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Use of Surface Wave Techniques for the Identification of Shallow Rock

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USE OF SURFACE WAVE TECHNIQUES FOR THE IDENTIFICATION OF
SHALLOW ROCK

BY
FRIEDERIKE WULFF

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE
REQUIREMENTS FOR THE DEGREE OF
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OF
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ABSTRACT

Knowing the depth to bedrock is important in designing and constructing foundations for buildings and transportation infrastructure. Rock is typically a strong and competent foundation material, however if it is close to the ground surface it can be costly to remove. This is especially true if the presence of shallow rock is not known until construction. In many transportation projects where geotechnical borings are widely spaced along a road alignment, areas of shallow rock can be easily missed until construction of drainage structures beneath the road. More research is needed on the viability of cost effective tools to identify the presence of shallow rock before construction.

Non-destructive evaluation (NDE) techniques to characterize the stiffness of soils may be a good tool for this problem. Spectral Analysis of Surface Waves (SASW) is a wave propagation method in which vertical shear wave velocity profiles and elastic moduli of subsurface layers of soil and rock can be estimated. The profiles are obtained from the analysis of surface wave data, usually generated from a falling weight and measured by an array of two or more geophones.

The objective of this thesis is to evaluate the efficiency of the SASW system for use in transportation projects in Rhode Island. It will focus on the identification of shallow rock for aid in construction of drainage structures.

SASW tests were performed at five different locations. The resulting shear wave velocity profiles were analyzed and evaluated for the following: 1) identification of shallow rock, 2) global vs. array approach for modeling the dispersion curve and 3) influence of the initial layer thickness.

The results showed that it was possible to identify the presence of rock layers

with the SASW system. However, the SASW system was not that accurate in identifying the depth to rock. A key lesson from this study is that the process to estimate the shear velocity requires considerable experience and personal judgment. There are many factors that affect the prediction of shear wave velocities, including the selection of data for analysis (masking), the type of approach for modeling the dispersion curve, and the steps used in the inversion.

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CHