geophones, is quite good. This verified the calculation procedures of the SPAC method implemented in Matlab for this research.

Figure 5.6 Comparison of dispersion curves for Case History 1

5.3.3 Estimating the Shear Wave Velocity Profile

The shear wave velocity profile was obtained by inverting the dispersion curve for external combination of geophones using Lai and Rix (1998) linear inversion method. Estrella and González (2003) did not present any description of the inversion method that they used. Both the computed and Estrella and González (2003) shear wave velocity profile are shown in Figure 5.7. The computed shear wave velocity profile agrees well with that obtained by Estrella and González (2003). The difference is attributed to the different inversion algorithm used.

Figure 5.7 Comparison of shear wave velocity profiles for Case History 1

5.4 Case History 2: Matsuoka et al. (2003)

5.4.1 Description

Matsuoka et al. (2003) estimated the deep V_s profiles in the central part of the Kanto plain, Japan, using the SPAC method at 67 sites. Each site was about 5 km away from other site. Microtremor survey was performed with three circular arrays at each site. Each array consisted of four seismometers, with radii of 100 m, 300 m and 600 m as shown in Figure 5.8. Subsequently, the arrays with radii of 100 m, 300 m and 600 m will be denoted by $r = 100$ m, $r = 300$ m and $r = 600$ m, respectively, to facilitate discussion.

The duration of the collected data was about 50 minutes with a sampling interval of 0.02 s (i.e. sampling frequency of 50 Hz). A low-pass filter with cut-off frequency of 1 Hz was used.

Figure 5.8 Arrangement of geophones for Case History 2 (from Matsuoka et al., 2003)

5.4.2 Estimating the Dispersion Curve

To describe the SPAC method of determining the dispersion curve Matsuoka et al. (2003) used only one site data but did not name of the site. The obtained phase velocities were assumed fundamental-mode Rayleigh waves in their microtremor surveys. The wave frequencies range from 0.2 Hz to 2 Hz. Matsuoka et al. (2003) presented only the spatial auto-correlation coefficients. Therefore, Bessel functions J_0 are curve-fitted to the spatial auto-correlation coefficients. The fitted Bessel functions J_0 are shown in Figure 5.9. The dispersion curves obtained using the SPAC method is shown in Figure 5.10. It is seen from the dispersion curve (Figure 5.10) that the calculated phase velocity for $r = 100$ m and $r = 600$ m arrays fitted the dispersion curve obtained by Matsuoka et al. (2003) well except for the $r = 300$ m array. In order to check the discrepancy of the $r = 300$ m array, spatial auto-correlation coefficients were determined by back calculation from the dispersion curve data of Matsuoka et al. (2003). Figure 5.11 shows the back-calculated spatial auto-correlation coefficients from the dispersion curve of Matsuoka et al. (2003). It seems that Matsuoka et al. (2003) only used half of the spatial auto-correlation coefficients for the $r = 300$ m array to calculate the phase velocity. This explained the differences in phase velocities for the $r = 300$ m array in Figure 5.10.

Figure 5.9 Fitted Bessel function J_0 for Case History 2

Figure 5.10 Comparison of estimated dispersion curve for Case History 2

Figure 5.11 Comparison of spatial auto-correlation coefficients and back-calculated spatial auto-correlation coefficients for Case History 2.

The Bessel function J_0 fitted for the frequency range as used by Matsuoka et al. (2003) and the dispersion curve were recomputed and new results are shown in Figure 5.12 and Figure 5.13, respectively. It is observed that the results compared well with those of Matsuoka et al. (2003) as expected because both sets of results were based on the SPAC method.

Figure 5.12 Comparison of refitted Bessel function J_0 for Case History 2

Figure 5.13 Comparison of dispersion curves for Case History 2

5.4.3 Estimating the Shear Wave Velocity Profile

Matsuoka et al. (2003) conducted microtremor surveys at 67 sites. However, only three sites: IWS, TZS and HKS were reported to find the V_S profile. Dispersion curves for the three sites are shown in Figure 5.14.

(a) IWS site

(b) TZS site

(c) HKS site

Figure 5.14 Dispersion curves for different sites (from Matsuoka et al., 2003)

Matsuoka et al. (2003) inverted the dispersion curves using the inversion method based on a genetic algorithm developed by Yamanaka and Ishida (1995). In the inversion, V_S and h were treated as variables while V_P and ρ were assumed constant values. In this study, the dispersion curves in Figure 5.14 were used to find the V_S profiles using Lai and Rix (1998) linear inversion method.

Figure 5.15 Comparison of shear wave velocity profile for Case History 2

The computed and Matsuoka et al. (2003) V_S profiles are shown in Figure 5.15 for IWS, TZS and HKS sites. Figure 5.15 shows that the shear wave velocity profiles show reasonable agreement with those obtained by Matsuoka et al. (2003) except for the shear wave velocity in the lowest soil layer. The discrepancy is attributed to the different inversion schemes used.

5.5 Case History 3: Bettig et al. (2001)

5.5.1 Description

Bettig et al. (2001) applied modified spatial auto-correlation coefficients method (modified SPAC method) to obtain velocity profiles for compression and shear waves. The microtremor surveys were performed at Grenoble which has very thick late quaternary deposits. The results were compared with existing geophysical information from gravimetric, seismic reflection and borehole data.

Figure 5.16 Microtremor survey array for Case History 3. The triangles show the location of the stations (large array); the small circle in the upper part of the figure corresponds to the small array location (from Bettig et al., 2001).

Microtremor survey was performed using circular arrays with radii ranging from 53 m to 600 m as shown in Figure 5.16. Data were collected for at least three hours. The modified SPAC method was used to derive the Rayleigh wave dispersion curve for each array. The averaged spatial auto-correlation ratios for several radii were computed for several time windows.

5.5.2 Estimating the Dispersion Curve

Spatial auto-correlation coefficients were averaged for different arrays diameters. The fitted Bessel functions J_0 for the spatial auto-correlation coefficients of the different arrays are shown in Figure 5.17. Using the fitted Bessel functions J_0 , the dispersion curves were obtained and compared with Bettig et al. (2001) average dispersion curve in Figure 5.18.

(a)

(b)

Figure 5.17 Comparison of spatial auto correlation coefficients for Case History 3

The dispersion curve obtained by Bettig et al. (2001) was from the averaged spatial auto-correlation coefficients for different array diameters. Figure 5.17 shows that the individual dispersion curve agrees well with the average dispersion curve of Bettig et al. (2001). However, Bettig et al. (2001) used the modified SPAC method and some discrepancies are expected between the dispersion curves and Bettig et al. (2001) average dispersion curve.

Figure 5.18 Comparison of dispersion curves for Case History 3

5.5.3 Estimating the Shear Wave Velocity Profile

Bettig et al. (2001) used a nonlinear inversion method based on least squares adjustment (Tarantola and Valette, 1992) to obtain the shear wave velocity profile. In this study, the average dispersion curve of Bettig et al. (2001) was inverted using Lai and Rix (1998) linear inversion method to obtain the shear wave velocity profile. A comparison of the calculated shear wave velocity profile with that obtained by Bettig et al. (2001) is shown in Figure 5.19. The agreement for the shear wave velocity profiles is very good.

Figure 5.19 Comparison of shear wave velocity profiles for Case History 3

5.6 Case History 4: Saccorotti et al. (1999)

5.6.1 Description

Saccorotti et al. (1999) described the operation of two short-period seismic arrays deployed at Deception volcano, Southern Shetland Island, Antarctica. Using the assumption that the wave field of the background seismic noise is dominated by scattered surface waves, Aki (1957) method was applied to investigate the dispersion properties of Rayleigh waves in the 1 Hz to 8 Hz frequency band.

The array was deployed at the top of an alluvial fan. The array has an aperture of approximately 240 m with stations located along two semicircles, one of radius 120 m and the other of radius 60 m (Figure 5.20). The angular spacings between stations of the outer and inner semicircles were 45° and 60° , respectively. The array was equipped with three-component geophone having a natural frequency of 1 Hz. The sampling frequency was 200 Hz per channel. The background noise for 150 s long duration was periodically recorded by the array.

Figure 5.20 Layout of array for Case History 4 (from Saccorotti et al., 1999)

5.6.2 Estimating the Dispersion Curve

Saccorotti et al. (1999) used Aki (1957) SPAC method to obtain the spatial auto-correlation coefficients. The spatial auto-correlation coefficients presented by Saccorotti et al. (1999) were fitted with Bessel function J_0 as shown in Figure 5.21 for $r = 120$ m and $r = 60$ m.

Figure 5.21 Fitted Bessel function J_0 for Case History 4

Figure 5.22 Comparison of dispersion curve for Case History 4

The dispersion curves computed by the fitted Bessel function J_0 are compared with the dispersion curve of Saccorotti et al. (1999) in Figure 5.22. There is reasonable agreement between the dispersion curves. Saccorotti et al. (1999) did not present a shear wave velocity profile or borehole information, therefore inversion of the dispersion curve was not performed for this case.

5.7 Summary

Four case histories are presented in this chapter with comparisons of Bessel functions, J_0 dispersion curves and shear wave velocity profiles. Out of the four case histories, Case Histories 1, 2 and 4 used Aki (1957) SPAC method and Case History 3 used the modified SPAC method. Although Aki (1957) SPAC method is also used in this report, the algorithm for calculation of spatial auto-correlation coefficients and curve-fitting with Bessel functions J_0 are different from the case histories. The use of different algorithms for calculation might cause some discrepancies in the spatial auto-correlation coefficients and hence the dispersion curve.

Inversion of the dispersion curve to calculate the shear wave velocity profile can be performed using different algorithms as mentioned in Chapter 2. This research used Lai and Rix (1998) linear inversion method implemented in a Matlab script file. This inversion program requires the thickness of the soil layers, Poisson's ratio, density of soil and initial shear wave velocity for each layer of soils as input parameters. The

layer thickness was manually adjusted and input into Lai and Rix (1998) linear inversion program. The case histories did not elaborate on the inversion method used. Differences in the inversion methods caused the discrepancies in the shear wave velocity profiles obtained.

Even though there are differences in the computation of the dispersion curve and inversion process, the comparisons presented in this chapter generally showed good agreement. This verified the SPAC method as implemented in Matlab and the inversion procedures used.

5.8 Dispersion Curves of Microtremor Survey Conducted in Singapore

In this study, microtremor surveys were conducted at the seven sites mentioned in Chapter 3. The microtremor surveys were performed using the CSWS sans the vibrator. The CSWS software was modified (Courtesy of Snelling, 2004) to enable continuous recording of the geophones for 15 minutes. The vertical geophones were arranged into a triangular array with one geophone at the centroid of the triangle as described in Chapter 3.

The vertical component of microtremor is predominantly Rayleigh waves (Tokimatsu, 1997; Asten et al., 2003). The dispersion curves obtained using the SPAC method for the seven sites are shown in Figure 5.23 to Figure 5.29. Lai and Rix (1998) method was used to invert the dispersion curves.

Figure 5.23 Dispersion curve for Site 1

Figure 5.24 Dispersion curves for Site 2

Figure 5.25 Dispersion curve for Site 3

Figure 5.26 Dispersion curve for Site 4

Figure 5.27 Dispersion curve for Site 5

Figure 5.28 Dispersion curve for Site 6

Figure 5.29 Dispersion curve for Site 7

5.9 Comparison of Field Dispersion Curves from CSW and Microtremor Survey Methods

For the seven sites, CSW and microtremor surveys were conducted on the same day, same spot and under similar testing conditions. Borehole log information are available for all the sites. As described in Chapter 3, four orthogonal CSW survey lines were conducted for each site to determine the lateral variation of the soil profile. The field dispersion curves from CSW and microtremor surveys are compared in Figure 5.30 to Figure 5.36 for the seven sites, respectively. The figures show that the dispersion curve from CSW survey match well with the dispersion curve from microtremor survey method for Site 7. For sites 1, 3, 4, 5 and 6, the dispersion curve overlap but the trend of the dispersion curve from the microtremor survey is different from the CSW survey. The results are not in close agreement because of the differences in principles of the two methods and the differing accuracy of the two methods. The microtremor survey method has a lower resolution. It is a well known fact that shear wave velocities from different geophysical methods show differences due to the

different principles of the methods. The reason for the different trend observed in the cases of Sites 1 to 6 is due to the resolution of the microtremor survey data, as discussed in the literature review; the microtremor survey does not give very accurate resolutions. Therefore, data at the deeper part of the dispersion curve gave lower phase velocity of the field. If the array size and depth of the investigation for the microtremor surveys are compared, it can be observed that the dispersion curves of Sites 2 to 6 have depths of investigation in term of wavelength as 45, 37, 41, 40 and 32 m, respectively, and for Site 7 it was 18 m, whereas array size was almost the same for Sites 2 to 7. This observation implies that the depth of investigation is consistent with the array size and the deeper the depth of investigation the lesser the accuracy of the microtremor survey method when compared with CSW survey method.

Furthermore it was observed that the depth of investigations were lower in microtremor survey. The effective penetration depth for any array size in microtremor survey is limited to between twice the minimum sensor distance and thrice the maximum sensor distance of the array (Asten and Henstridge, 1984; Tokimatsu, 1997). Arrays of larger diameter will enable a deeper depth of investigation (Estrella and González, 2003; Matsuoka et al., 2003, Bettig et al., 2001 and Saccorotti et al., 1999). In this study, array sizes were $r = 1$ m for Site 1, $r = 5$ m and 10 m for Site 2, $r = 4$ m for Site 3 and $r = 5$ for Sites 4, 5, 6 and 7. Dispersion curves for Site 2 with different array sizes are shown in Figure 5.37. It can be seen from Figure 5.37 that the $r = 5$ m array can investigate to a wavelength depth of 40 m and $r = 10$ m array can investigate to a wavelength depth of more than 50 m. Due to site constraint at the other sites, the array size was limited to between $r = 1$ m to $r = 5$ m.

The depth of investigation in CSW method is dependent on the source and soil type. The stiffer the soil, the greater is the depth of investigation. Generally the depth of investigation can be up to 30 m. In this research, the maximum depth of investigation by CSW was up to about 20 m. The effect of array size is briefly described in Section 2.6 and the array sizes used by other researchers are much larger than those used in this research as can be seen in Table 2.1. The maximum array size used in this research was $r = 10$. Therefore, the depth of investigation obtained in this research is consistent with site condition in case of CSW method and with array size in case of microtremor survey method.

Figure 5.30 Comparison of field dispersion curves for Site 1

Figure 5.31 Comparison of field dispersion curves for Site 2

Figure 5.32 Comparison of field dispersion curves for Site 3

Figure 5.33 Comparison of field dispersion curves for Site 4

Figure 5.34 Comparison of field dispersion curves for Site 5

Figure 5.35 Comparison of field dispersion curves for Site 6

Figure 5.36 Comparison of field dispersion curves for Site 7

Figure 5.37 Dispersion curve from Site 2 for different array sizes ($r = 5$ m and $r = 10$ m)

The dispersion curves from CSW using Lai and Rix (1998) linear inversion method and modified wavelength method and microtremor surveys were inverted using Lai and Rix (1998) linear inversion method. The shear wave velocity profiles of the seven

sites are shown in Figure 5.38 to Figure 5.44. The comparison showed reasonable agreement in the shear wave velocity profiles obtained from inversion of dispersion curves of CSW and microtremor surveys.

The shear wave velocity profiles are also in good agreement with borehole log data. The strata descriptions are as given in the borehole logs given in Appendix A. The site investigations were performed in other projects and there will be inconsistencies in the description as different site investigation companies performed the works. Therefore the strata descriptions should only be viewed as qualitative descriptions. For site 1, the phase velocity V_ϕ below 8 m increases from 200 m/s to 550 m/s (Figure 4.10a). The V_s obtained from the inversion analysis gave the lower limit of 220 m/s. For sites 2 to 6, the V_s shows good agreement with SPT N values. For site 7, the maximum depth of investigation in both methods is about 7m. The borehole log indicates that 0 to 5m consists of a backfill. The 200 m/s reported for V_s probably reflects more the interface condition between the backfill and soft marine clay and not the V_s of the marine clay which should be in the region of about 100 m/s.

The assumption used in the inversion is that the dispersion curve is dominated by the fundamental mode. Zhang and Chan (2003) showed that only the fundamental mode is able to give an accurate soil profile. Therefore, it is evident that the dispersion curves obtained from the CSW and microtremor surveys are predominantly of the fundamental mode. Figure 5.38 to Figure 5.43 show that for Sites 1 to 6, the inverted shear wave velocity profiles are still in good agreement although the dispersion curves do not coincide.

Figure 5.38 Shear wave velocity profiles for Site 1

Figure 5.39 Shear wave velocity profiles for Site 2

Figure 5.40 Shear wave velocity profiles for Site 3

Figure 5.41 Shear wave velocity profiles for Site 4

Figure 5.42 Shear wave velocity profiles for Site 5

Figure 5.43 Shear wave velocity profiles for Site 6

Figure 5.44 Shear wave velocity profiles for Site 7

5.10 Summary

The correctness of the implemented SPAC method to analyze microtremor survey data in Matlab has been established by using case histories from the literature. The dispersion curves obtained from CSW method and microtremor survey method were compared in this chapter. The comparison showed reasonable match although the trends of the dispersion curve from CSW and microtremor survey were not similar. It was also found from the comparison that depth of investigation was shallower in case of microtremor survey than CSW method due to array size. It is established in this chapter that the larger the array size the deeper the depth of investigation in microtremor survey. The microtremor survey data obtained from SPAC method of analyses were compared with CSW survey data in terms of shear wave velocity profile and also with borehole log. The comparison showed good match even though there was a difference in the dispersion curves. Good comparison of shear wave velocity profiles with borehole log implies that CSW and microtremor survey data were fundamental mode Rayleigh wave data as only fundamental mode Rayleigh wave data can provide a good match with borehole log data (Zhang et al., 2003).

CHAPTER 6

ULTRASONIC VELOCITY TEST AND TRIAXIAL TEST

6.1 Introduction

Shear stiffness at very small-strain level has become an important and useful parameter to geotechnical engineers (Barański, 2003). Knowing the velocity of seismic waves enables the stiffness of the soil to be determined. However, the stiffness of the soil depends on the properties of the medium, e.g. soil type, particle size distribution, soil structure, confining pressure and moisture content of soil (Inci et al., 2003; Prasad et al., 1999; Schneider et al., 1999). The coexistence of solid and fluid phases adds significant complexity to the behaviour of soil. Santamarina (2001) showed that V_P and V_S decrease with clay content and increase with confining pressure. The compression wave velocity in saturated soil is commonly measured to be the speed of compression wave in water. Unsaturated soil could show higher shear modulus of 1.5 to 2 times of that of fully saturated soil or fully dry soil (Gazetas, 1991). Prasad et al. (1999) indicated that confining pressure also plays an important role in V_P and V_S determination of soil. Therefore, it is very important to investigate the effect of degree of saturation as well as confining pressure on seismic wave velocities of soil from which stiffness is determined. However, very little research has been done on unsaturated cohesive soil (Inci et al., 2003). In this research, Attempt has been made to investigate the effects of different parameters (degree of saturation, confining pressure and soil structure) on V_P and V_S of unsaturated soils.

The stiffness determined from seismic velocity is the small-strain stiffness. Atkinson (2000), Burland (1989) and Tatsuoka et al. (1994) have highlighted the importance of measuring stiffness at various strain levels. In this study, triaxial compression tests with local strain measurements were also performed to obtain the stiffness-strain relationship of unsaturated soils. The small-strain stiffness determined from the seismic velocity is used together with the stiffness-strain relationship obtained from triaxial compression test to produce the complete stiffness-strain relationship of the soil. This chapter discusses the ultrasonic velocity test system, the triaxial apparatus with local measurement followed by a description of soils and test procedure.

6.2 Ultrasonic Velocity Test System

The ultrasonic velocity test system developed by GCTS (2001) was used in this research. The schematic diagram of this system is shown in Figure 6.1. The ultrasonic velocity test system is incorporated into a triaxial apparatus. The ultrasonic test system consists of 70 mm diameter platens to contain the transducers, pulser 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. The main 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, e.g. Matlab, thereby enabling a better understanding and interpretation of the signal.

The compression (P) and shear (S) wave transducers consisting of ceramic piezoelectric crystals are bonded to the inside face of the stainless steel platen. Both the P- and S- wave transducers are compactly mounted inside the loading cap and pedestal of a triaxial testing apparatus. The arrangement of the P- and S- wave transducers within the platens is shown schematically in Figure 6.2. 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 research, the sampling rate was fixed at 2 MHz. Control of the data acquisition system and its internal P- and S-wave switching circuit is performed through a software program. The P-wave transducer generates compressional pulse motion while the S-wave transducer creates torsional shear motion. The ends of the transducer are fixed in order to transfer most of the applied energy in the direction of the specimen. In the test, the pulse is sent from the transducers at one end of the specimen and received by the transducer at the other end of the specimen.

Figure 6.1 Ultrasonic velocity test system (from Leong et al., 2004)

Figure 6.2 Arrangement of compression and shear wave transducers

The signal strength could vary but it will always show a similar frequency spectrum. Therefore, if the signal properties are known, the experimental signal can be checked. The natural frequencies of the piezoelectric crystals of the P- and S-wave transducers are between 200 to 1000 kHz. However, the natural frequency of the piezoelectric crystal within the platens depends on the manufacturing process and their bonding to the platen. The operating frequencies of the P- and S-wave transducers vary from system to system. The operating frequencies of P- and S-wave transducers of the ultrasonic velocity test system were determined by placing the two platens in face-to-face contact with each other. The frequency spectra of the P- and S-wave signals from the face-to-face test are shown in Figure 6.3 and Figure 6.4, respectively. The

frequency spectrum of the P-wave signal shows a single peak at 90 kHz whereas the frequency spectra of the S-wave signal shows three peaks at 76, 78 and 90 kHz. Same phenomena were observed by Leong et al. (2004), who used the same ultrasonic velocity test system. The peaks at 76 and 78 kHz are higher than that at 90 kHz. The double peaks at 76 and 78 kHz could be due to bonding procedure of the piezoelectric crystals by the manufacturer. Even though, different peaks have been found in the signal, it will not affect the result. These frequency spectra will only demonstrate the reliability of the signal during the experiment.

Figure 6.3 Frequency spectra of compression waves (face-to-face)

One of the limitations of this system was mentioned in the GCTS (2001) user manual as a caution is that the signal can be weaker when the contact between transducer and test specimen is poor. It was also mentioned that a face-to-face platen correction on the travel time might be needed. P- and S-wave transducers are placed inside the platens. There is a “gap” between the transducers and soil specimen. This gap will cause a delay for the signal to reach the soil specimen. A correction for this face delay is needed. The face delay can be measured in two ways: (a) using standard test specimens such as aluminium, mild steel, nylon of different lengths, or (b) by placing the platens in face-to-face contact called face-to-face test.

Figure 6.4 Frequency spectra of shear waves (face-to-face)

In the first method, the variation of the travel time is determined for a specimen with different lengths and then plotted against length of the specimen. The plotted line without face delay should be straight and passing through the origin of the plot. However, due to face delay, there will be a y-intercept. This y-intercept is the face delay of the transducer. Three kinds of specimen, aluminum, mild steel and nylon, have been used in this research. The variations of travel time for different length of specimens are shown in Figure 6.5, Figure 6.6 and Figure 6.7, and the obtained face delays are summarized in Table 6.1. It is seen from Table 6.1 that there is a difference in the face delays for different materials. The difference is attributed to the slight difference in contact condition between the test specimen and platens. However, its effect is small with respect to the travel time, and V_P and V_S are hardly affected by this difference. Average values, $4.79 \mu\text{s}$ for S-wave and $2.91 \mu\text{s}$ for P-wave, can be used as the face delays.

Table 6.1 Face delays for P- and S-waves

Specimen	Face delay (μs)	
	S-wave	P-wave
Aluminium	4.78	2.86
Mild steel	4.75	2.90
Nylon	4.84	2.97
Average	4.79	2.91

Figure 6.5 Variation of travel time with length of aluminium

Figure 6.6 Variation of travel time with length of mild steel

Figure 6.7 Variation of travel time with length of nylon

In the face-to-face method, P and S-wave signals from the receiver are plotted against time. If there is no face delay, the plotted data will start from zero time. The time lag

of the receiver signal gives the face delay. The receiver signals from the face-to-face test are shown in Figure 6.8 and Figure 6.9. It can be seen from Figure 6.8 and Figure 6.9 that the face delay is $2.9 \mu\text{s}$ for P-wave and $4.8 \mu\text{s}$ for the S-wave which is almost same as determined from the first method. Therefore in this study, face delays of $2.9 \mu\text{s}$ for P-wave velocity and $4.8 \mu\text{s}$ for the S-wave velocity were used.

Figure 6.8 Receiver signal for P-wave (face-to-face).

Figure 6.9 Receiver signal for S-wave (face-to-face).

Aluminum, mild steel and nylon have standard V_P and V_S . Therefore, comparison of measured V_P and V_S with the values given in the literature will give a validation of the

result of the ultrasonic velocity test system. The results are summarized and compared with the data obtained from literature in Table 6.2 and Table 6.3 for V_P and V_S , respectively.

Table 6.2 Comparison of V_P

Materials	Length of the specimen (mm)	V_P (m/s)		
		Calculated	Average	Standard
Aluminium	200	6116	6125±0.16%	6186 (Yeo, 2005) 6250-6350 (Yoseph and Ajit, 1989)
	150	6122		
	50	6135		
Mild steel (Grade 43)	200	4651	4653±0.43%	4551 (Yeo, 2005) 3500-6140 (Yoseph and Ajit, 1989)
	150	4658		
	100	4673		
	50	4630		
Nylon (Nylon 6)	200	2590	2589±0.96%	2646 (Yeo, 2005) 1800-2200 (Yoseph and Ajit, 1989)
	135	2596		
	100	2564		
	50	2604		

Table 6.3 Comparison of V_S

Materials	Length of the specimen (mm)	V_S (m/s)		
		Calculated	Average	Standard
Aluminium	200	3105	3108±0.12%	3160 (Yeo, 2005) 3100 (Yoseph and Ajit, 1989)
	150	3112		
	50	3105		
Mild steel (Grade 43)	200	2692	2692±2.0%	2755 (Yeo, 2005) 2200-3260 (Yoseph and Ajit, 1989)
	150	2683		
	100	2747		
	50	2645		
Nylon (Nylon 6)	200	1132	1133±0.88%	1141 (Yeo, 2005) 1120 (Yoseph and Ajit, 1989)
	135	1135		
	100	1123		
	50	1139		

It can be seen from Table 6.2 and Table 6.3 that V_P and V_S for the standard materials used in this research are very close to the values in the literature. The discrepancies in the calculated velocities for different lengths of the same material are generally less than 1% except for V_S of mild steel which is about 2%. Molyneux, and Schmittz (1999) found travel times can differ by more than 4% when testing rock material. Therefore, the discrepancies measured using the ultrasonic velocity test system are within testing norm and could be attributed to the picking of the arrival time.

6.3 Acoustic Coupling

To improve signal quality, an acoustic couplant can be used to improve P- and S-wave signal amplitude (GCTS, 2001). The acoustic couplant reduces air voids or irregularities between platen and specimen and improve wave transmission (Couvreur and Thimus, 1996). Different materials have been used in previous researches as an acoustic couplant in ultrasonic test: glycerine (Kim et al., 2001), grease (Rao and Ramana, 1993; Li and Nordlund, 1993), honey (Siggins 1993, Guillaume et al., 1994), fibreglass jelly (Leong et al., 2004), water (Kim et al., 2001; Li and Nordlund, 1993).

Water could not be used in this research as a couplant in this research since adding of water will change the degree of saturation of unsaturated soil specimen. Moreover, water is not a suitable couplant for V_s measurement in ultrasonic velocity test (Couvreur and Thimus, 1996). Kim et al. (2001) found that the amplitude of the first echo measured by using pure glycerine is about three times higher than that using pure water in ultrasonic velocity test. Fibreglass jelly was successfully used by Leong et al. (2004) with the same ultrasonic velocity test system. In this research, fibreglass jelly was used as the acoustic couplant. The thickness of the acoustic couplant must be controlled otherwise it will increase the travel time and cause error in velocity determination. Kim et al. (2001) found the thinner the layer of acoustic couplant, the stronger is the intensity of a transmitted wave and suggested that thickness of an acoustic couplant layer should be much thinner than the wavelength of the transmitted wave.

The differences in the P- and S-wave signals can be easily observed from the face-to-face tests with and without acoustic couplant in the frequency domain as shown in Figure 6.10 and Figure 6.11, respectively. It can be seen from the plots that the receiver signals with acoustic couplant is stronger and clearer than the receiver signals without acoustic couplant.

Figure 6.10 Frequency spectrum for compression wave with and without couplant (face-to-face)

Figure 6.11 Frequency spectrum for shear wave with and without couplant (face-to-face)

6.4 Test Programme for Compacted Soil Samples

An investigation on effect of soil type and degree of saturation on V_P and V_S was conducted. Two soils were used: commercial kaolin and sand. The reason for using kaolin and sand is because these materials can be easily reconstituted into soil specimens of different density and degree of saturation. The reconstituted soil specimens are homogeneous and enable a systematic investigation of the effect of degree of saturation on V_P and V_S . The grain size distributions of kaolin and sand are shown in Figure 6.12.

Figure 6.12 Grain size distributions of sand and kaolin

In order to investigate the effect of soil types, three different mixtures of kaolin and sand have been used: kaolin (K100), 80% kaolin and 20% sand (K80S20) and 60% kaolin and 40% sand (K60S40). Each of the soil types was compacted at different water content in order to investigate the effect of degree of saturation on the seismic velocity of the soil. The physical properties of the compacted samples are summarised in Table 6.4.

Table 6.4 Physical properties of compacted soil samples

Soil properties	K100	K80S20	K60S40
Liquid limit	62	56	50
Plastic limit	41	37	35
Plasticity index	21	19	15
Specific gravity	2.59	2.62	2.64
USCS	MH	MH	MH

As the ultrasonic platen diameter is 70 mm, the soil specimen height should 140 mm to maintain height to diameter ratio of 2. The standard Proctor compactor machine was used to prepare the soil samples. The standard Proctor mould's height and diameter are 101 mm. As the specimen's height needs to be 140 mm to maintain height to diameter ratio of 2, the standard Proctor's mould height was extended by 50 mm. In the standard Proctor test, soil is compacted in three layers in the mould by a 2.495 kg hammer. The compaction energy is fixed for each layer, i.e. 25 blows per layer with a height of fall of 304.88 mm. Since the compacted sample size was 150

mm in height and 101 mm in diameter, the compaction energy was adjusted to standard Proctor compaction energy (ASTM, 1997) by increasing the number of layers to 5 where the number of blows per layer remains at 25. The compaction curves of K100, K80S20 and K60S40 are shown in Figure 6.13. The measured water content, degree of saturation and void ratio of the compacted soil specimens are summarized in Table 6.5.

Table 6.5 Conditions of compacted soil specimens for ultrasonic test

Soil type	Measured water content, w (%)	Degree of saturation, (%)	Void ratio, e	Confining pressure
K100	20	44.6	1.18	V_p and V_s were measured at 0, 50, 100 and 200 kPa for each sample
	23	53.2	1.12	
	27	63.1	1.11	
	29	67.5	1.13	
	33	69.5	1.23	
K80S20	19	43.3	1.19	
	22	54.2	1.07	
	25	63.8	1.05	
	28	68.0	1.09	
	30	69.6	1.16	
K60S40	19	43.2	1.16	
	23	59.8	1.02	
	24	65.9	1.00	
	29	70.0	1.09	
	32	71.6	1.18

Figure 6.13 Compaction curves of soil samples

6.5 Triaxial Apparatus with local Strain Measurement

The ultrasonic velocity test system is incorporated into a triaxial apparatus. The schematic diagram of this system is shown in Figure 6.1. The triaxial apparatus consists of:

- (a) Load cell: A submersible load cell was used to measure axial load on the soil specimen. The load cell used was manufactured by Applied Measurements, UK, model: STALC3-5kN. The capacity of the load cell is 5 kN.
- (b) Pore-water pressure transducer: A pore-water pressure transducer was used to measure the excess pore water pressure during triaxial shearing test. The pore-water pressure transducer used was manufactured by Druck, model: PDCR-810. The capacity of the pore-water pressure transducer is 500 kPa.
- (c) Linear Voltage Differential Transducer (LVDT): A LVDT was used to measure the axial deformation. The LVDT was manufactured by MPE transducers, model: LSC type HS 2331-50. The capacity of the LVDT is 50 mm.
- (d) Local Deformation Transducers (LDT): a pair of LDTs was used to measure local axial deformation of the specimen during triaxial test. The LDTs were fabricated in-house based on Goto et al. (1991). TML strain gauge, model FTA-2-350-11 were used to prepare the LDTs, which was placed on phosphor bronze plate with a dimension of 100 mm x 5 mm x 0.50 mm. The capacity of the LDT is 1 mm. Two LDTs were placed diametrically opposite to each other and were fixed onto the specimen using hinges superglued onto the membrane. The reason behind the use of a pair of LDTs is to obtain an average value of axial displacement. To avoid membrane penetration due to application of the confining pressure, gypsum was applied onto the surface of the soil specimen at the spots where the hinges were located. A thin nylon sting was connected to the LDT to enable the LDT to be dislodged from the hinge when axial strain reached the maximum range of the LDT i.e., 1 mm.

- (e) Proximity transducer: a pair of proximity transducers from Applied electronics, model: 55 MS-M and sensor model: PU-09 was used to measure local radial deformation of the specimen during triaxial test. The capacity of the proximity transducer is 4 mm. It is non-contacting and the transducers are placed diametrically opposite to each other in water-protected holder positioned at the middle of the soil specimen. An aluminum foil was pasted onto the rubber membrane to serve as the target. Care should be taken to ensure that the target material is firmly attached to the rubber membrane; as any movement of the target material will affect the readings.

The cell pressure was supplied by a 2 MPa/1000 cc GDS digital pressure/volume controller. The consolidated undrained triaxial test was done according to ASTM D 4767-95 (1996). All the transducers were connected to a high-speed data acquisition system consisting of the ultrasonic interface unit and a personal computer. The calibration curves for all the transducers used in the triaxial apparatus are given in Appendix C.

6.6 Test Programme for Undisturbed Soil Samples

In this research, undisturbed soil samples were collected from five out of the seven test sites (Site 3 to Site 7). Soil samples from Site 1 and Site 2 were not available as no soil sampling was performed at these sites. Ultrasonic velocity test followed by the Consolidated Undrained (CU) triaxial test was conducted. CU triaxial test were performed to determine the variation of shear modulus with strain and to correlate shear modulus with soil properties.

The specimens were labelled according to its site number and its depth as SXDY where 'S' stands for 'Site', 'X' the site number, 'D' stands for 'Depth', and the depth 'Y' is in metre. The basic properties of the soil samples are summarized in Table 6.6 and the different conditions of CU test specimens are summarized in Table 6.7. The borehole logs of the sites are given in Appendix A.

Table 6.6 Properties of undisturbed soil sample

Specimen	w (%)	G _s	ρ (Mg/m ³)	e	LL (%)	PL (%)	PI	Grain size distribution (%)			USCS
								Gravel	Sand	Clay	
Site 3											
S3D10.15	18	2.65	2.1	0.49	34	22	12	0	32	68	CL
S3D16.6	20	2.68	2.1	0.53	37	23	14	2	31	67	CL
Site 4											
S4D12.3	22	2.58	1.99	0.58	35	24	21	2	54	44	CL
S4D21.2	22	2.61	2	0.59	37	28	19	0	78	22	CL
S4D22.5	22	2.61	2	0.59	37	28	19	0	78	22	CL
Site 5											
S5D8.9	41	2.63	1.79	1.07	57	35	22	0	10	90	ML
D5D14.9	32	2.64	1.85	0.88	43	26	17	1	46	53	CL
S5D17.85	30	2.57	1.89	0.77	42	26	18	1	45	54	CL
S5D20.85	31	2.57	1.89	0.78	43	27	16	1	43	56	ML
Site 6											
S6D12.35	35	2.63	1.85	0.92	50	26	24	0	55	45	CH/CL
Site 7											
S7D12.5	55	2.3	1.59	1.24	72	30	42	1	0	99	CH
S7D21.5	58	2.75	1.67	1.6	68	27	41	1	2	97	CH

Table 6.7 Initial conditions of triaxial test

Specimen	σ_3 (kPa)	B
Site 3		
S3D10.15	125	0.94
S3D16.6	200	0.97
Site 4		
S4D12.3	120	0.97
S4D21.2	240	0.97
S4D22.5	250	0.96
Site 5		
S5D8.9	70	0.94
D5D14.9	130	0.97
S5D17.85	160	0.95
S5D20.85	185	0.96
Site 6		
S6D12.35	110	0.95
Site 7		
S7D12.5	75	0.96
S7D21.5	120	0.96

6.7 Summary

The ultrasonic velocity test system developed by GCTS (2001) and the triaxial apparatus were introduced in this chapter. The verification of the ultrasonic velocity test system is done by using standard materials: aluminium, mild steel and nylon. Two types of soils, compacted and undisturbed, have been used in this research. The properties as well as test conditions of the experiments are also described in this chapter. All the test results are analysed and discussed in the following chapter.

CHAPTER 7

ANALYSIS OF LABORATORY TEST RESULTS

7.1 Introduction

This chapter contains three parts:

- (1) Results of ultrasonic velocity tests on compacted kaolin and kaolin-sand mixtures are presented to provide a better understanding of the effects of confining pressure σ_3 , degree of saturation S , and soil type on V_P and V_S . The relationship of V_P with V_S and S is investigated;
- (2) Results of ultrasonic velocity tests on undisturbed soil samples from five sites are presented together with their stiffness-strain relationships from triaxial test with local strain measurement; and
- (3) Shear wave velocities V_S , from ultrasonic velocity test are compared with V_S from CSW and microtremor tests. In addition empirical equations for V_S are evaluated.

7.2 Effect of Confining Pressure, Degree of Saturation and Soil Type on V_P and V_S

Wave velocities, V_P and V_S , of soil increases with increase in confining pressure (Hardin and Richart, 1963; Zimmer et al., 2002). In this study, confining pressures of 0, 50, 100 and 200 kPa were applied to compacted soil specimens of kaolin and kaolin-sand mixtures. Wave velocities, V_P and V_S were determined using ultrasonic velocity test and are plotted against degree of saturation in Figure 7.1 and Figure 7.2, respectively. The degree of saturation of the compacted soils range from 43% to 70%.

(a) K100

(b) K80S20

(c) K60S40

Figure 7.1 Effect of degree of saturation on V_p

(a) K100

(b) K80S20

(c) K60S40

Figure 7.2 Effect of degree of saturation on V_s

Compression wave velocity V_P in saturated soil is commonly measured as the speed of compression wave in water (Nakagawa et al., 1996) which is about 1500 m/s

(Lorenz, 2002). Bourbie' et al. (1987) showed that the presence of moisture modifies the body wave behaviour through the medium. As expected, Figure 7.1 shows V_P increasing with degree of saturation for each soil type. In contrast, Figure 7.2 shows V_S increasing with degree of saturation up to the optimum moisture content and decreases beyond the optimum moisture content. Prasad et al. (2004) found that at very low confining pressures, V_S is strongly dependent on composition and microstructure. The shear modulus of the skeleton is a complex function that depends on the conditions of inter-particle forces and local particle contact interactions (Santamarina et al., 2001). This implies that V_S is more heavily dependent on void ratio. By plotting V_P/V_S versus degree of saturation S as shown in Figure 7.3 the effect of void ratio on V_P is eliminated from Figure 7.1. Figure 7.3 shows that V_P is relatively constant between 42% to 63% and increases with S rapidly after 63 %. In the following section, dependency of wave velocities on different soil parameters is investigated.

7.2.1 Effect of Void Ratio and Confining Pressure on V_P and V_S

Foti et al. (2002) showed that there is a relationship between void ratio in fluid-saturated porous media and V_S . The lower the void ratio the higher the V_S . There is a correlation between G_{max} to the void ratio, e , and the effective confining pressure, σ_3' (Hardin and Richart, 1963; Hardin and Drnevich, 1972; Kramer, 1996). In this section, an attempt is made to determine the relationship between V_P and V_S with σ_3' . If it is assumed that at $\sigma_3' = 0$, V_{S0} , will only be a function of e , it can be then expressed as,

$$V_{S0} = f'(e) \quad (7.1)$$

where, $f'(e)$ is a function of e . If $\sigma_3' > 0$, V_S can be expressed as,

$$V_s = f'(e) + f(e)\left(\frac{\sigma_3'}{P_a}\right)^\beta \quad (7.2)$$

where $f(e)$ is a function of e , P_a is the atmospheric pressure (~ 100 kPa), used to normalized the confining pressure and β is a constant.

(a) K100

(b) K80S20

(c) K60S40

Figure 7.3 V_p/V_s with degree of saturation

If it is further assumed that $f(e) = Af'(e)$ where A is a constant, Equation 7.2 can be generalized for both V_P and V_S as follows:

$$V = V_0[1 + A(\frac{\sigma_3}{P_a})^\beta] \quad (7.3)$$

which is similar to Equation 8.1 in Santamarina et al. (2001). Using the test data and Microsoft Excel solver, it was found that the value of β is constant for both P- and S-waves but it varies with soil type and the value of A is constant irrespective of soil type but varies with wave type. The values of β and A are summarized in Table 7.1.

Table 7.1 Constants n and A

Soil type	β	A	
		P-wave	S-wave
K100	0.58	0.05	0.10
K80S20	0.74		
K60S40	0.78		

Comparing Equation 7.2 and Equation 7.3,

$$V^* = \frac{V}{(1 + A(\frac{\sigma_3}{P_a})^\beta)} \quad (7.4)$$

where, V^* will just be a function of void ratio. Equation (7.4) can be rewritten as follows: for P-wave:

$$V_P^* = \frac{V_P}{(1 + 0.05(\frac{\sigma_3}{P_a})^\beta)} \quad (7.5)$$

for S-wave:

$$V_S^* = \frac{V_S}{(1 + 0.1(\frac{\sigma_3}{P_a})^\beta)} \quad (7.6)$$

V_P^* and V_S^* are plotted against degree of saturation as shown in Figure 7.4 and Figure 7.5, respectively. It is seen from these plots that all the curves collapsed into a narrow band. Hence, knowing wave velocity at zero effective confining pressure enables wave velocities at other effective confining pressures to be estimated from Equation 7.5 and Equation 7.6.

(a) K100

(b) K80S20

(c) K60S40

Figure 7.4 V_p^* with degree of saturation

V_s^* in Figure 7.5 has the same shape as Figure 7.2. In order to ascertain that V_s is dependent on soil structure, the V_s^* values in Figure 7.5 are plotted against void ratio

in Figure 7.6. Figure 7.6 shows that there is a unique relationship between V_s and e , i.e. V_s is affected by e .

(a) K100

(b) K80S20

(c) K60S40

Figure 7.5 V_s^* with degree of saturation

(a) K100

(b) K80S20

(c) K60S40

Figure 7.6 V_s^* with void ratio

7.2.2 Relationship of V_p with V_s and Degree of Saturation

It was shown in the earlier section that V_p depends mostly on the degree of saturation of soils. In this section the effect of degree of saturation will be quantified. From elastic theory, the compression wave velocity of a medium, V_p , is given by

$$V_p = \sqrt{\frac{K + \frac{4}{3}G_{\max}}{\rho}} \quad (7.7)$$

where, K is bulk modulus of the medium and can be expressed by,

$$K = \frac{2G_{\max}(1+\nu)}{3(1-2\nu)} \quad (7.8)$$

where $G_{\max}$ is the maximum shear modulus of the medium and ν is the Poisson's ratio. Equation 7.7 considers the medium to consist of single phase material. However, soil is a three phase material, i.e. soil skeleton, water and air, where degree of saturation plays an important role in determining V_p . Compression wave velocity V_p has been used to ascertain S of near-saturated samples (Tsukamoto et al., 2002). Seismic velocities depend not only on the degree of saturation but also on the distribution of the fluid phase within the pore space (Mavko, and Nolen-Hoeksema, 1994). Water phase of soil is not continuous when S is less than 80% and air phase is occluded when S is between 80% to 90% (Fredlund and Rahardjo, 1993). That means degree of saturation plays a more important role in V_p when S is greater than 80%. To determine V_p , bulk modulus of soil, K_s , and bulk modulus of fluid, K_f (combination of bulk modulus of water, K_w , and air, K_a) should be included in Equation 7.7, i.e.,

$$V_p = \sqrt{\frac{K_s + K_f + \frac{4}{3}G_{\max}}{\rho}} \quad (7.9)$$

Wave propagation in unsaturated soil can be studied by considering the variation of the bulk modulus of pore water K_w (Mitsuji et al., 2000). In a saturated soil V_p is primarily controlled by K_w (Santamarina, 2001). The value of K_f changes drastically even with a small increase in air content as shown in Figure 7.7. Therefore, it is

essential to estimate K_f accurately in order to estimate V_P . The complexity in calculating K_f arises because soil consists of three phases: solid, liquid and gas. Attempts have been made to determine K_f (Schuurman, 1966; Akagawa, 1974, Yeo, 2005).

Figure 7.7 Bulk modulus of a fluid mixture of air ($K_a = 142$ kPa) and Water ($K_w = 2.18$ GPa) (from Santamarina, 2001)

Schuurman (1966) proposed the following equation for K_f using Boyle's law and Henry's law:

$$K_f = (V_a + V_w) \left\{ \frac{(V_{ao} + V_{as})P_{a0}}{(V_a + V_{as})^2} - \frac{4T_i}{3d_0V_a} \left(\frac{V_{a0}}{V_a} \right)^{1/3} \right\} \quad (7.10)$$

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, T_i is the surface tension, d_0 is the diameter of the pore air V_{as} is the volume of air dissolved in water and H_s is the coefficient of solubility ($= 0.02$ at S.T.P). Schuurman (1966) assumed that all the air bubbles have the same diameter which is practically unreasonable for a soil.

Akagawa (1974) proposed an equation for K_f :

$$K_f^* = \frac{\rho_f V_a^2}{(1-S)^2 + (1-S)S \frac{\rho_w}{\rho_a} + \frac{V_a^2}{V_w^2} S \left(1 - (1-S) \left(1 - \frac{\rho_a}{\rho_w} \right) \right)} \quad (7.11)$$

where, S is the degree of saturation, v_a is the compression wave velocity of air, v_w is the compression wave velocity of water, ρ_a is the density of air and ρ_w density of water. Akagawa (1974) assumed that momentum of water and air is conserved at the wavefront of P-wave and change in unit weight of soil due to pressure is isotropic. He assumed that all the air bubbles have the same diameter and does not dissolve due to P-wave propagation.

Tamura (2002) used K_f^* proposed by Akagawa (1974) (Equation 7.11) to determine V_P as follows:

$$V_P = \sqrt{\frac{K_s + K_f^* + \frac{4}{3}G_{max}}{\rho}} \quad (7.12)$$

Yeo (2005) reviewed the Reuss, Vigot and occluded phase models to account for the effects of S on V_P . Ruess model assumed that soil phases are in series connection, i.e. strain in soil mass is equal to the strain of soil skeleton, air and water phases of the soil. Vigot model assumed that soil phases are in parallel connection, i.e. strain in soil skeleton is equal to strain in water phase which is also equal to strain in air phase. Occluded air phase assumed that strain in soil skeleton is equal to strain in water phase and air phase. Yeo (2005) found that these models cannot estimate V_P accurately. The K_f equations proposed by Schuurman (1966) and Akagawa (1974) were also investigated but could not fully account for the relationship between S and V_P . Using V_P for S from 70% to 90%, Equation 7.12 could not estimate V_P accurately. As Akagawa (1974) assumed spherical and uniform shape of air bubble, Yeo (2005) assumed size and shape of air bubble could be the cause of discrepancy and proposed a constant C_S to be included in Equation 7.12 to account for the discrepancy as follows:

$$K_f^{**} = \frac{\rho_f v_a^2}{C_S \left((1-S)^2 + (1-S)S \frac{\rho_w}{\rho_a} + \frac{v_a^2}{v_w^2} S \left(1 - (1-S) \left(1 - \frac{\rho_a}{\rho_w} \right) \right) \right)} \quad (7.13)$$

Using experimental data from different researchers, Yeo (2005) obtained a relationship of C_s with S as shown in Figure 7.8. The relationship of C_s with S (in percent), is given by:

$$\log C_s = -1.6(100 - S)^{0.29} e^{-0.006(100 - S)^{1.29}} \quad (7.14)$$

Figure 7.8 C_s versus S plot (from Yeo, 2005)

The fitted curve seems to be a reasonable fit but there was insufficient data when S is lower than 55%. Further improvement is necessary in this model.

Wyllie et al. (1956) investigated V_p for numerous synthetic and natural porous media, saturated with different kinds of fluid (e.g. water, brine, organic liquids and plastics) to determine the effect of saturation fluid on V_p . For water, the effect of degree of saturation on V_p is shown in Figure 7.9. Wyllie et al. (1956) found that higher V_p occurred at low S ($< 20\%$) and at high S ($> 80\%$) and is almost constant in between.

Figure 7.9 Variation for V_p with S (from Wyllie et al., 1956)

Lorenz (2002) showed that speed of compression wave of a two-phase mixture is dependent on void fraction (gas volume fraction) as shown in Figure 7.10. Water steam is shown as a solid line; methane-rich Titan fluids are shown as a dashed line. It can be seen from Figure 7.10 that higher V_P occurred at high gas volume fraction ($S < 20\%$) and at low gas volume S ($S > 80\%$) and is almost constant in between.

In order to examine the liquefaction resistance of partially saturated sand, Tsukamoto et al. (2002) measured V_P and V_S in a cyclic triaxial test apparatus. They found that V_P increases drastically when S increases from 90% to 100%. Domenico, (1976) suggested that the transition to the high V_P values starts at $S = 85\%$. However, Velea, et al. (2000) did not observe this phenomenon even at $S = 98\%$, therefore, they suggested that air presence have a large effect for almost fully water-saturated sand.

Figure 7.10 Variation for sound speed with gas volume fraction (from Lorenz, 2002)
(Solid line for Water steam and dashed line for methane-rich Titan fluids)

Tittmann et al. (1980) found that in the intermediate saturation range ($\sim 2\text{--}98\%$) V_P depends on the distribution of liquid–gas mixture, pore geometry and size and at the very low saturation end ($\sim 2\%$), small amount of gases affect V_P drastically.

From the above discussion, it can be observed there are two transition points when V_P is related to S (at $\sim 10\%$ and $\sim 85\%$). Therefore, the relationship between C_S and S as

proposed by Yeo (2005) seems to be unreasonable for the whole range of S (0% to 100%). One of the difficulties faced by Yeo (2005) was the lack of data for S less than 55%.

In order to develop a complete relationship between C_s and S , unsaturated compacted samples were used. The density ρ , degree of saturation S , Poisson's ratio ν , V_P and V_S of the specimens are summarized in Table 7.2. All the V_P and V_S were measured at zero confining pressure.

Table 7.2 Ultrasonic test results and soil properties

Specimen	ρ kg/m ³	S (%)	ν (assumed)	V_P (m/s)	V_S (m/s)
K100	1432.0	44.6	0.3	881	269
	1505.8	53.2	0.3	941	283
	1592.8	63.1	0.3	971	310
	1577.8	67.5	0.3	1000	282
	1547.7	69.5	0.3	1083	253
K80S20	1433.8	43.3	0.3	893	274
	1548.1	54.2	0.3	946	286
	1603.0	63.8	0.3	980	313
	1606.5	68.0	0.3	1008	279
	1587.6	69.6	0.3	1121	267
K60S40	1453.3	43.2	0.3	900	294
	1607.4	59.8	0.3	966	299
	1650.5	65.9	0.3	993	321
	1627.5	70.0	0.3	1101	298
	1599.1	71.6	0.3	1132	273

The Poisson's ratio ν has some impact on calculating the value of C_s . The value of C_s will increase with an increase in ν and vice versa. The intention is to develop an equation to estimate V_P given V_S and degree of saturation with the help of Equation 7.15. Therefore, Poisson's ratio is not known a priori. Hence, in order to be consistent, Poisson's ratio was assumed based on typical values. In this research, typical Poisson's ratio for different soil types as given in Table 7.3 was used.

The kaolin and kaolin-sand mixture are classified as silt and silty sand whose Poisson's ratio varies from 0.2 to 0.4. An average value of 0.3 was assumed when computing C_s .

Extensive study has been done on V_P of concrete (D'Angelo, 1995; Nagy, 1997; Rapoport et al., 2000; Popovics, 2001). It is easy to make a homogenous concrete specimen. The degree of saturation of the concrete specimen can be changed and the concrete specimen can be reused. In this study, a concrete specimen was used. The concrete specimen was first soaked in water to increase S . It is then dried slowly in the oven at 40^0 C to remove all the water inside the specimen. Fully dry condition was assumed when the oven dried concrete specimen shows the same weight over a period of time. During drying V_P and V_S of the concrete specimen were determined using the ultrasonic velocity test. Parameters, ρ , S , ν , V_P and V_S of the concrete specimen determined during drying are summarized in Table 7.4. All the tabulated data are at zero confining pressure.

Table 7.3 Representative values of Poisson's ratio (from Das, 1998, Bowles, 1997 and BS EN 1992-1-1:2004)

Reference	Type of Soil	Poisson's ratio, ν
Das (1998)	Loose sand	0.2-0.4
	Medium sand	0.25-0.4
	Dense sand	0.3-0.45
	Silty sand	0.2-0.4
	Soft clay	0.15-0.25
	Medium clay	0.2-0.5
Bowles (1997)	Clay, saturated	0.4-0.5
	Clay, unsaturated	0.1-0.3
	Sandy clay	0.2-0.3
	Silt	0.3-0.35
	Sand, gravely sand Commonly used	0.3-0.4
BS EN 1992-1-1:2004	Concrete	0.2

Values of C_s calculated from Equation 7.13 are shown in Figure 7.11. The results obtained are different from Yeo (2005). For further verifications, materials other than soil are used.

Table 7.4 Ultrasonic test results and properties concrete specimen

Specimen	ρ kg/m ³	S (%)	ν	V_p (m/s)	V_s (m/s)
Concrete	2305.5	96.40	0.2	4143	2493
	2293.9	91.61	0.2	4143	2489
	2270.9	82.10	0.2	4140	2500
	2222.5	62.37	0.2	4135	2499
	2144.1	31.15	0.2	4103	2498
	2121.0	22.15	0.2	4103	2498
	2095.7	12.34	0.2	4100	2500
	2079.5	6.15	0.2	4100	2507

The calculated values of C_s from V_p of concrete at different degrees of saturation are also plotted in Figure 7.11. It can be seen from Figure 7.11 that the concrete data also follow the same trend as the unsaturated compacted kaolin and kaolin-sand specimens.

Figure 7.11 C_s from experimental data and fitted curve

The following relationship was found to give a good fit to the data shown in Figure 7.11.

$$C_s = \frac{a}{(S^2 + b^2(1-S)(1-Sc))^{1.3}} \quad (7.15)$$

where, a , b and c are constants. The values of a , b and c are 0.2, 3.65 and 50, respectively. It can be seen from Figure 7.11 that the shape of the relationship is almost the same shape as Figure 7.9 and Figure 7.10. The shape of the curve implies that pore shape and size has a constant impact when S is from 10% to 90%. However, beyond the range, pore shape and size have a very significant effect on S . The developed relationship between C_s and S was based on the experimental results

obtained in this research. In the following section the developed relationship between C_S and S will be evaluated by using data from the literature.

7.2.3 Evaluation of Equation 7.15

Equation 7.15 was evaluated using data for different soils from the literature: Compacted residual soil from Jurong Formation (Yeo, 2005), compacted clayey soil (Inci, 2003), Kenya sand (Fioravante, 2000), Ticino sand (Fioravante, 2000), coarse monterey (Nakagawa et al., 1997), medium monterey (Nakagawa et al., 1997), fine crystal silica (Nakagawa et al., 1997) and Ottawa sand (Hardin and Richart, 1963). In the validation the value of C_S was first calculated using Equation 7.15 and then V_P was calculated using Equation 7.12 and 7.13. The obtained V_P was then compared with the measured V_P . A summary is given in Table 7.5. A plot between calculated V_P and measured V_P is shown in Figure 7.12. The maximum variation between calculated V_P and measured V_P was about $\pm 20\%$ illustrating that the C_S given by Equation 7.15 is reasonable.

Figure 7.12 Evaluation of Equation 7.15 for different soils

Table 7.5 Comparison of V_p (measured) and V_p (calculated)

Specimen	G_s	ρ (kg/m ³)	S (%)	V_s (m/s)	V_p (m/s) (measured)	V_p (m/s) (calculated)	ν (Assumed)	C_s
K100	2.59	1432.0	44.6	269	881	881	0.3	0.000342358338
		1505.8	53.2	283	941	941	0.3	0.000333412228
		1592.8	63.1	310	971	971	0.3	0.000400417781
		1577.8	67.5	282	1000	1000	0.3	0.000386004560
		1547.7	69.5	253	1083	1083	0.3	0.000318364004
K80S20	2.62	1433.8	43.3	274	893	893	0.3	0.000325781037
		1548.1	54.2	286	946	946	0.3	0.000328099304
		1603.0	63.8	313	980	980	0.3	0.000395046365
		1606.5	68.0	279	1008	1008	0.3	0.000371049055
		1587.6	69.6	267	1121	1121	0.3	0.000291365700
K60S40	2.64	1453.3	43.2	294	900	900	0.3	0.000338282654
		1607.4	59.8	299	966	966	0.3	0.000353377197
		1650.5	65.9	321	993	993	0.3	0.000402743505
		1627.5	70.0	298	1101	1101	0.3	0.000322344249
		1599.1	71.6	273	1132	1132	0.3	0.000305847259

Specimen	G_s	ρ (kg/m ³)	S (%)	V_s (m/s)	V_p (m/s) (measured)	V_p (m/s) (calculated)	ν (Assumed)	C_s
Concrete	2.7	2305.5	96.40	2493	4143	4143	0.2	0.002883639354
		2293.9	91.61	2489	4143	4143	0.2	0.001142480956
		2270.9	82.10	2500	4140	4140	0.2	0.000736787089
		2222.5	62.37	2499	4135	4135	0.2	0.000380712449
		2144.1	31.15	2498	4103	4103	0.2	0.000493093734
		2121.0	22.15	2498	4103	4103	0.2	0.000440800667
		2095.7	12.34	2500	4100	4100	0.2	0.000537923549
		2079.5	6.15	2507	4100	4100	0.2	0.001455343562
Soil 1 (Inci, 2003)	2.68	2200.00	10.18	858.50	1512.10	1518.42	0.25	0.000950905731
		2200.00	14.05	773.20	1404.40	1388.22	0.25	0.000418903718
		2200.00	28.40	700.00	1343.10	1326.75	0.25	0.000269276365
		2200.00	40.37	617.10	1192.90	1251.33	0.25	0.000384823929
		2200.00	46.20	568.30	1109.70	1206.01	0.25	0.000455817644
		2200.00	48.28	512.20	1037.80	1137.57	0.25	0.000429275194
		2200.00	3.48	929.30	1743.70	1620.19	0.25	0.000148339365
		2200.00	5.55	900.00	1695.90	1575.30	0.25	0.000152819505
		2200.00	41.91	624.40	1325.70	1268.73	0.25	0.000188538752
		2200.00	69.02	424.40	1278.90	1105.95	0.25	0.000189731358
		2200.00	10.97	1051.20	1841.60	1848.62	0.25	0.000946205200
		2200.00	12.37	985.40	1733.40	1740.53	0.25	0.000801805013
		2200.00	48.51	587.80	1234.70	1243.32	0.25	0.000256223211
		2200.00	83.48	585.40	1398.30	1323.35	0.25	0.000420226872
		2200.00	86.95	219.50	756.50	925.30	0.25	0.001152929914
		2200.00	6.52	1168.30	2113.80	2038.36	0.25	0.000184469790
		2200.00	7.10	1124.40	2025.60	1964.24	0.25	0.000223404235
		2200.00	62.93	892.70	2036.80	1920.92	0.25	0.000098798141
		2200.00	66.67	707.30	1937.10	1871.95	0.25	0.000085774150
		2200.00	67.27	607.30	1740.50	1741.09	0.25	0.000102289613
		2200.00	2.94	1058.50	1900.20	1841.52	0.25	0.000265883899
		2200.00	9.00	1019.50	1768.90	1789.22	0.25	0.006511181335
		2200.00	48.88	829.30	1507.90	1605.10	0.25	0.000598171082
		2200.00	62.40	661.00	1727.40	1395.43	0.25	0.000102321185
		2200.00	66.13	595.10	1591.60	1313.95	0.25	0.000129216076
		2200.00	66.02	439.00	1222.10	1114.20	0.25	0.000206945374
Soil 2 (Inci, 2003)	2.68	2500.00	92.39	281.50	866.40	904.98	0.25	0.001449481617
		2500.00	94.10	185.10	647.70	811.05	0.25	0.003028913133
		2500.00	86.79	544.00	1052.30	1230.77	0.25	0.001952246418
		2500.00	90.45	462.40	993.40	1115.40	0.25	0.001716643956
		2500.00	91.65	438.40	949.80	1080.24	0.25	0.002081712780
		2500.00	89.29	561.30	1263.80	1248.02	0.25	0.000810567927
		2500.00	96.16	306.20	950.70	889.42	0.25	0.002366262502
		2500.00	96.47	204.80	707.90	791.83	0.25	0.004269355948
Soil 3 (Inci, 2003)	2.68	2200.00	12.06	860.90	1717.20	1528.62	0.25	0.000100934993
		2200.00	26.60	707.80	1410.90	1331.56	0.25	0.000179848482
		2200.00	49.79	604.00	1261.40	1271.49	0.25	0.000258138243
		2200.00	89.13	402.50	1339.30	1087.75	0.25	0.000452712250
		2200.00	89.98	637.10	1184.60	1381.06	0.25	0.003458644659
		2200.00	94.07	343.50	961.90	992.61	0.25	0.001898658875
		2200.00	95.05	286.20	864.70	924.90	0.25	0.002585402201
		2200.00	68.05	346.30	634.90	638.20	0.25	0.004650539655
Ottawa sand (Hardin and Richart, 1963 and Allen et al., 1980)		1500	99.95	69.28	1020	1111.07	0.2	0.162753692767
		1500	99.8	55.37	850	838.76	0.2	0.064033350705
		1500	99.7	74.83	748	815.45	0.2	0.055893182593
		1500	99.7	74.83	680	939.86	0.2	0.068866307822
		1500	99.9	55.37	1088	838.76	0.2	0.075891190781

Specimen	G_s	ρ (kg/m ³)	S (%)	V_s (m/s)	V_p (m/s)		ν (Assumed)	C_s
Bukit timah Formation, (Yeo, 2005)		1910	88.10	445	932	1157.56	0.3	0.003547700293
		1950	88.52	374	840	1079.31	0.3	0.002927029639
		1980	90.40	482	1047	1175.17	0.3	0.002631672119
		2020	93.08	342	753	1009.61	0.3	0.006423964345
		2060	99.44	380	1318	920.98	0.3	0.009773346897
		2070	100.00	279	1323	2328.61	0.3	0.705096231313
		2110	100.00	320	1353	2335.81	0.3	0.704045126410
		2200	100.00	296	1416	2267.23	0.3	0.577293762437
Jurong Formation, (Yeo, 2005)		2100	55.23	474	925	1096.08	0.25	0.000829618905
		2190	71.93	461	944	1330.19	0.25	0.000908399403
		2190	74.75	549	1163	1473.22	0.25	0.000571314212
		2210	76.21	474	938	1332.81	0.25	0.001308644023
		2230	82.85	645	1281	1485.96	0.25	0.000942499214
		2010	85.93	460	943	1388.42	0.25	0.001967597413
		2030	87.63	537	1121	1332.81	0.25	0.001440436403
		2040	87.98	643	1294	1485.96	0.25	0.001329694172
		2090	89.57	585	1287	1388.42	0.25	0.001030919412
		2110	100.00	381	1359	1385.79	0.25	0.637016086924
		2150	100.00	505	1322	1400.83	0.25	0.897972745183
		2190	100.00	519	1345	1407.99	0.25	0.865424777859
		2200	100.00	591	1300	1309.93	0.25	0.874419393413
Fine crystal Silica (Nakagawa et al., 1997)		1462	0.00	146	314	326.20	0.35	0.015560076836
		1462	0.00	159	342	351.55	0.35	0.013069932789
		1462	0.00	183	385	398.94	0.35	0.031193629029
		1462	100.00	146	1705	1719.83	0.35	0.532293037465
		1462	100.00	159	1705	1703.78	0.35	0.535562301643
		1462	100.00	183	1709	1723.83	0.35	0.539790361097
		1462	100.00	217	1711	1708.93	0.35	0.550115238176
		1462	100.00	249	1713	1745.39	0.35	0.562037265372
		1462	100.00	289	1727	1723.12	0.35	0.571708366883
		1462	100.00	343	1744	1721.78	0.35	0.591779282215
Ticino sand (Fioravante, 2000)		1365	0.000	220	400	382.02	0.28	0.064857782051
		1365	0.000	300	550	556.40	0.28	0.013045679590
		1365	0.000	380	690	698.29	0.28	0.029496045584
Kaolin (Nakagawa et al., 1996)		1711	100.0	95	1513	2535.35	0.25	0.565930074140
		1711	100.0	113	1521	2537.57	0.25	0.562686428637
		1711	100.0	139	1516	2541.44	0.25	0.571437318749
		1711	100.0	177	1573	2548.51	0.25	0.537816425174
		1711	100.0	197	1616	2552.91	0.25	0.513095048247
		1711	100.0	207	1623	2555.29	0.25	0.510933996625
		1711	100.0	130	1520	2540.01	0.25	0.566529638948
		1711	100.0	113	1519	2537.57	0.25	0.564194164234
		1711	100.0	109	1520	2537.04	0.25	0.562779683778
		1711	100.0	108	1519	2536.91	0.25	0.563371100878
		1711	100.0	106	1518	2536.66	0.25	0.563806044321
		1711	100.0	105	1526	2536.54	0.25	0.557669658243
		1711	100.0	104	1525	2536.41	0.25	0.558259142287
		1711	100.0	104	1524	2536.41	0.25	0.559002389880
		1711	100.0	102	1527	2536.17	0.25	0.556477967836
		1711	100.0	101	1523	2536.05	0.25	0.559295942783
		1711	100.0	101	1522	2536.05	0.25	0.560040975506

7.3 Ultrasonic Velocity Test and Triaxial Tests on Undisturbed Soil Specimen

Undisturbed soil samples from 12 different depths at five of the seven sites in Singapore were collected to perform laboratory test. Undisturbed soil samples from

Site 1 and Site 2 were not available. The basic properties of the soil samples are shown in Table 7.6 and the borehole logs are given in Appendix A.

Table 7.6 Properties of Undisturbed soil sample

Specimen	w (%)	G _s	ρ (Mg/m ³)	e	LL (%)	PL (%)	PI (%)	Grain size distribution (%)			USCS
								Gravel	Sand	Clay	
Site 3											
S3D10.15	18	2.65	2.1	0.49	34	22	12	0	32	68	CL
S3D16.6	20	2.68	2.1	0.53	37	23	14	2	31	67	CL
Site 4											
S4D12.3	22	2.58	1.99	0.58	35	24	21	2	54	44	CL
S4D21.2	22	2.61	2	0.59	37	28	19	0	78	22	CL
S4D22.5	22	2.61	2	0.59	37	28	19	0	78	22	CL
Site 5											
S5D8.9	41	2.63	1.79	1.07	57	35	22	0	10	90	ML
S5D14.9	32	2.64	1.85	0.88	43	26	17	1	46	53	CL
S5D17.85	30	2.57	1.89	0.77	42	26	18	1	45	54	CL
S5D20.85	31	2.57	1.89	0.78	43	27	16	1	43	56	ML
Site 6											
S6D12.35	35	2.63	1.85	0.92	50	26	24	0	55	45	CH/CL
Site 7											
S7D12.5	55	2.3	1.59	1.24	72	30	42	1	0	99	CH
S7D21.5	58	2.75	1.67	1.6	68	27	41	1	2	97	CH

7.3.1 Ultrasonic Velocity Tests

Ultrasonic velocity test was performed to measure V_P and V_S of the soil specimens at different confining pressures: 0, 50, 100, 200 and 400 kPa. The maximum applied effective confining pressure was limited to the field overburden pressure of the soil specimen so that the soil specimen could be subsequently sheared in the triaxial test. Ultrasonic velocity test results for all the specimens were shown in Figure 7.13 to Figure 7.24. The results of each specimen are shown in two plots, (a) and (b). Plot (a) of each figure shows the P- and S-wave signals. The travel time for each wave is indicated by a dotted line. Plot (b) of each figure shows V_P and V_S variation with confining pressure. It can be seen that V_P increases with confining pressure but V_S does not increase significantly with confining pressure. The increase in V_P with confining pressure is due to increase in degree of saturation. Initially the soil specimen was not 100% saturated. But with the increase of confining pressure the soil specimen was slightly compressed and the volume of air reduces and hence degree of saturation of the soil specimen was increased. However, the change in soil structure of

the specimen is small and therefore, V_S did not vary significantly with confining pressure.

Figure 7.13 P- and S-waves for soil specimen S3D10.15

Figure 7.14 P- and S-wave for soil specimen S3D16.6

Figure 7.15 P- and S-waves for soil specimen S4D12.3

Figure 7.16 P- and S-waves for soil specimen S4D21.2

Figure 7.17 P- and S-waves for soil specimen S4D22.5

Figure 7.18 P- and S-waves for soil specimen S5D8.9

Figure 7.19 P- and S-waves for soil specimen S5D14.9

Figure 7.20 P- and S-waves for soil specimen S5D17.85

Figure 7.21 P- and S-waves for soil specimen S5D20.85

(a) P- and S-wave signals

(b) V_P and V_S with confining pressure

Figure 7.22 P- and S-waves for soil specimen S6D12.35

(a) P- and S-wave signals

(b) V_P and V_S with confining pressure

Figure 7.23 P- and S-waves for soil specimen S7D12.5

(a) P- and S-wave signals

(b) V_P and V_S with confining pressure

Figure 7.24 P- and S-waves for soil specimen S7D21.5

7.3.2 Triaxial Compression Test

The variation of G with strain is very important in geotechnical engineering. Triaxial Consolidated Undrained (CU) tests with local strain measurement were performed on undisturbed soil samples from five different sites. Data were presented in six plots (a to f) for each tested soil sample (e.g. Figure 7.25). Plot (a) shows the variation of deviator stress with axial and radial strain. The axial strain was measured by both LVDT and LDT shown in plot (b). Secant Poisson's ratio was determined from this plot. Plot (c) shows the LDT axial strain measurement in enlarged scale compare to plot (b). Plot (d) shows the variation of shear modulus with axial strain ε_a measured from LVDT and LDT. Shear modulus was computed using the following equation:

$$G = \frac{E}{2(1 + \nu)} \quad (7.16)$$

where E is the elastic modulus and ν is the Poisson's ratio. The shear strain γ can be obtained from the axial strain using relation $\gamma = \varepsilon_a / (1 + \nu)$. The very small-strain shear modulus obtained from ultrasonic velocity test and CSW survey are also shown in plot (d). Plot (e) shows total and effective stress paths and plot (f) shows the variation of pore-water pressure during undrained shearing. Initial and final conditions of test specimens are summarized in Table 7.7.

Table 7.7 Initial and final conditions of triaxial test specimens

Specimen	σ_3 (kPa)	B	$(\sigma_1 - \sigma_3)_f$ (kPa)	u_{wf} (kPa)	σ_{1f} (kPa)
Site 3					
S3D10.15	125	0.94	400	138	525
S3D16.6	200	0.97	330	148	530
Site 4					
S4D12.3	120	0.97	64	200	184
S4D21.2	240	0.97	172	105	412
S4D22.5	250	0.96	288	116	538
Site 5					
S5D8.9	70	0.94	138	58	208
S5D14.9	130	0.97	293	108	423
S5D17.85	160	0.95	459	242	619
S5D20.85	185	0.96	329	169	514
Site 6					
S6D12.35	110	0.95	315	85	425
Site 7					
S7D12.5	75	0.96	468	148	543
S7D21.5	120	0.96	373	180	493

The strain level in ultrasonic pulse transmission tests was determined in this research to obtain strain level at $G_{\max}$. The ultrasonic velocity system acquires signal due to the response of piezoelectric crystals in the platen because of the stress wave through the test specimen. In the determination of the strain level, the vibration on the face of the platen is measured with an accelerometer, and then the vibration and accelerometer signals are correlated. By associating the receiver signal with 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 can be obtained. The particle velocity is then divided by wave velocity to obtain strain level. The platen acceleration constants used in this research are 0.33 and 38 g/mV for the P and S waves, respectively. These values were obtained by Leong et al. (2004) where a 500g accelerometer (PCB series 353 quartz-shear ICP 353A03 accelerometer) with a sensitivity of 9.59 mV/g was used. Details on the determination of the strain level are available in Leong et al. (2004).

7.3.2.1 CU Test Results for Site 3

Site 3 is in Jurong Formation. The undisturbed soil sample was collected from 2 different depths for this site, at a depth of 10.15 m of (S3D10.15) and 16.6 m (S3D16.6). According to borehole information, specimen S3D10.15 consisted of very stiff fine to coarse grained sandy clay with rock fragments and specimen S3D16.6 consisted of sandy clay with rock fragments. The test results for specimens S3D10.15 and S3D16.6 are shown in Figure 7.25 and Figure 7.26, respectively.

Specimen S3D10.15 and S3D16.6 failed at 400 kPa and 330 kPa, respectively. Figure 7.25(b) and Figure 7.26(b) show that the Poisson's ratio of the soil increases during loading. Figure 7.25(c) and Figure 7.26(c) show that the Poisson's ratio for axial strain less than 1% is almost constant, 0.15 and 0.38 for soil specimens S3D10.15 and S3D16.6, respectively. Poisson's ratio is expected to be 0.5 for undrained loading of saturated soils. Therefore the test results indicated that the soil specimens were not fully saturated even with B value of 0.94 and 0.97. The shear modulus, G , versus ϵ_a curve (Figure 7.25(d) and Figure 7.26(d)) shows the typical trend for both the specimens, S3D10.15 and S3D16.6. Figure 7.25(e) and Figure 7.26(e) show the soil

specimen behave as normally consolidated soil with generation of positive excess pore-water pressure during shearing (Figure 7.25f and Figure 7.26f).

(a) Deviator stress versus strain

(b) Radial strain versus axial strain

(c) Radial strain versus LDT axial strain

(d) Shear modulus versus shear strain

(e) Deviator stress versus mean stress

(f) Excess pore-water pressure versus axial strain

Figure 7.25 CU test results for soil specimen S3D10.15

(a) Deviator stress versus strain

(b) Radial strain versus axial strain

(c) Radial strain versus LDT axial strain

(d) Shear modulus versus shear strain

(e) Deviator stress versus mean stress

(f) Excess pore-water pressure versus axial strain

Figure 7.26 CU test results for soil specimen S3D16.6

7.3.2.2 CU Test Results for Site 4

Site 4 is in Old Alluvium. The undisturbed soil sample was collected from 3 different depths for this site, at a depth of 12.3 m (S4D12.3), 21.2 m (S4D22.2) and 22.5 m (S4D22.5). According to borehole information, specimen S4D12.3 and S4D22.2 consisted of silty sand and specimen S4D22.5 consisted of dense sandy clay. The test results for sample S4D12.3, S4D22.2 and S4D22.5 are shown in Figure 7.27, Figure 7.28 and Figure 7.29, respectively. Unfortunately the soil specimens, S4D21.2 and S4D22.5 were only 100 mm height. Therefore, LDT readings were not available for these specimens as the LDT is of length 100 mm.

Specimen S4D12.3, S4D22.2 and S4D22.5 failed at 64 kPa 172 kPa and 288 kPa, respectively. Figure 7.27(b), Figure 7.28(b) and Figure 7.29(b) show that the Poisson's ratio of the soil increases during loading. Figure 7.27(c) shows that the Poisson's ratio for axial strain less than 1% is almost constant, 0.2 for soil specimen S4D12.3. Similar to the CU test results for Site 3, the soil specimens were not fully saturated even with B value greater than 0.95. The shear modulus, G , versus ε_a curve (Figure 7.27d, Figure 7.28d and Figure 7.29d) shows the typical trend for the specimens, S4D12.3, S4D22.2 and S4D22.5, respectively. Figure 7.27(e), Figure 7.28(e) and Figure 7.29(e) show the soil specimen behave as normally consolidated soil with generation of positive excess pore-water pressure during shearing (Figure 7.27f, Figure 7.28f and Figure 7.29f).

(a) Deviator stress versus strain

(b) Radial strain versus axial strain

(c) Radial strain versus LDT axial strain

(d) Shear modulus versus shear strain

(e) Deviator stress versus mean stress

(f) Excess pore-water pressure versus axial strain

Figure 7.27 CU test results for soil specimen S4D12.3

(a) Deviator stress versus strain

(b) Radial strain versus axial strain

Not available

(c) Radial strain versus LDT axial strain

(d) Shear modulus versus shear strain

(e) Deviator stress versus mean stress

(f) Excess pore-water pressure versus axial strain

Figure 7.28 CU test results for soil specimen S4D21.2

(a) Deviator stress versus strain

(b) Radial strain versus axial strain

Not available

(c) Radial strain versus LDT axial strain

(d) Shear modulus versus shear strain

(e) Deviator stress versus mean stress

(f) Excess pore-water pressure versus axial strain

Figure 7.29 CU test results for soil specimen S4D22.5

7.3.2.3 CU Test Results for Site 5

Site 5 is in Bukit Timah Granite. The undisturbed soil sample was collected from four different depths for this site, at a depth of 8.9 m (S5D8.9), 14.9 m (S5D14.9), 17.85 m (S5D17.85) and 20.85 m (S5D20.85). According to borehole information, specimen S5D8.9 and S5D14.9 consisted of loose clayey/silty coarse sand and specimens S5D17.85 and S5D20.85 consisted of very stiff to hard sandy silt. The test results for specimens S5D8.9, S5D14.9, S5D17.85 and S5D20.85 are shown in Figure 7.30, Figure 7.31, Figure 7.32 and Figure 7.33, respectively.

Specimens S5D8.9, S5D14.9, S5D17.85 and S5D20.85 failed at 138 kPa, 293 kPa, 459 kPa and 329 kPa, respectively. Figure 7.30(b), Figure 7.31(b), Figure 7.32(b) and Figure 7.33(b) show that the Poisson's ratio of the soil increases during loading. Figure 7.30(c), Figure 7.31(c), Figure 7.32(c) and Figure 7.33(c) shows that the Poisson's ratio for axial strain less than 1% is almost constant: 0.21, 0.37, 0.20 and 0.44 for soil specimens S5D8.9, S5D14.9, S5D17.85 and S5D20.85, respectively. Similar to CUT test results for Site 3 and 4, the soil specimens were not fully saturated for B value of 0.94 and higher. However, the Poisson's ratios obtained show that the higher the B value, the closer the Poisson's ratio to 0.5. The shear modulus, G , versus ε_a curve (Figure 7.30d, Figure 7.31d, Figure 7.32d and Figure 7.33d) show typical trend for all specimens. Figure 7.30(e), Figure 7.31(e), Figure 7.32(e) and Figure 7.33(e) show the soil specimen behave as normally consolidated soil with generation of positive excess pore-water pressure during shearing (Figure 7.30f, Figure 7.31f, Figure 7.32f and Figure 7.33f).

(a) Deviator stress versus strain

(b) Radial strain versus axial strain

(c) Radial strain versus LDT axial strain

(d) Shear modulus versus shear strain

(e) Deviator stress versus mean stress

(f) Excess pore-water pressure versus axial strain

Figure 7.30 CU test results for soil specimen S5D8.9

(a) Deviator stress versus strain

(b) Radial strain versus axial strain

(c) Radial strain versus LDT axial strain

(d) Shear modulus versus shear strain

(e) Deviator stress versus mean stress

(f) Excess pore-water pressure versus axial strain

Figure 7.31 CU test results for soil specimen S5D14.9

(a) Deviator stress versus strain

(b) Radial strain versus axial strain

(c) Radial strain versus LDT axial strain

(d) Shear modulus versus shear strain

(e) Deviator stress versus mean stress

(f) Excess pore-water pressure versus axial strain

Figure 7.32 CU test results for soil specimen S5D17.85

(a) Deviator stress versus strain

(b) Radial strain versus axial strain

(c) Radial strain versus LDT axial strain

(d) Shear modulus versus shear strain

(e) Deviator stress versus mean stress

(f) Excess pore-water pressure versus axial strain

Figure 7.33 CU test results for soil specimen S5D20.85

7.3.2.4 CU Test Results for Site 6

Site 6 is in Bukit Timah Granite. The undisturbed soil sample was collected from a depth of 12.35 m (S6D12.35). According to borehole information, specimen S6D12.35 consisted of firm, very stiff to hard sandy clay. The test results for specimen S6D12.35 is shown in Figure 7.34 . It is seen from Figure 7.34 (b) The axial strain (LVDT) versus radial strain curve is not smooth as a part of the aluminium foil which was stuck to the rubber membrane became unstuck during the consolidation process of the specimen. This unstuck aluminium foil affected the proximity transducer reading during the test.

Specimen S6D12.35 failed at 110 kPa. Figure 7.34 (b) shows that the Poisson's ratio of the soil increases during loading. Poisson's ratio for axial strain less than 1% is almost constant, 0.18, for soil specimen S6D12.35 (Figure 7.34c). Similar to the CU test results for Sites 3 to 5, the soil specimens were not fully saturated for B value of 0.95. The shear modulus, G , versus ε_a curve (Figure 7.34d) shows the typical trend. Figure 7.34 (e) shows the soil specimen behave as normally consolidated soil with generation of positive excess pore-water pressure during shearing (Figure 7.34f)

Figure 7.34 CU test results for soil specimen S6D12.35

7.3.2.5 CU Test Results for Site 7

Site 7 is in Kallang Formation. The undisturbed soil sample was collected from 2 different depths for this site, at a depth of 12.5 m (S7D12.5) and 21.5 m (S7D21.5). According to borehole information, both the specimens S7D12.5 and S7D21.5 consisted of very soft marine clay. The test results for specimens S7D12.5 and S7D21.5 are shown in Figure 7.35 and Figure 7.36, respectively.

Specimens S7D12.5 and S7D21.5 failed at 75 kPa and 120 kPa, respectively. Figure 7.35(b) and Figure 7.36(b) show that the Poisson's ratio of the soil increases during loading. Figure 7.35(c) and Figure 7.36(c) show that the Poisson's ratio for axial strain less than 1% is almost constant, 0.19 and 0.33 for soil specimens S7D12.5 and S7D21.5, respectively. Similar to the CU test results for Site 3 to 6, the soil specimens were not fully saturated for B value of 0.96. The shear modulus, G , versus ε_a curve (Figure 7.35d and Figure 7.36d) shows the typical trend for both the specimens, S7D12.5 and S7D21.5. Figure 7.35(e) and Figure 7.36(e) show the soil specimen behave as normally consolidated soil with generation of positive excess pore-water pressure during shearing (Figure 7.35f and Figure 7.36f).

The elastic modulus measured in this research was secant modulus. The elastic moduli during initial loading E_{initial} , unloading and reloading E_{ur} and at 50% of maximum deviatoric stress E_{50} are summarized in Table 7.8 for reference.

Table 7.8 Elastic moduli E_{initial} , E_{ur} and E_{50}

Specimen	E_{loading} (MPa)	$E_{\text{unloading}}$	E_{50} (MPa)
Site 3			
S3D10.15	31.6	28.2	10.2
S3D16.6	136.0	129.8	58.0
Site 4			
S4D12.3	27.7	20.8	5.9
Site 5			
S5D8.9	154.0	148.6	35.3
D5D14.9	48.8	47.9	4.2
S5D17.85	750.0	600.5	104.0
S5D20.85	20.0	18.3	5.8
Site 6			
S6D12.35	38.0	30.4	8.9
Site 7			
S7D12.5	60.3	54.9	11.4
S7D21.5	38.8	28.9	3.0

(a) Deviator stress versus strain

(b) Radial strain versus axial strain

(c) Radial strain versus LDT axial strain

(d) Shear modulus versus shear strain

(e) Deviator stress versus mean stress

(f) Excess pore-water pressure versus axial strain

Figure 7.35 CU test results for soil specimen S7D12.5

Figure 7.36 CU test results for soil specimen S7D21.5

7.3.2.6 Stiffness-Strain Relationship

The stiffness-strain relationship of soil is affected by a number of parameters: e , σ_3 , PI, OCR, geologic age, cementation, strain rate and number of loading cycles (Kramer, 1996). The test results showed that all the soil specimens exhibited normally consolidated behaviour, therefore OCR is not discussed. Furthermore, Schneider et al. (1999) found that OCR did not have any noticeable effect on the stiffness-strain curve. In this study, only the effects of e , σ_3 and PI on stiffness-strain relationship

were investigated. As the tested specimens were undisturbed and were tested under the in-situ stress conditions, the effects of e , σ_3 and PI could only be investigated approximately and to a limited extent. The soil specimens are grouped as summarized in Table 7.9 to study the effects of e , σ_3 and PI.

Table 7.9 Grouping of undisturbed soil specimens to study the effects of e , σ_3 and PI

Parameter	Specimen	e	σ_3 (kPa)	PI
e	S4D12.3	0.58	120	21
	S6D12.35	0.92	110	24
σ_3	S5D14.9	0.88	130	17
	S5D20.85	0.78	185	16
PI	S5D14.9	0.88	130	17
	S6D12.35	0.92	110	24

Specimens S4D12.3 and S6D12.35 showed similar PI and σ_3 but different e (0.58 and 0.92, respectively). The plotted data are shown in Figure 7.37. It can be seen from Figure 7.37 that the lower void ratio shows higher shear modulus, which was found by other researchers as well (Kramer, 1996).

Figure 7.37 Effect of void ratio

Specimens S5D14.9 and S5D20.85 showed similar PI and e but different σ_3 (130 kPa, and 185 kPa, respectively). The plotted data are shown in Figure 7.38. It can be seen from Figure 7.38 that the shear modulus was higher for specimen S5D20.85 than specimen S5D14.9, which was expected, as the higher the confining pressure the higher the shear modulus.

Figure 7.38 Effect of confining pressure

Specimens S5D14.9 and S6D12.35 showed similar e and σ_3 but different PI (17 and 24, respectively). The plotted data are shown in Figure 7.39. It can be seen from Figure 7.39 that specimen S5D14.9 which has lower PI (=17) shows higher shear modulus than specimen S6D12.35 which has higher PI (=24). This observation does not agree with the literature. For S5D14.9 and S6D12.35, e are 0.88 and 0.92, respectively and σ_3 are 130 and 110 kPa, respectively. From earlier discussion based on e alone, S5D14.9 should show a higher shear modulus than S6D12.35; based on σ_3 , S5D14.9 should also show a higher shear modulus. That implies that effects of e and σ_3 on shear modulus are more dominant than the effect of PI.

Figure 7.39 Effect of PI

7.4 Comparison of Laboratory and Field Measured V_s

Laboratory measurements have long been the reference standard for determining the properties of geomaterials (Schneider et al., 1999). Prasad et al. (2004) showed that laboratory measurements could be powerful predictive tools to interpret field seismic data by establishing relationships between physical properties of unconsolidated sediments and P- and S-wave propagation through them.

The comparison of V_s obtained from CSW surveys and microtremor surveys and ultrasonic velocity tests are shown in Figure 7.40 to Figure 7.44. It is seen from Figure 7.40 to Figure 7.44 that the discrepancy is not significant.

Figure 7.40 Comparison of shear wave velocity profile for Site 3

Figure 7.41 Comparison of shear wave velocity profile for Site 4

Figure 7.42 Comparison of shear wave velocity profile for Site 5

Figure 7.43 Comparison of shear wave velocity profile for Site 6

Figure 7.44 Comparison of shear wave velocity profile for Site 7

The comparison of field and laboratory measured data is summarized in Table 7.10. The physical properties of the soil specimen are reported in Chapter 6. However to make it more convenient for comparison, the soil properties are repeated in Table 7.10. The specimens listed in Table 7.10 are only for those where both laboratory and field measured V_S are available. V_S measured by CSW and by ultrasonic velocity test are plotted in Figure 7.45. It can be seen from Figure 7.45 that the difference in V_S is $\pm 30\%$.

Table 7.10 Comparison of field and laboratory measured V_S

Specimen	G_s	ρ (Mg/m ³)	e	PI	V_S (m/s)			USCS
					Lab.	Field	$\frac{V_S(lab.) - V_S(field)}{V_S(lab.)}$	
S3D10.15	2.65	2.1	0.49	12	317	359.5	-13.4	CL
S4D12.3	2.58	1.99	0.58	21	224.5	282	-25.6	CL
S5D8.9	2.63	1.79	1.07	22	153	169	-10.5	ML
S5D14.9	2.64	1.85	0.88	17	329	270	17.9	CL
S5D17.85	2.57	1.89	0.77	18	382	270	29.3	CL
S6D12.35	2.63	1.85	0.92	24	240	205	14.6	CH
S7D12.5	2.3	1.59	1.24	42	233	198	15	CH

Figure 7.45 Comparison between field and laboratory measured V_S

7.5 Comparison of Laboratory and Field Measured V_S with V_S from Empirical Relationships

Apart from the field and laboratory measurement, V_S can also be calculated from empirical correlations. Researchers have tried to find the relationship between V_S and soil properties over the last few decades, some of these relationships are:

Hardin and Black (1968) reported an empirical relationship to calculate G_{max} for angular grained sandy materials as follow:

$$G_{max} = \frac{3230(2.97 - e)^2}{1 + e} \sigma_0^{0.5} \quad (7.17)$$

where σ_0 is field mean effective stress.

For rounded grained sandy materials, G_{max} is given as follow:

$$G_{max} = \frac{6908(2.17 - e)^2}{1 + e} \sigma_0^{0.5} \quad (7.18)$$

As no information is available on K_0 , it is assumed that K_0 is 1. Therefore, in this research, the effective overburden stress used is mean effective stress. If the soil types in the borehole logs are examined, it can be appreciated that the soil specimens are either residual soil or sand. Therefore, Equations 7.17 and 7.18 were used in this research.

Laboratory and field measured V_S can be compared with V_S computed by empirical relationships. This comparison will enable the evaluation of the applicability of the empirical correlations. The calculated V_S are summarized in Table 7.11. It can be seen from Table 7.11 that V_S calculated by CSW differs from V_S by ultrasonic velocity test by $\pm 30\%$; V_S by Equation 7.17 by $\pm 26\%$, V_S by Equation 7.18 by $\pm 17\%$. Therefore, the empirical formulas, Equation 7.17 and Equation 7.18, can be used to estimate V_S to within $\pm 30\%$.

Table 7.11 Comparison of field and laboratory measured V_S with V_S computed by empirical relationships

Soil Specimen	V_S (m/s)				% Error with CSW test measurement		
	CSW test	Ultrasonic velocity test	Equation		Ultrasonic Velocity test	Equation	
			(7.17)	(7.18)		(7.17)	(7.18)
S3D10.15	359.5	317	303.4	300.5	-13.4	15.6	16.4
S4D12.3	282	224.5	296.2	288.0	-25.6	-5.0	-2.1
S5D8.9	169	153	212.5	179.8	-10.5	-25.7	-6.4
S5D14.9	270	329	270.5	244.1	17.9	-0.2	9.6
S5D17.85	270	382	298.1	277.4	29.3	-10.4	-2.7
S6D12.35	205	240	254.1	226.5	14.6	-23.9	-10.5
S7D12.5	198	233	207.4	163.0	15	-4.8	17.7

Standard Penetration Test (SPT) was conducted during the site investigation for the five sites. Since N-value is available, laboratory and field measured V_S can also be compared with V_S estimated from N-value.

The empirical relationship based on N-value (Equation 7.19) to calculate V_S was developed by Seed and Idriss (1970). This relationship was developed for sand and silty sand but was used for all five sites in this study.

$$V_S = 56.388\sqrt{N} \quad (7.19)$$

Computed V_S using Equation 7.19 is shown in Figure 7.46 to Figure 7.50 to compare with V_S obtained from CSW surveys. In Figure 7.46 to Figure 7.50 $\pm 40\%$ variation of V_S obtained from CSW surveys are also plotted. The V_S obtained from Equation 7.19 are within $\pm 30\%$ of the V_S obtained from CSW surveys.

Figure 7.46 Comparison of V_S for Site 3

Figure 7.47 Comparison of V_s for Site 4

Figure 7.48 Comparison of V_s for Site 5

Figure 7.49 Comparison of V_s for Site 6

Figure 7.50 Comparison of V_s for Site 7

7.6 Summary

Result of ultrasonic velocity tests on compacted soil samples are presented in this chapter. A relationship between compression wave velocity with shear wave velocity and degree of saturation was developed in this chapter. The relationship could be used to estimate compression wave velocity of an unsaturated soil/rock/concrete within $\pm 20\%$.

Ultrasonic velocity tests and triaxial compression tests were conducted on undisturbed soil samples to investigate the small-strain stiffness and stiffness-strain relationship. The result showed that stiffness-strain relationship is affected by void ratio, confining pressure and plasticity index. Increase in void ratio reduces the shear stiffness. Increase in confining pressure and plasticity index increase the shear stiffness of soil. . Although these are known behaviour of the soil, this research reaffirmed the effects of void ratio, confining pressure and plasticity index on shear stiffness of the soil. It was also found that the effect of void ratio is more significant than the effect of plasticity index on shear stiffness of the soil. The comparison of V_S obtained CSW surveys and microtremor surveys and ultrasonic velocity tests are made in this chapter. V_S obtained from CSW surveys showed $\pm 30\%$ differences with V_S obtained from ultrasonic velocity tests. Some empirical relationships to compute V_S were also evaluated. The computed V_S also evaluated from empirical formulas showed $\pm 17\%$ to $\pm 25\%$ discrepancy with V_S obtained from CSW survey. The comparison of V_S obtained by CSW surveys with V_S computed by empirical correlation using N-value showed $\pm 40\%$ difference.

CHAPTER 8

CONCLUSION AND RECOMMENDATIONS

8.1 Introduction

The main objective of this study is to investigate the use of seismic velocity for characterization of soil properties. Soil profiles were characterized using CSW and microtremor survey methods with different methods of inversion. This study also aimed to investigate the small-strain stiffness by ultrasonic velocity test and then to make a comparison between field and laboratory measurement of small-strain stiffness. The relationship between small-strain stiffness from seismic velocities and stiffness from triaxial compression were also investigated in this study.

The scope of the research includes conducting CSW surveys and microtremor surveys at a number of sites in Singapore. Inversion of the dispersion curves obtained from CSW and microtremor surveys using different inversion methods. Conducting laboratory ultrasonic velocity tests on compacted and undisturbed soil samples to measure V_P and V_S and triaxial compression tests with local strain measurement to determine the variation of shear modulus-strain.

This chapter summarized the important findings of this research followed by recommendations for future research.

8.2 Conclusion

8.2.1 Field Tests

Two field test methods, CSW and microtremor survey methods, were performed at different sites in Singapore using CSWS. The findings from field experiments are given below:

(a) Site characterization by CSW performed at seven sites in Singapore with geophones along four different orthogonal lines showed the same shear wave velocity profile. This method of survey can be used to ascertain whether the sites are horizontally stratified. The depth of investigation in CSW method depends on soil type. The investigation depth for Site 1 to Site 6 was about 70 m in terms of wavelength but for Site 7 which consists of soft soil, it was 40 m.

(b) Microtremor surveys were also conducted at the seven sites in Singapore where CSW method was applied. The SPAC method (Aki, 1957) which computes the dispersion curve from field data was implemented in Matlab and evaluated using four case histories. Comparison with the dispersion curves in the case histories showed good agreement. To convert the dispersion curves to shear wave velocity profile, the linear elastic inversion algorithm proposed by Lai and Rix (1998) was used. Comparison with the shear wave velocity profiles obtained from CSW method and microtremor survey method showed good agreement. The good comparison of shear wave velocity profiles with borehole log implies that CSW and microtremor survey data are of fundamental mode of Rayleigh wave data. However, resolution of the profile in case of microtremor survey was not good. It was found from the comparison that depth of investigation was shallower for microtremor survey. The shallower depth of investigation in microtremor survey was due to the small array size (less than 10 m).

(c) In this study, a modified wavelength-depth factor method was developed. A relationship between wavelength-depth factor and wavelength was established for different frequencies. This relationship can be used to obtain the approximate wavelength-depth factor given the dispersion curve. However, this relationship is developed for a regular soil profile, i.e., phase velocities of the soil layers increase with depth. Evaluation of the modified wavelength-depth method showed that it can give good estimation of soil layer thickness but a fair estimate of shear velocity. Nevertheless, the modified wavelength-depth method can provide a good initial model for more accurate inversion schemes.

8.2.2 Laboratory Tests:

Two laboratory tests, ultrasonic velocity test as well as triaxial compression test was performed in the laboratory.

(a) *Ultrasonic test*

Ultrasonic velocity tests were conducted on compacted unsaturated soil samples to investigate the effect of degree of saturation and confining pressure on compression wave and shear wave velocities. It was found that V_P and V_S did not vary significantly with the variation of confining pressure. A relationship was developed to account for the effect of confining pressure on V_P and V_S (Equation 7.5 and Equation 7.6, respectively). This relationship can be used to determine V_P and V_S when V_P and V_S at zero confining pressure are known.

It was found that degree of saturation did not have significant effect on shear wave velocity but compression wave velocity was found to be dependent on degree of saturation. A relationship between compression wave velocity with shear wave velocity and degree of saturation was developed (Equation 7.15). The relationship could be used to estimate compression wave velocity of an unsaturated soil/rock/concrete with ± 20 % error.

The compression and shear wave velocities obtained from ultrasonic velocity tests of undisturbed soil samples were compared with those obtained from CSW tests. It was found that the difference is within ± 20 %.

Some empirical relationships to estimate shear wave velocities were also evaluated. The estimated shear wave velocities from empirical relationships differ from those obtained by CSW tests from ± 17 % to ± 40 %.

(b) Triaxial compression test

Triaxial compression tests were conducted on undisturbed soil specimens to investigate the stiffness-strain relationship of the soil. In this research, small-strain measurements of the axial deformation as well as radial deformation were made using LDT and proximity transducers, respectively. Poisson's ratios were obtained from these deformation measurements. The Poisson's ratios obtained showed that the soil specimens were not fully saturated even with a B valued as high as 0.97. The stiffness-strain relationship from the very small-strain stiffness obtained from ultrasonic velocity test and the stiffness from the triaxial test show a continuous curve. The effect of void ratio, confining pressure and plasticity index as the stiffness-strain relationship were investigated. It was found that G increases with increase in plasticity index and confining pressure but decreases with the increase in void ratio. It was also found that the effect of void ratio and confining pressure is more significant than the effect of plasticity index on G of soil.

8.3 Recommendations

This research covers a number of interesting aspects of seismic velocities in the characterization of soil properties. However limitation of time and resources inevitably means a less than thorough investigation in some areas of the research which should be addressed in future research. The following should be looked into in future research:

- Only CSW and microtremor surveys were conducted in this study. It will be useful if invasive geophysical tests are also available to better understand the differences in the geophysical tests.
- Microtremor surveys are able to achieve a deeper depth of investigation if the array size is larger than those conducted in this study. It will be useful to use the microtremor survey as a means to extend the depth of investigation of the CSW surveys. Therefore, in future array size larger than 10 m should be used in the microtremor surveys.

- More meaningful validation on accuracy and reliability of the characterisation depth and shear wave velocity values from CSW and microtremor survey can be obtained by comparison with cross-hole and downhole seismic surveys.
- The modified wavelength-depth method proposed in this study has been found to work well for regular soil profile, i.e., where soil stiffness increases with depth. An evaluation of the modified wavelength-depth method with irregular soil profiles should be conducted to determine if the method is applicable or could be further improved.
- The estimation of shear wave velocity in the modified wavelength method is unsatisfactory and should be improved further.
- The inversion method of Lai and Rix (1998) was used as the more accurate inversion method in this study. Other inversion methods should be evaluated especially newer inversion methods where attenuation is also accounted for.
- Relationship between compression wave velocity with shear wave velocity and degree of saturation was developed in this study. Though the relationship showed generally good estimation of the compression wave velocity over the full range of saturation, the relationship may not yield accurate results near 100% saturation where compression wave velocity is very sensitive to small amount of air present in the water phase. More research is needed to address this problem.
- Although the very small-strain shear stiffness from ultrasonic velocity test and the stiffness from triaxial test show a reasonable variation with strain, there is a gap in the data for strain between 0.001% and 0.00001%. Improvement should be made to the local strain measurements to bridge this data gap.
- The stiffness-strain relationships of Singapore soils have not been fully investigated. More tests on undisturbed soils should be conducted to provide more information.
- The effects of void ratio, confining pressure and PI were only investigated in a limited way in this research due to the small number of tests. This can be overcome with more tests.
- Many relationships have been suggested for the stiffness-strain curve. These relationships have not been evaluated in this study and should be investigated.

References:

1. Abbiss, C. P., 1981, Shear wave measurement of the elasticity of the ground. *Géotechnique*, Vol. 31, No. 1, pp. 91-104.
2. Achenbach, J. D., 1975, "Wave propagation in elastic solids", *North- Holland, Amsterdam*, pp. 425.
3. Addo, K. O. and Robertson, P. K., 1992, Shear Wave Velocity Measurement of Soils Using Rayleigh Waves, *Canadian Geotechnical Journal*, Vol. 29, pp. 558-568.
4. Akagawa, K., 1974, Gas-liquid two phase flow, *Corona Publishing Company* (in Japanese)
5. Aki, K., 1957, Space and time spectra of stationary stochastic waves, with special reference to microtremors, *Bull. Earthq. Res. Inst.*, Vol. 35, pp. 415-456.
6. Anand, S., 2007, Measurement and Modelling of Ground Response due to Dynamic Loading, *PhD thesis*, Nanyang Technological University, Singapore.
7. Anand, S., 2001, Measurement and Modelling of Ground Response due to Dynamic Loading, *First year report*, Nanyang Technological University, Singapore.
8. Aanderson, D. G. and Richart, F. E. Jr., 1976, Cyclic simple shear size effect, *Journal of Geotechnical Engineering*, Vol. 113, No. 7, pp. 693-707.
9. Anderson, D. G. and Woods, R. D., 1976, Time dependent increase in shear modulus of clay, *J. Geotech. Div., ASCE*, Vol. 102, No. 5, pp. 525-537.

10. Apostolidis, P., Raptakis, D., Roumelioti, Z. and Pitilakis, K., 2004, Determination of S-wave velocity structure using microtremors and SPAC method applied in Thessaloniki (Greece), *Soil Dynamics and Earthquake Engineering*, 24, pp. 49–67.
11. Aracne-Ruddle, C. M., Bonner, B. P., Trombino, C. N., Hardy, E. D., Berge, P. A., Boro, C. O., Wildenschild, D., Rowe, C. D. and Hart, D. J., 1999, Ultrasonic Velocities in Unconsolidated Sand/Clay Mixtures at Low Pressures, *American Geophysical Union Fall Meeting*, San Francisco, CA, December 13 to 17.
12. Arai, H. and Tokimatsu, K., 2005, S-wave velocity profiling by joint inversion of microtremor dispersion curve and horizontal-to-vertical spectrum, *Bulletin of the Seismological Society of America*, Vol. 95, No. 5, pp. 1766–1778.
13. Arulnathan, R., Boulager, R. W., and Riemer, M. F, 1998, Analysis of bender element tests. *Geotechnical Testing Journal*, Vol. 21, No. 2, pp. 120-131.
14. Ashford, S. A. and Jakrapiyanun, W., 1999, Comparison of estimated and measured v_s profile for Bangkok, Thailand, *Geotechnical Engineering Journal*, Southeast Asian Geotech. Soc., Vol. 30, No. 3, pp. 167-181.
15. Asten, M. W., 1978, Geological Control on the Three-Component Spectra of Rayleigh-Wave Microseisms, *Bulletin of the Seismological Society of America*, Vol. 68, No. 6, pp. 1623-1636.
16. Asten, M. W. and Henstridge, J. D., 1984, Array estimator and the use of microseisms for reconnaissance of sedimentary basins, *Geophysics*, Vol. 49, No. 11, pp. 1828–1837.
17. Asten, M. W., 2001, The spatial auto-correlation method for phase velocity of microtremors- another method for characterization of sedimentary overburden, *Earthquake codes in the real world*, Aust. Earthquake Eng. Soc., Canberra, Nov. 2001.

18. Asten, M. W., Lam, N., Gibson, G. and Wilson, J., 2002, Microtremors survey design optimised for application to site amplification and resonance modeling, *Aust. Earthquake Eng. Soc.*, Adelaide S.A., October 2002.
19. Asten, M. W., Dhu, T., Jones, A. and Jones, T., 2003, Comparison of shear-velocities measured from microtremor array studies and SCPT data acquired from earthquake site hazard classification in the northern suburbs of Perth W. A., *Earthquake Risk Mitigation*, (Eds) J. L. Wilson, N. K. Lam, G. Gibson, B. Butler, Proceedings of a Conference of the Australian Earthquake Engineering Soc., Melbourne, Paper 12.
20. Asten, M. W., 2003, Lessons from alternative array design used for high-frequency microtremor array studies: in *Earthquake Risk Mitigation*, Wilson, J.L., Lam, N.K., Gibson, G.M., and Butler, B., *Proceedings of a Conference of the Australian Earthquake Engineering Society*, Melbourne, Paper No. 14.
21. Asten, M. W., 2004, Passive seismic methods using the microtremor wave field,; *17th Geophysical Conference and Exhibition*, Australian Society of Exploration Geophysicists, Extended Abstracts.
22. ASTM, 1996, D4767-95, Standard test method for consolidated undrained triaxial compression test for cohesive soils, *Annual Book of ASTM Standards*, 4.08.
23. ASTM, 1997, D2845-95, Laboratory compaction characteristics of soil using modified effect, *Annual Book of ASTM Standards*, 4.08.
24. Atkinson, J. H., 2000, Non linear soil stiffness in routine design, *Géotechnique*, Vol. 48, No. 5, pp. 487-508.
25. Ballard, R. F. and McLean, F. G., 1975, Seismic field methods for in-situ moduli, *Proceedings Conference on In-situ measurement of soil properties*, Specialty Conference of the Geotechnical Engineering Division ASCE, Raleigh, North Carolina, 1, pp. 121-150.

26. Barański, M., 2003, Laboratory measurements of the stiffness of normally consolidated till polluted by benzene with bender element tests, *Geological Quarterly*, Vol. 47, No. 2, pp. 211-214.
27. Bath, M., 1973, Introduction to seismology: *A Halsted Press Book*, pp. 395.
28. Beaty K. S., Schmitt D. R. and Sacchi M., 2002, Simulated annealing inversion of multimode Rayleigh wave dispersion curves for geological structure. *Geophysical Journal International*, Vol. 151, pp. 622-631.
29. Bettig, B., Bard, P. Y. F., Scherbaum, J. R., Cotton, F., Cornou, C. and Hatzfeld, 2001, Analysis of dense array noise measurements using the modified spatial auto-correlation method (SPAC): application to the Grenoble area, *Bolletino di geofisica teorica ed applicata*, Vol. 42, No. 3-4, pp. 281-304.
30. Bolt, B. A., 1988, Earthquakes, *W. H. Freeman and Company*, New York, 282.
31. Boore, D. M. and Brown, L. T., 1998, Comparing shear-wave velocity profiles from inversion of surface-wave phase velocities with downhole measurements; systematic differences between the CXW method and downhole measurements at six USC strong-motion sites, *Seismol. Res. Lett.*, Vol. 69, pp. 222-229.
32. Borchardt, R. D., 1994, Estimates of site-dependent response spectra for design-methodology and justification, *Earthquake Spectra* 10, pp. 617– 653.
33. Borden, R. H., Shao, L. and Gupta, A., 1996, Dynamic properties of Piedmont residual soils, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 122 (10), pp. 813-821.

34. Bowles, J. E., 1997, *Foundation Analysis and Design*, Fifth edition, *McGraw-Hill International Editions*, Civil Engineering Series, printed in Singapore.
35. Braile, L. W. and Keller, G. R., 1975, Fine structure of the crust inferred from linear inversion of Rayleigh-wave dispersion, *Bulletin of the Seismological Society of America*, Vol. 65, pp. 71-83.
36. Breuer, H., 1975, *Physics for Life Science Students*. *Prentice-Hall, Inc. Englewood Cliffs*, N.J.
37. Brignoli, E.G.M., Gotti, M. and Stokoe II, K. H., 1996, Measurement of Shear Waves in Laboratory Specimens by Means of Piezoelectric Transducers, *Geotechnical Testing Journals*, American Society for Testing and Materials. Vol. 19. No. 4, pp.384-397.
38. Brown, L. T., 1998, Comparison of V_s profiles from SASW and borehole measurements at strong motion sites in southern California, *M. Sc. Eng. Thesis*, University of Texas at Austin, pp. 349.
39. Brown, L. T., Boore, D. M. and Stokoe II, K. H., 2000, Comparison of shear-wave velocity profiles from SASW and downhole seismic tests at a strong-motion site, *Proc. 12th World Conference on Earthquake Engineering*, Auckland, New Zealand, 30 January–4 February, 2000.
40. BS EN 1992-1-1:2004, *Eurocode 2*, Design of concrete structures. General rules and rules for buildings.
41. Burland, J. B., 1989, Small is beautiful: the stiffness of soils at small strains, *Canadian Geotech. J.*, Vol. 26, No. 4, pp. 499-516.
42. Burton, C. K., 1964, The older Alluvium of Johore and Singapore, *Journal of Tropical Geography*, Vol. 18, pp. 30-42.

43. Butcher, A. P., Powell, J. J. M., 1996, Practical considerations for field geophysical techniques used to assess ground stiffness, *Proceedings, International Conference on Advances in Site Investigation Practice*, London, Thomas Telford, pp.701-714.
44. Butcher, A. P. and Tam, W., 1994, An example of the use of Rayleigh waves to detect the depth of a shallow landfill, *Proceedings 30th Conference Engineering Group Geological Society*, Modern geophysics in engineering geology, Liege, 1994.
45. Capon, J., Greenfield, R. J. and Kolker, R. J., 1967, Multidimensional maximum-likelihood processing of a larger aperture seismic array, *Proc. IEEE*, Vol. 55, pp. 192-211.
46. Capon, J., 1969, High-resolution frequency-wavenumber spectral analysis, *Proc. IEEE*, Vol. 57, pp. 1408-1419.
47. Chin, F., 1965, The upper Triassic sediments of Pasir Panjang-Jurong are of Singapore, *Honours thesis*, Department of Geology, University of Malaya.
48. Cho, I., Tada, T. and Shinozaki, Y., 2004, A new method to determine phase velocities of Rayleigh waves from microseisms: *Geophysics*, Vol. 69, pp. 1535-1551.
49. Clayton, C. R. I., Matthews, M. C., Gunn, M. J., Foged, N. and Gordon, M. A., 1995, Reinterpretation of surface wave tests for the Oresund crossing, *Proceedings 11th European Conference on Soil Mechanics and Foundation Engineering*, Copenhagen, Danish Geotech. Soc., Copenhagen, Vol. 1, pp. 141-147.
50. Constable, S. C., Parker, R. L. and Constable, G. G., 1987, Occam's Inversion: A Practical Algorithm for Generating Smooth Models from Electromagnetic Sounding Data, *Geophysics*, Vol. 52, pp. 289-300.

51. Cramer, C., Frankel, A. and Muller, C., 2002, A state-of-the-art seismic hazard map with site effects for Memphis, Shelby County, Tennessee. *7th U.S. National Conference on Earthquake Engineering (7NCEE)*, CD-ROM Proceedings, 21–25 July, Boston, Massachusetts.
52. Couvreur, J. F. and Thimus, J. F., 1996, The Properties of Coupling Agents in Improving Ultrasonic Transmission, *International Journal of Rock Mechanics and Mining Science and Geomechanics Abstracts*, Vol. 33. No. 4, pp. 417-424.
53. D'Angelo, R., Plona, T.J., Schwartz, L. M. and Coveney, P., 1995, Ultrasonic measurements on hydrating cement slurries, *Advanced Cement Based Materials* 2 (1), pp. 8-14.
54. Das, B. M., 1998, Principles of Geotechnical Engineering, Fourth Edition, *PWS publishing company*, 20 Park plaza, Boston, USA.
55. DeAlba, P., Baldwin, K., Janno, V., Roe, G. and Celikkol, B., 1984, Elastic wave velocities and liquefaction potential, *ASTM Geotech. Testing J.*, Vol. 7(2), pp 77-87.
56. Der, Z., Masse, R. and Landisman, M., 1970, Effects of observational errors on the resolution of surface waves at intermediate distances, *Journal of Geophysical Research*, Vol. 75, pp. 3399-3409.
57. Dobry, R. and Vucetic, M., 1987, Dynamic properties and seismic response of soft clay deposits, *Proceedings of International Symposium on Geotechnical Engineering of Soft Soils*, Mexico City, Vol. 2, pp. 51-87.
58. Domenico, S. N., 1976, Effect of brine-gas mixture on velocity in an unconsolidated sand reservoir, *Geophysics* Vol. 41, pp. 882–894.

59. Dorman, J. and Ewing, M., 1962, Numerical inversion of seismic surface wave dispersion data and crust-mantle structure in the New Your-Pennsylvania area, *Journal of Geophysical Research*, Vol. 67, No. 13, pp. 5227-5241.
60. Dyvik, R. and Madshus, C., 1985, Lab measurement of $G_{\max}$ using bender elements, *Proc. ASCE Conf. on Adv. Art of Testing Soils under Cyclic Conditions*, ASCE, New York, N. Y., pp. 186-196.
61. Elmore, W. C. and Mark A. H., 1969, Physics of Waves, *Dover Publications*, Inc., New York, pp.477.
62. Engl, H. W., 1993, Regularization Methods for the Stable Solution of Inverse Problems, *Surveys on Mathematics for Industry*, Vol. 3, pp. 71-143.
63. Estrella, H. F. and González, J. A., 2003, SPAC: An alternative method to estimate earthquake site effects in Mexico City. *Geofísica Internacional*, Vol. 42, No. 2, pp. 227-236.
64. Evans, J. M. Jr. and Pezeshk, S., 1998, West Tennessee site specific studies. *EERI Annual Meeting*, 4–7 February, San Francisco, Poster Session. Electric Power Research Institute (1993). Guidelines for determining design basis ground motions, Technical Report TR-102293.
65. Ewing, W. M., Jardetzky, W. S. and Press, F., 1957. Elastic Waves in Layered Media, *McGraw-Hill*, New York.
66. Fernandez, A. L. and Santamarina, J. C., 2001, Effect of cementation on the small-strain parameters of sands, *Canadian Geotechnical Journal*, Vol. 38, pp. 191–199.
67. Ferrazzini, V., Aki, K. and Chouet, B., 1991, Characteristics of seismic waves composing Hawaiian volcanic tremor and gas piston events observed by a near source array, *Journal of Geophysical Research*, Vol. 96, pp. 6199-6209.

68. Fioravante, V., 2000, Anisotropy of small strain stiffness of Ticino and Kanya sands from seismic wave propagation measured in triaxial testing, *Soils and Foundation*, Vol. 40(4), pp. 129-142.
69. Foti, S., 2000, Multistation Methods for Geotechnical Characterization using Surface Waves, *Ph.D. thesis*, Politecnico di Torin, Italy.
70. Foti, S., Lai, C. G. and Lancellotta, R., 2002, Porosity of fluid-saturated porous media from measured seismic wave velocities, *Géotechnique* Vol. 52, No. 5, pp. 359–373.
71. Fowler, C. M. R., 1990. *The Solid Earth*, Cambridge University Press, Cambridge
72. Ganji, V., Gucunski, N. and Nazarian, S., 1998, Automated inversion procedure for spectral analysis of surface waves, *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 124, No. 8, pp. 757-770.
73. Gazetas, G., 1982, Vibrational characteristics of soil deposits with variable velocity, *International Journal for Numerical and Analytical Methods in Geomechanics*, Vol. 6, pp. 1-20.
74. GCTS, 2001, Geotechnical Consulting and Testing Systems, *Data acquisition & ultrasonic velocity software user's guide*, Tempe, Arizona. USA.
75. GDS, 1998, The GDS Continuous Surface Wave System, *User handbook*, GDS, Instrument Ltd., 1998.
76. Gordon, M. A., Clayton, C. R. I., Thomas, T. C. and Matthews, M. C., 1995, The selection and interpretation of seismic geophysical methods for site investigation, *Proceedings Institution of Civil Engineers Conference on Advances in Site Investigation Practice*, March 1995, pp. 727-736.

77. Goto, S., Tatsuoka, F., Shibuya, S., Kim, Y-S. and Sato, T., 1991, A simple gauge for local small strain measurements in the laboratory, *Soils and Foundations*, Vol. 31, No. 1, pp. 169-180.
78. Gucunski, N. and Woods, R. D., 1991, Instrumentation for SASW testing, in geotechnical special, *Recent advances in instrumentation, data acquisition and testing in soil dynamics*, edited by S.K. Bhatia and G.W. Blaney, American Society of Civil Engineers, publication no. 29, pp. 1-16.
79. Guillaume, S., du Mouza, J. and Brulhet, J., 1994, Ultrasonic Control of the Textural Evolution of Salt Rocks Samples During Long Term Creep Tests, *Modern Geophysics in Engineering Geology*, Edited by McCann, D.M., Fenning, P. and Cripps, J., pp. 141-150, Engineering Group of the Geological Society, Liège.
80. Hardin, B. O. and Black, W. L., 1968, Vibration modulus of normally consolidated clay, *Journal of the Soil Mechanics and Foundation division*, ASCE, Vol. 94., No. SM2, pp. 353-369.
81. Hardin, B. O. and Drnevich, V. P., 1972, Shear modulus and damping in soils: Design equations and curves, *Journal of the Soil Mechanics and Foundation Division*, ASCE, Vol. 98, No. SM7, pp. 667-692.
82. Hardin, B. O. and Richart Jr., F. E., 1963, Elastic wave velocities in granular soils, *Journal of the Soil Mechanics and Foundation Division*, ASCE, Vol. 89, No. SM1, pp. 33-35.
83. Harkrider, D. G., 1964, Surface waves in multilayered elastic media I. Rayleigh and Love waves from buried sources in a multilayered elastic half-space, *Bulletin of the Seismological Society of America*, Vol. 54, No. 2, pp. 627-679.

84. Harvey, D., 1981, Seismogram synthesis using normal mode superposition: the locked mode approximation, *Geophysics Journal of the Royal Astr Society*, Vol. 66, pp. 37-70.
85. Haskell, N. A., 1953, The dispersion of surface waves on multilayered media, *Bulletin of the Seismological Society of America*, Vol. 43, pp. 17-34.
86. Henstridge, J. D., 1979, A signal processing method for circular arrays”, *Geophysics*, Vol. 44, No. 2, pp. 179-184.
87. Herrmann R. B., 1994, Computer programs in seismology. *User's Manual*, S. Louis University, Missouri (USA)
88. Heukelon, W. and Foster, C., 1962, Dynamic testing of pavements, Transportation, *American Society of Civil Engineers*, Vol. 127, pp. 425-457.
89. Hidaka, E., 1985, Phase velocity of Rayleigh waves and S-wave velocity distribution estimated from long period microtremors, *M. Sc. Thesis*, Hokkaido University: Japan. (in Japanese).
90. Hochstadt, H., 1986, The functions of mathematical physics, *Dover Publications*, Inc., New York, pp. 322.
91. Honjo, Y., Iwamoto, S., Sigimoto, M., Onimaru, S. and Yoshizawa, M., 1998, Inverse analysis of dynamic soil properties based on seismometer array records using the extended bayesian method, *Soils and Foundations*, Japanese Geotechnical Society, Vol. 38, No. 1, pp.131-143.
92. Hoque, E., Sato, T. and Tatsuoka, F., 1997, Performance Evaluation of LDTs for USE in Triaxial Tests, *Geotechnical Testing Journal*, GTJODJ, Vol. 20, pp. 149-167.

93. Horike, M., 1985, Inversion of phase velocity of long-period microtremors to the S-wave velocity structure down to the basement in urbanized areas, *J. Phys. Earth*. Vol. 33, pp. 59–96.
94. Horike, M., 1996, Geophysical exploration using microtremor measurement, *Eleventh World Conference on Earthquake Engineering*, Paper no. 2033.
95. Hough, S. E., et al., 1992, Ambient noise and weak motion excitation of sediment responses: Results from the Tiber Valley, Italy, *Bulletin of the Seismological Society of America*, Vol. 82, pp. 1186-1205.
96. Hoyos Jr., L. and Macari, E. J. 1999, Influence of in-situ factors on dynamic response of piedmont residual soils, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 125, No. 4, pp. 271-279.
97. Hryciw, R. D. and Thomson, T. G., 1993, Stress-history based model for G of cohesionless soils, *J. Geotech. Engrg.*, ASCE, Vol. 119(7), pp. 1073-1093.
98. Hunaidi, O., 1998, Evolution-based genetic algorithms for analysis of non-destructive surface wave tests on pavements, *NDT&E Int.*, Vol. 31, No.4, pp. 273-280.
99. Huo-Ni, S., 1987, Dynamic properties of sand under true triaxial stress states from resonant-column/torsional-shear tests, *PhD dissertation*, University of Texas, Austin.
100. Hutchison, C. S., 1964, Agabbro-granodiorite association in Singapore Island, *Quarterly Journal of the Geological Society*, London, Vol. 120, pp. 283-297.
101. Idriss, I. M. and Sun, J. I., 1992, *User's Manual for SHAKE91*, Center for Geotechnical Modeling, Department of Civil Engineering, University of California, Davis.

102. Ikegami, R., 1964, On the microseisms observed at Koganei, Tokyo(I): *Zisim Ser.II*, Vol. 17, pp. 68-88 (in Japanese).
103. Inci, G., Yesiller, N. and Kagawa, T., 2003, Experimental investigation of dynamic response of compacted clayey soils, *Geotechnical Testing Journal*, Vol. 26(2), pp. 1-13.
104. Jackson, D. D., 1972, Interpretation of inaccurate, insufficient, and inconsistent data, *Geophys. J. R. Astron. Soc.*, Vol. 28, pp. 97-109.
105. Jivanuwong, S., 1998, Nondestructive Detection of Hollow Heart in Potatoes Using Ultrasonics, *Masters of Science thesis*, Virginia Polytechnic Institute and State University, USA
106. Joh, S-H., 1996, Advances in Interpretation and Analysis Techniques for Spectral Analysis of Surface Waves (SASW) Measurements, *Ph. D. Dissertation*, University of Texas at Austin, Austin, TX.
107. Jones, R. B., 1958, In-situ measurement of the dynamic properties of soil by vibration methods, *Géotechnique*, Vol. 8, No. 1, pp. 1-21.
108. Jong-Sub, L. J-S. and Santamarina, J. C., 2005, Bender Elements: Performance and Signal Interpretation, *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 131, No. 9.
109. Jovicic, V., Coop, M. R. and Simic, M., 1996, Objective criteria for determining $G_{\max}$ from bender element tests, *Géotechnique*, Vol. 46, No. 2, pp. 357-362.
110. Kausel, E. and Roesset, J. M., 1981, Stiffness Matrices for Layered Soils, *Bulletin of the Seismological Society of America*, Vol. 71, No. 6, pp. 1743-1761.

111. Kenneth, B. L. N. and Kerry, N. J., 1979, Seismic waves in stratified half-space, *Geophysics Journal of the Royal Astr Society*, Vol. 57, pp. 557-583.
112. Kennett, B.L.N., 1983, Seismic Wave Propagation in Stratified Media, *Cambridge University Press*, UK, p 342.
113. Kim, Y. H., Song, S-J., Lee, S-S., Lee, J-K. and Hong, S-S., 2001, A study on the couplant effects in contact ultrasonic testing, *The 10th Asia-Pacific conference on Non-Destructive Testing*, Brisbane, Australia.
114. Knight, R. and Nolen-Hoeksema, R., 1990, A laboratory study of the dependence of elastic wave velocities on pore scale fluid distribution: *Geophysical Research Letter*, Vol. 17, No. 10, pp. 1529-1 532.
115. Konno, K. and Kataoka, S., 1999, An estimating method for a phase velocity of Rayleigh wave in multi-layered media, *Third Int. Conf. Seismology and Earthq. Eng.*, Iran, 1999.
116. Kramer, S. L., 1996, Geotechnical Earthquake Engineering, *Prentice Hall*, New York.
117. Krautkrämer, J. and Krautkrämer, H., 1990, Ultrasonic Testing of Materials. *Springer-Verlag Berlin Heidelberg*, New York.
118. Kudo, K., Kanno, T., Okada, H., Özel, O., Erdik, M., Sasatani, T., Higashi, S., Takahashi, M. and Yoshida, K., 2001, Site specific issues for strong ground motions during the Kocaeli, Turkey Earthquake of August 17, 1999, as inferred from array observations of microtremors and aftershocks: *Bulletin of the Seismological Society of America*, Vol. 92, pp. 448-465.
119. Kudo et al., 1976, Observation of 1- to 5-sec microtremors and their application to earthquake engineering, Part IV, *Elucidation of preparative characteristics by use of a temporary array net*: Zisin, Ser II, Vol. 29, pp. 223-337 (in Japanese).

120. Kudo, K, Kanno, T., Okada, H., Sasatani, T., Apostolidis, P. and Raptakis, D., 2003, S-wave velocity structure at euro-seistes, Volvi, Gereece, Determined by array observation of microtremors, *Abstracts for SS04 and SW04 sessions at the IUGG Assembly*, Sapporo, Japan.
121. Kundu, T., 2000, Non-destructive Testing Techniques for Material Characterization, *In modelling in Geomechanics*. Edited by M. Zaman, G. Gioda, G. and J. Booker. John Wiley and Sons Ltd., pp. 267-298.
122. Lai, C. G. and Rix, G. J., 1998, Simultaneous inversion of Rayleigh phase velocity and attenuation for near-surface site characterization, *Research Report, National Science Foundation and U.S. Geological Survey*, Georgia Institute of Technology.
123. Lee, J-S and Santamarina, C., 2004, Bender elements: Performance and Signal Interpretation, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE.
124. Lennartz electronic, 2004, Seismometers General Description, from internet: http://www.lennartz-electronic.de/PDF_documents/Seismometers.pdf
125. Leong, E. C., Yeo, S. H. and Rahardjo, H., 2004, Measurement of wave velocities using ultrasonic test system, *Canadian Geotech. J.*, Vol. 41, pp. 844-860.
126. Leow, J. H., 1962, A glimpse of the sedimentary structure of Singapore, *Malayan Nature Journal*, Vol. 16, pp. 54-60.
127. Li, C. and Nordlunc, E., 1993, Effects of Couplants on Acoustic Transmission, *Rock Mech. Rock Eng.*, Vol. 26, pp. 63-69.
128. Lim, M. S. W., 1974, Biostratigraphy of the Jurong area, Singapore, *Honours thesis*, Department of Geology, University of Malaya.

129. Ling, S., 1994, Research on the estimation of phase velocities of surface waves in microtremors, *Ph. D. Thesis*, Hokkaido University: Japan, (in Japanese).
130. Liu, H. P., Boore, D. M., Joyner, W. B., Oppenheimer, D. H., Warrick, R. E., Zhang, W., Hamilton, J. C. and Brown, L. T., 2000, Comparison of phase velocities from array measurements of Rayleigh waves associated with microtremor and results calculated from borehole shear-wave velocity profiles, *Bulletin of the Seismological Society of America*, Vol. 90, pp. 666-678.
131. Locoss, R.T., Kelly, E. J. and Toksöz, M. N., 1969, Estimation of seismic noise structure using array, *Geophysics*, Vol. 29, pp. 21-38.
132. Lorenz, R. D., 2002, Thermodynamics of Geysers: Application to Titan, *Lunar and Planetary Laboratory*, University of Arizona, Tucson, Arizona, Vol. 156, pp. 176-183.
133. Louie, J. N., 2001, Faster, better: shear-wave velocity to 100 meters depth from refraction microtremor arrays, *Bulletin of the Seismological Society of America*, Vol. 91, No. 2, pp. 347-364.
134. Luke, B. and Calderón-Macías, C., 2007, Inversion of Seismic Surface Wave Data to Resolve Complex Profiles, *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 133, No. 2, pp. 155-165.
135. Luna, R. and Jari. H., 2000, Determination of Dynamic Soil Properties Using Geophysical Methods, *Proceedings of the First International Conference on the Application of Geophysical and NDT Methodologies to Transportation Facilities and Infrastructure*, December, St. Louis, MO
136. Mancuso, C. and Vinale, M., 1988, Propagazione delle onde sismiche: teoria e misura in sito, *Atti Del Convegno Del Gruppo Nazionale Di Coordinamento Per Gli Studi Di Ingegneria Geotecnica*, Monselice, pp. 115-138.

137. Mareshal, J. C., 1985, Inversion of Potential Field Data in Fourier Transform Domain, *Geophysics*, Vol. 50, No. 4, pp. 685-691.
138. Matsuhima, T. and Ohshima, H., 1989, Estimation of underground structures unsin long-period microtremors- Kuromatsunai Depression, Hokkaido: *Butsuri- Tansa, Geophys. Explor.*, Vol. 42, pp. 97-105 (in Japanese).
139. Matsui, T., O'Hara, H., and Ito, T., 1980, Cyclic stress-strain history and shear characteristics of clay. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 106, No. 10 pp. 1101-1120.
140. Matsushima, T. and Okada, H., 1990, Determination of deep geological structures under urban sreass using long-period microtremors, *Geophys. Explor.*, Vol. 43, No. 1, pp. 21-23.
141. Matsuoka, T., Shiraishi, H., and Hachinohe, S., 2003, Estimation of three dimensional S-wave velocity structure using microtremor array observations in the central Kanto Plain, Japan, *Proceedings of the 6th SEGJ international symposium*, Tokyo, pp. 429-436.
142. Matsuoka. T, Umezawa, N. and Makishima, H., 1996, Experimental studies on the applicability of the spatial autocorrelation method for estimation of geological structures using microtremors, *Butsuri-Tansa Geophy. Explor*, Vol. 49, pp. 26-41 (in Japanese).
143. Matthews, M.C. and Clayton, C. R. I, 1993, Influence of intact porosity on the engineering properties of a weak rock, *Geotech Engineering of Hard Soils and Soft Rocks*, Ed., Vol. 1, pp. 693-701, Balkema, Rotterdam.
144. Matthews, M. C., Hope, V. S. and Clayton, C. R. I., 1996, The use of surface waves in the determination of ground stiffness profiles, *Proceedings Institution of Civil Engineers Geotechnical Engineering*, 119, pp. 84-95.

145. Matthews, M. C., Clayton, C. R. I and Owen, Y., 2000, The use of field geophysical techniques to determine geotechnical stiffness parameters *Proceedings Institution of Civil Engineers Geotechnical Engineering*. January, pp. 92-99.
146. Matthias, O., Estelle, S., Cecile, C., Sylvette, B-C., Marc, W., Alexandros S., Frank, S. and Denis, J., 2004, Frequency Wavenumber and Spatial Autocorrelation Methods for Dispersion Curve Determination from Ambient Vibration Recordings, *13th World Conference on Earthquake Engineering*, Vancouver, B.C., Canada, August 1-6, 2004 Paper No. 0946.
147. Mavko, G. and Nolen-Hoeksema, R., 1994, Estimating seismic velocities at ultrasonic frequencies in partially saturated rocks, *Geophysics*, Vol. 59, No. 2, pp. 252-258.
148. Maxwell, A. A. and Fry, Z. B., 1967, A procedure for determining elastic moduli of in situ soils by dynamic wave propagation and dynamic properties of earth material. *Aberquerque, USA*. University of New Mexico Press.
149. McCormick, G. P., 1983, Nonlinear programming, *John Wiley and Sons, Inc.*, New York, N. Y.
150. Menke, W., 1989, Geophysical data analysis: discrete inverse theory. *Academic Press. San Diego*. pp. 260
151. Menzies B. K., 2001, Near-Surface Site Characterization by Ground Stiffness Profiling Using Surface Wave Geophysics, *Instrumentation in Geotechnical Engineering*, H. C. Verma Commemorative Volume. Eds. K. R. Saxena and V. M. Sharma. Oxford & IBH Publishing Co. Pvt. Ltd., New Delhi, Calcutta. pp 43-71.
152. Miller, R. D., Xia, J. Park, C. B. and Ivanov, J., 2001, Shear Wave Velocity Field to Detect Anomalies under Asphalt, *the 52nd Highway Geology Symposium*, May 16, Baltimore, Maryland.

153. Miller, G. F. and Pursey, H., 1954, the field and radiation impedance of mechanical radiators on the free surface of a semi-infinite isotropic solid, *Proc. Roy. Soc.*, 223, pp. 521-541
154. Mitsuji, K., Sugimura, Y. and Karkee, M. B., 2000, Effect of the Spatial Variation in the Hydrogeotechnical Properties on the Dynamic Response of Porous Soil Media, *8th ASCE Specialty Conference on Probabilistic Mechanics and Structural Reliability*, PMC2000-093.
155. Miyakoshi, K., Okada, H. and Ling, S., 1996, A range of wavelength possible to estimate phase velocities of surface waves in microtremors, *Proc. 94th SEGJ Conf.*, pp. 178-182 (in Japanese).
156. Molyneux, J. B. and Schmittz, D. R., 1999, First-break timing: Arrival onset times by direct correlation, *Geophysics*, Vol. 64, No. 5, pp. 1492-1501.
157. Moxhay, A. L., Tinsley, R. D. and Sutton, J. A., 2001, Monitoring of Soil Stiffness During Ground Improvement Using Seismic Surface Waves, Article featured in *Ground Engineering* magazine, pp. 34-37.
158. Nakagawa, K., Soga, K. and Mitchell, J. K., 1996, Pulse transmission system for measuring wave propagation in soils, *J. of Geotechnical Engineering*, Vol. 122, No. 4, pp. 302-308.
159. Nakagawa, K., Soga, K. and Mitchell, J. K., 1997, Observation of Biot compressional wave of the second kind in granular soils, *Géotechnique*, Vol. 47(1), pp. 133-147.
160. Nakamura, Y., 1989, A method for dynamic characteristic estimation of subsurface using microtremors on the ground surface: *Quarterly Reports of the Railway Technical Research Institute*, Tokyo, Vol. 30, pp. 25-33.

161. Nagy, A., 1997, Determination of E-modulus of young concrete with nondestructive method, *ASCE Journal of Materials in Civil Engineering*, Vol. 9 (1), pp. 15-20.
162. Nazarian, S., 1984, In situ determination of elastic moduli of soil deposits and pavement systems by spectral-analysis-of-surface-waves method, Ph.D. thesis, The University of Texas at Austin, Austin.
163. Nazarian, S. and Stokoe II, K. H., 1984, In situ shear wave velocities from spectral analysis of surface waves, *Proceedings of the World Conference on Earthquake Engineering*, Vol. 8, San Francisco, Calif., July 21-28.
164. Nguyen, F., Rompaey, V., Teerlynck, H., Van Camp, M., Jongmans, D. and Camelbeeck, T., 2004, Use of microtremor measurement for assessing site effects in Northern Belgium-interpretation of the observed intensity during the Ms = 5.0 June 11 1938 earthquake: *Journal of Seismology*, Vol. 8, pp. 41-56.
165. Okada H. and Sakjiri, N., 1983, Estimates of an S-wave velocity distribution using long period microtremors, *Geophys. Bull., Hokkaido University*, Vol. 42, pp. 119-143, (in Japanese).
166. Okada, H., Matsushima, K., Moriya, T. and Sasatani, T., 1990, An exploration technique using long-period microtremors for determination of deep geological structures areas, *Geophys. Explor.*, Vol. 43, No. 6, pp. 402-417 (in Japanese).
167. Okada, H., 2003, the Microtremor Survey Method: Society of Exploration Geophysicists of Japan, Translated by Koya Suto, *Geophysical Monograph Series No. 12*, Society of Exploration Geophysicists.
168. Okada, H., Matsuoka, T., Shiraishi, H. and Hachinohe, S., 2003, Spatial autocorrelation algorithm for the microtremor survey method using semicircular arrays: *Proceedings of the 109th SEGJ Conference*, pp. 183-186.

169. Okada, H., 2004, Theory of efficient array observations of microtremors with special reference to the SPAC method, *Exploration Geophysics*, Vol. 37, pp.73-85. Butsuri-Tansa (Vol. 59, No. 1), Mulli-Tamsa (Vol. 9, No. 1)
170. Orozco M. C., 2003. Inversion method for spectral analysis of surface waves (SASW), Ph.D. Thesis, Georgia Institute of Technology.
171. O'Neill, A., 2004, Full waveform reflectivity for inversion of surface wave dispersion in shallow site investigations, *Proc. ISC-2 on Geotechnical and Geophysical Site Characterization*, Viana da Fonseca & Mayne (eds.), Millpress, Rotterdam, pp. 547-554.
172. Park, C. B., 1995, Characterization of Geotechnical Sites by Multi-Channel of Surface Waves (MCASW), *Kansas Geological Survey*, The University of Kansas, USA.
173. Park, C. B., Miller, R. D. and Xia, J., 1999, Multichannel analysis of surface waves, *Geophysics*, Vol. 64, No. 3, pp. 800-808.
174. Parker, R. L., 1994, Geophysical Inverse Theory, *Princeton University Press*, New Jersey, pp. 386.
175. Pezeshk, S. and Zarrabi, M., 2005, A New Inversion Procedure for Spectral Analysis of Surface Waves Using a Genetic Algorithm, *Bulletin of the Seismological Society of America*, Vol. 95, No. 5, pp. 1801-1808.
176. Pezeshk, S., Camp, C. V., Liu, L, Evans Jr., J. M. and He, J., 1998, Seismic acceleration coefficients for west Tennessee and expanded scope of work for seismic acceleration coefficients for west Tennessee phase 2—field investigation. *Final Report, Project Number TNSPRRES116*, January, Prepared for the Tennessee Department of Transportation and the U.S. Department of Transportation Federal Highway Administration, 390 pages.

177. Pezeshk, S., and Liu, L., 2001, Site specific analysis program (SSAP), *Final Report, Project Number TNSPR-RES1036*, Tennessee Department of Transportation in Cooperation with U.S. Department of Transportation Federal Highway Administration.
178. Pfeiffer, D., 1972, Investigations for groundwater, *Singapore Islands, eastern end. No. 2 (Final report)*. U. N. E. C. A. F. E. Natural Resources Division, Mineral Resources Section. Sala Santitham. Bangkok 2. Thailand.
179. Popovics, S., 2001, Analysis of the concrete strength versus ultrasonic pulse velocity relationship, *Materials Evaluation*, Vol. 59 (2), pp. 123-129.
180. Public Works Department (PWD), 1976, Geology of the Republic of Singapore, *Public Works Department*, Singapore.
181. Prasad, M., Zimmer, M. A., Berge, P. A. and Bonner, B. P., 1999, Laboratory measurement of velocity and attenuation in sediments, *Society of exploration geophysics*, Tulsa, UCRL. JRNL 205155..
182. Quan, Y. and Huang, L., 2007, Sound-speed tomography using first-arrival transmission ultrasound for a ring array, *Report to Department of Geophysics*, Stanford University, CA No. 6513-5 Vol. 1.
183. Rao, V. and Ramana, Y. V., 1992, A Study of Progressive Failure of Rock Under Cyclic Loading by Ultrasonic and AE Monitoring Techniques, *Rock Mech. Rock Eng.*, Vol. 25, pp. 237-251.
184. Rapoport, J., Popovics, J. S., Subramaniam, K. V. and Shah, S. P., 2000, The use of ultrasound to monitor the stiffening process of Portland cement concrete with admixtures, *ACI Materials Journal*, Vol. 97 (6), pp. 675-683.
185. Rayleigh, L., 1885, On waves propagated along the plane surface of an elastic solid, *Proceedings London Mathematical Society*, Vol 17, pp. 4-11.

186. Richart, F. E., 1962, Foundation vibrations, *Journal of Soil Mechanics and Foundations Division*, Vol. 127, pp. 863-898.
187. Richart, F. E., Hall, J. R., and Woods, R. D., 1970, Vibrations of soils and strength characteristics of sands, *Soils and Foundations*, Vol. 33, pp. 121-134.
188. Rix, G. J., Stokoe, K. H. and Roesset, J. M., 1991, Experimental study of factors affecting the spectralanalysis-of-surface-waves method, *Research Report 1123-5*, Center for Transportation Research, Bureau of Engineering Research, Univ. Texas Austin.
189. Rix, G. J. and Lai, C. G., 1998, Simultaneous inversion of surface wave velocity and attenuation, *Proceedings 1st International Conference on Site Characterisation*, Atlanta, Georgia, USA, April 1998.
190. Roberts, J. C. Asten, M. W., 2004, Resolving a Velocity Inversion at the Geotechnical Scale Using the Microtremor (passive seismic) Survey Method, *Exploration Geophysics*, Vol. 35, pp. 14-18.
191. Rosenblad, B., Rathje, E. M. and K. H. Stokoe, 2002, Shear wave velocity profiling by the SASW method at selected strong-motion stations in Turkey, *internet*, http://peer.berkeley.edu/lifelines/lifelines_pre_2006/final_reports/2A02a-FR.pdf
192. Saccorotti, G., Almendros, J., Carmon, E., Ibanez, J. and Pezzo, E. D., 1999, Seismic array at deception Island 1998-99 survey -1st phase, *report of the Spanish project 'Sismicidad volcanica, locally regional del area de las Shetland del Sur y Estrecho de Bransfield, ANT98-111'*.
193. Santamarina, J., 2001, Soils and Waves, *John Wiley & Sons, Ltd.*, Chichester, West Sussex, England.

194. Sasatani, T., Yoshida, K., Okada, H., Nakano, O., Kobayashi, T. and Ling, S., 2001, Estimation of deep subsurface structures and observation of strong ground motions in Sapporo urban districts: *J. JSNDS*, Vol. 20, pp. 325-342.
195. Satoh, T., Kawase, H. and Matsushima, S. 2001(a), Estimation of S-wave velocity structures in and around the Sendai Basin, Japan, using array records of microtremors, *Bulletin of the Seismological Society of America*, Vol. 91, pp. 206-218.
196. Satoh, T., Kawase, H. and Matsushima, S., 2001(b), Differences between site characteristics obtained from microtremors, S-waves, P-waves, and codas, *Bulletin of the Seismological Society of America*, Vol. 91, pp. 313-334.
197. Scherbaum, F., Riepl, J., Bettig, B., Ohrnberger, M., Cotton, F. and Bard, P. Y., 2002, Dense array measurements of ambient vibrations in the Grenoble basin to study local site effects, *AGU*.
198. Schneider, J. A., Hoyos, L. Jr., Mayne, P. W., Macari, E. J., and Rix, G. J., 1999, Field and laboratory measurements of dynamic shear modulus of Piedmont residual soils, Behavioral Characteristics of Residual Soils, *GSP 92, ASCE*, Reston, VA, pp. 12-25.
199. Seed, H. B., and Chan, C. K., 1966, Clay strength under earthquake loading conditions. *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 92, No. 2, pp. 53-78.
200. Seed, H. B. and Idriss, I. M., 1970, Soils moduli and damping factors for dynamic response analyses, *Earthquake Engineering Research Center*, Report No. EERC 70-10, University of California, Berkeley, USA.
201. Seet, C. P., 1974, The igneous complex of Pulau Ubin, *Honours thesis*, Department of Geology, University of Malaya.
202. Sen, M. and Stoffa, P. L., 1995. Global Optimization Methods in Geophysical

- Inversion, *Advances in Exploration Geophysics 4*, series editor: A. J. Berkhout, Elsevier.
203. Sezawa, K. and Kanai, K., 1935, Discontinuity in the dispersion curves of Rayleigh waves. *Bulletin of Earthquake Research Institute*, Tokyo, 13, pp 238-243.
204. Sheu, J. C., Stokoe, K. H., Roesset, J. M. and Hudson, W. R., 1986, Applications and limitations of the spectral analysis-of-surface-waves method, *research rept. 437-3F*, Center for Transportation Research, Bureau of Engineering Research, Univ. Texas Austin.
205. Siggins, A. F., 1993, Dynamic Elastic Tests for Rock Engineering, *Comprehensive Rock Engineering*, (Edited by Hudson, J.A.), Chapter 24, Vol. 3, pp. 601-618. Pergamon, Oxford.
206. Silk, M. G., 1984, Ultrasonic Transducer for Nondestructive Testing, *Adam Hilger Ltd.*, Bristol.
207. Snelling, K., 2004, *Personal contact through e-mail*.
208. Stokoe, K. H. II, and S. Nazarian, 1985, Use of Rayleigh waves in liquefaction studies, in *Proc. Conf. on Measurement and Use of Shear Wave Velocity for Evaluating Dynamic Soil Properties*, Denver, Colorado, 1 May 1985, R. D. Woods, (Editors), American Society of Civil Engineers, New York.
209. Stephenson, R. W., 1978, Ultrasonic testing for determining dynamic soil moduli. In *Dynamic geotechnical testing: a symposium*. Edited by M.L. Silver and D. Tiedemann. *American Society for Testing and Materials (ASTM)*, Special Technical Publication STP Vol. 654, pp. 179-195.
210. Stokoe II, K. H., Wright, G. W., James, A. B. and Jose, M. R., 1994, Characterization of geotechnical sites by SASW method, in *geophysical*

characterization of sites, *ISSMFE Technical Committee #10*, edited by R.D. Woods, Oxford Publishers, New Delhi.

211. Stokoe, K. H., Jennie, C. and Milto, T., 2004, SASW Measurement at the NEES Garner Valley Test Site, California, *Data report, Department of Civil Engineering*, The University of Texas at Austin.
212. Sutherland, A. J. and Logan, T. C., 1998, SASW measurement for the calculation of site amplification, *Earthquake Commission Research Project 97/276: unpub. Central Laboratories Report 98-522422*, Lower Hutt, New Zealand, pp. 22.
213. Szilard, J., 1982, Techniques Using Dry Coupling or Non-Contact Coupling. In ultrasonic testing. Edited by J. Szilard, *John Wiley & Sons Ltd.*, New York, pp. 381-409.
214. Tai, S. A., 1972, Geology of Eastern Singapore Island with special emphasis on the Old Alluvium, *Honours thesis*, Department of Geology, University of Malaya.
215. Tamura, S., Tokimatsu, K., Abe, A. and Sato, M., 2002, Effects of air bubbles on B-value and P-wave velocity of a partly saturated sand, *Soils and Foundation*, Vol. 40(1), pp.121-129.
216. Tarantola, A., 1987, Inverse Problem Theory: Methods for Data Fitting and Model Parameter Estimation, *Elsevier*.
217. Tarantola, A. and Valette, B., 1992, Generalized nonlinear inverse solved using the least squares criterion, *Rev. Geophysics*, Vol. 20, pp. 219-232.
218. Tatsuoka, F., Teachavorasiuskun, S., Dong, J., Kohata, Y. and Sato, T., 1994, Importance of measuring local strain in cyclic triaxial tests on granular soils, *Dynamic Geotechnical Testing II*, ASTM STP 1213, pp. 288-302.

219. Tatsuya, N. and Ryohei, N., 2002, Determination of subsurface structure of Tottori Plain using microtremors and gravity anomaly, *Journal of Natural Disaster Science*, Vol. 24, No. 1, pp. 1-13.
220. Thum, K. L. and Yong, C. H., 1987, Review of the geotechnical properties of Singapore geology, *Final year project*, NTU.
221. Thomson, W. T., 1950, Transmission of elastic waves through a stratified solid, *J. Appl. Phys.* Vol. 21, pp. 89-93.
222. Tokimatsu, K., Kuwayama, S., Tamura, S. and Miyadera, Y., 1991, Vs determination from steady state Rayleigh wave method, *Soils and Foundation*, Vol. 31(2), pp. 153-163.
223. Tokimatsu, K. and Miyadera, Y., 1992, Characteristics of Rayleigh waves in microtremors and their relation to underground structures, *J. Struct. Constr. Engng.*, AIJ, pp. 439, pp. 81-87 (in Japanese).
224. Tokimatsu, K., Shinzawa, K., and Kuwayama, S., 1992, Use of short-period microtremors from Vs profiling, *J. Geotech. Engrg.*, Vol. 118, pp. 1544-1558.
225. Tokimatsu, K., 1995, Geotechnical site characterization using surface waves, *Proceedings 1st Int. Conf. on Earthquake Geotechnical Eng.*, IS-Tokyo, Balkema, Rotterdam, pp. 1333-1368.
226. Tokimatsu, K., 1997, Geotechnical site characterization using surface waves: in Ishihara (ed.), *Earthquake Geotechnical Engineering*: Balkema.
227. Toksöz, M. N. and Lacoss, R.T., 1968, Microseisms-Mode Structure and Sourcees, *Science*,. 159, pp. 159-177.
228. Toköz, M. N., 1964, Microseisms and an attempted application to exploration, *Geophysics*, Vol. 29, pp. 154-177.

- 229. Tsukamoto, Y., Ishihara, K., Nakazawa, H., Kamada, K. and Huang, Y., 2002, Resistance of Partly Saturated Sand to Liquefaction with Reference to Longitudinal and Shear Wave Velocities, *Soils and Foundation*, Vol. 42, No. 6.
- 230. Veijayaratnam, M., Poh, K. B. and Tan, S. L., 1993, Seismic velocities in Singapore soils and some geotechnical applications, *Proceedings 11th Southeast Asian Geotechnical Conference*, Singapore, pp. 813-818.
- 231. Velea, D., Shields, F. D. and Sabatier, J. M., 2000, Elastic Wave Velocities in Partially Saturated Ottawa Sand: Experimental Results and Modeling, *Soil Science Society America. J.* Vol. 64, pp. 1226-1234.
- 232. Viggiani, G. and Atkinson, J. H., 1995, Stiffness of fine-grained soil at very small strains, *Géotechnique*, Vol. 45(2), pp. 249-265.
- 233. Vrettos, C., 1990, In-plane vibrations of soil deposits with variable shear modulus: I. Surface waves. *Int. J. Numer. Anal. Methods Geomech.* Vol. 14, pp. 209-222.
- 234. Vrettos, C. and Prange, B., 1990, Evaluation of in-situ effective shear modulus from dispersion measurements, *Journal of Geotechnical Engineering*, Vol. 116, No. 10, pp.1581-1585
- 235. Wallace, P. R., 1984, Mathematical analysis of physical problems, *Dover Publications, Inc.*, New York, pp. 616.
- 236. White, J. E., 1955, Seismic waves: radiation, transmission and attenuation. *New York: McGraw-Hill.*
- 237. Wiggins, R. A., 1972, The general linear inverse problem: Implication of surface waves and free oscillations for earth structure, *Rev. Geophys. Space Phys.*, Vol. 10, pp. 251-281.

238. Williams, T. P., Gucunski, N., 1995, Neural networks for back calculation of moduli from SASW test, *J. of Computing in Civil Eng.*, Vol. 9 (1), ASCE, pp. 1-8.
239. Wong, I. F. T., 1960, The petrology and structure of the sedimentary rocks of southwest Singapore, *Honours thesis*, Department of Geology, University of Malaya.
240. Woods, R. D., 1968, Screening of surface waves in soils, *Journal of Soil Mechanics and Foundation Division*, ASCE, Vol. 94, No. SM4, pp. 951-979.
241. Wyllie, M. R. J., Gregory, A. R. and Gardner, L. W., 1956, Elastic Wave Velocities in Heterogeneous and Porous Media, *Geophysics*, Vol. 21, No. 1, pp. 41-70.
242. Xia, J., Miller, R. D. and Park, C. B., 1999, Estimation of near-surface shear wave velocity by inversion of Rayleigh waves, *Geophysics*, Vol. 64, No. 3, pp. 691-700.
243. Yamamoto, H., 2000, An experiment for estimating phase velocities of Love waves from three-component microtremor array observation: Butsuri-Tansa, *Geophys. Explor.*, Vol. 53, pp. 153-166 (in Japanese with English abstract).
244. Yamanaka, H., Furuya, S., Nozawa, T., Sasaki, T., and Takai, T., 1995, Array measurements of long-period microtremors in the Kanto plain, *Jour. Struct. Constr. Eng.*, Vol. 478, pp. 99-105 (in Japanese).
245. Yamanaka, H. and Ishida, H., 1995, Phase velocity inversion using genetic algorithm, *J. Struct. Constr. Eng.*, AIJ, No.468, pp. 9-17 (in Japanese with English abstract).

- 246. Yasuia, Y. and Noguchib, T., 2004, Soil Profile Confirmation Observation through Microtremor, *Proceedings Third UJNR Workshop on Soil-Structure Interaction*, March 29-30, 2004, Menlo Park, California, USA.
- 247. Yeo, S. H., 2005, Relationship of Stiffness and Damping Ratio with Strain for Residual Soils. *Ph. D. dissertation*, School of Civil and Environmental Engineering, Nanyang Technological University, Singapore.
- 248. Yoseph, B. C. and Ajit, K. M., 1989, Ultrasonic Inspection, *Metals Handbook*, prepare under the direction of ASM handbook committee, Metals Park, Ohio, America Society for Metals.
- 249. Yuan, D. and Nazarian, S., 1993, Automated surface wave method: inversion technique, *Journal of Geotechnical Engineering*, ASCE, Vol. 119, pp. 1112-1126.
- 250. Zhang, S. X. and Chan, L. S., 2003, Possible Effects of Misidentified Mode Number on Rayleigh Wave Inversion, *Journal of Applied Geophysics*, Vol. 53, pp. 17-29.
- 251. Zhang, S. X., Chan, L. S., Chen, C. Y., Dai, F. C., Shen, X. K. and Zhong, H., 2003, Apparent Phase Velocities and Fundamental-Mode Phase Velocities, *Soil Dynamics and Earthquake Engineering*, Vol. 23, pp. 563-569.
- 252. Zimmer, M., Prasad, M., and Mavko, G., 2002, Pressure and porosity influences on V_p - V_s ratio in unconsolidated sands, *Leading Edge*.
- 253. Zywicki, D. J., 1999, Advanced signal processing methods applied to engineering analysis of seismic surface waves, *Ph.D. Dissertation*, Georgia Institute of Technology, Atlanta, Georgia.

Appendix A- Borehole Log

Figure A1 Borehole log for Site 1

Figure A3 Borehole log for Site 3

Figure A4 Borehole log for Site 4

Figure A5 Borehole log for Site 5

Figure A6 Borehole log for Site 6

Figure A7 Borehole log for Site 7

Appendix B- Matlab Script File for SPAC Method

```
% READING OF DATA
clear all;
[AA(:,1), AA(:,2), AA(:,3), AA(:,4), AA(:,5), AA(:,6)] =
textread('F:\914JWest\914Mic\914con2hzb5.txt','%f, %f, %f, %f, %f,
%f',-1);

A(:,1) = AA(:,1) ;
A(:,2) = AA(:,2) ;
A(:,3) = AA(:,3) ;
A(:,4) = AA(:,4) ;

npt=length(AA);
n = ceil(log2(npt));
n = 2^npt(1,1);
n = 2^14;
n_upp = n;
max_j = floor(npt/n);

ft = 5 ;
fr=2457.6/5;
fz=ft*fr;
fs = fz/6;
ts = 1/fs;
f =(fs/n_upp)*(0:n_upp-1);
f1=f';
f2=f1(1:100);
maxt = (length(AA)-1)*ts;
t=0 : ts : maxt;
for j=0:max_j-1;
    nn = j*n;

%Windowing of the data

    x111(1:n) = A(nn+1:nn+n,1);
    x222(1:n) = A(nn+1:nn+n,2);
    x333(1:n) = A(nn+1:nn+n,3);
    x444(1:n) = A(nn+1:nn+n,4);

    A1(j+1,:) = x111;
    A2(j+1,:) = x222;
    A3(j+1,:) = x333;
    A4(j+1,:) = x444;

%FFT (Fast Fourier Transform)

    fftx1= fft(x111,n);
    fftx2= fft(x222,n);
    fftx3= fft(x333,n);
    fftx4= fft(x444,n);

% wn= 2/fs; %filter
```

```
% [e,a]=butter(1,wn);
% fftx1 =filtfilt(e,a,fftx1);
% fftx2 =filtfilt(e,a,fftx2);
% fftx3 =filtfilt(e,a,fftx3);
% fftx4=filtfilt(e,a,fftx4);
%

%PSD (Power Spectral Density)
b = ones(1,100)/100;
b1 = ones(1,100)/100;

    ppy1= fftx1.*conj(fftx1)/n;
    ppy2= fftx2.*conj(fftx2)/n;
    ppy3= fftx3.*conj(fftx3)/n;
    ppy4= fftx4.*conj(fftx4)/n;

% 10 point averaging filter

    ppy1 = filtfilt(b,1,ppy1 );
    ppy2 = filtfilt(b,1,ppy2 );
    ppy3 = filtfilt(b,1,ppy3 );
    ppy4 = filtfilt(b,1,ppy4 );

    ppy1b1 = filtfilt(b1,1,ppy1 );
    ppy2b1 = filtfilt(b1,1,ppy2 );
    ppy3b1 = filtfilt(b1,1,ppy3 );
    ppy4b1 = filtfilt(b1,1,ppy4 );

%CSD (Cross Spectral Density)

    pxy12=fftx1.*conj(fftx2)/n;
    pxy23=fftx2.*conj(fftx3)/n;
    pxy31=fftx1.*conj(fftx3)/n;

    pxy12 = filtfilt(b,1,pxy12 );
    pxy23 = filtfilt(b,1,pxy23 );
    pxy31 = filtfilt(b,1,pxy31 );

    pxy12b1 = filtfilt(b1,1,pxy12 );
    pxy23b1 = filtfilt(b1,1,pxy23 );
    pxy31b1 = filtfilt(b1,1,pxy31 );

    a12=real(pxy12);
    a23=real(pxy23);
    a31=real(pxy31);

    a12b1=real(pxy12b1);
    a23b1=real(pxy23b1);
    a31b1=real(pxy31b1);

    b12=sqrt(ppy2.*ppy1);
    b23=sqrt(ppy3.*ppy2);
```

```

b31=sqrt(pyy1.*pyy3);

b12b1=sqrt(pyy2b1.*pyy1b1);
b23b1=sqrt(pyy3b1.*pyy2b1);
b31b1=sqrt(pyy1b1.*pyy3b1);

c12=a12./b12;
c23=a23./b23;
c31=a31./b31;

c12b1=a12b1./b12b1;
c23b1=a23b1./b23b1;
c31b1=a31b1./b31b1;

avg=(c12+c23+c31)/3;
avgb1=(c12b1+c23b1+c31b1)/3;
%
n1=n/2;

subplot(3,1,1);

switch j
    case 0;
        plot(f(1:n1),avg(1:n1), 'r');axis([0 50 -0.5 1]);
        hold on;
    case 1
        plot(f(1:n1),avg(1:n1), 'b'),grid;axis([0 50 -0.5 1]);
    case 2
        plot(f(1:n1),avg(1:n1), '-r'),grid; axis([0 50 -0.5 1]) ;
    case 3
        plot(f(1:n1),avg(1:n1), '-b'),grid; axis([0 50 -0.5 1]);
    case 4
        plot(f(1:n1),avg (1:n1), '--r'),grid;axis([0 50 -0.5 1]);
    case 5
        plot(f(1:n1),avg (1:n1), '--b'),grid; axis([0 50 -0.5 1]);
    case 6
        plot(f(1:n1),avg (1:n1), '-k'),grid; axis([0 50 -0.5 1]);
    case 7
        plot(f(1:n1),avgb1(1:n1), 'd'),grid; axis([0 50 -0.5 1]);

        hold off;
        legend(
'window1','window2','window3','window4','window5','window6','window7','
window8');
        end
        subplot(3,1,2);
    switch j
        case 0;
            plot(f(1:n1),avgb1(1:n1), 'r'),grid;;axis([0 50 -0.5 1]);
            hold on;
        case 1
            plot(f(1:n1),avgb1(1:n1), 'b'),grid;axis([0 50 -0.5 1]);

        case 2
            plot(f(1:n1),avgb1(1:n1), '-r'),grid; axis([0 50 -0.5 1]) ;

```

```
case 3
    plot(f(1:n1),avgb1(1:n1),'-b'),grid; axis([0 50 -0.5 1]);
case 4
    plot(f(1:n1),avgb1(1:n1),'--r'),grid;axis([0 50 -0.5 1]);

    case 5
        plot(f(1:n1),avgb1(1:n1),'--b'),grid; axis([0 50 -0.5 1]) ;
case 6
    plot(f(1:n1),avgb1(1:n1),'-k'),grid; axis([0 50 -0.5 1]);
% case 7
% plot(f(1:n1),avgb1(1:n1),'d'),grid;axis([0 50 -0.5 1]);
    hold off;

legend('window1','window2','window3','window4','window5','window6','win
dow7','window8');

end

    avg1(j+1,:) = avg;
    avgg =avg1';
    avglb1(j+1,:) = avgb1;
    avggbl=avgb1';

end

avgfinal= avglb1(5,:);
    avgfinal=avgfinal';
subplot (3,1,3);
plot(f(1:n1),avgfinal(1:n1),'-k'),grid; axis([0 50 -0.5 1]);
% DATA TRANSFERING TO EXCEL FILES
    wklwrite('avgfinal',avgfinal);
```

Appendix C- Laboratory Calibration Curves

Figure C1 Load cell calibration curve

Figure C2 Pore-water pressure transducer calibration curve

Figure C3 LVDT calibration curve

Figure C4 LDT (A) calibration curve

Figure C5 LDT (B) calibration curve

Figure C6 Proximity transducer (A) calibration curve

Figure C7 Proximity transducer (B) calibration curve

Appendix D
CU test results Shear modulus versus shear strain
(Enlarged)

Figure D1 - Shear modulus versus shear strain for soil specimen S3D10.15
(Enlarged Figure 7.25 d)

Figure D2 - Shear modulus versus shear strain for soil specimen S3D16.6
(Enlarged Figure 7.26 d)

Figure D3 - Shear modulus versus shear strain for soil specimen S4D12.3
(Enlarged Figure 7.27 d)

Figure D4 - Shear modulus versus shear strain for soil specimen S5D8.9
(Enlarged Figure 7.30 d)

Figure D5 - Shear modulus versus shear strain for soil specimen S5D14.9
(Enlarged Figure 7.31 d)

Figure D6 - Shear modulus versus shear strain for soil specimen S5D17.85
(Enlarged Figure 7.32 d)

Figure D7 - Shear modulus versus shear strain for soil specimen S5D20.85
(Enlarged Figure 7.33 d)

Figure D8 - Shear modulus versus shear strain for soil specimen S6D12.35
(Enlarged Figure 7.34 d)

Figure D9 - Shear modulus versus shear strain for soil specimen S7D12.5
(Enlarged Figure 7.35 d)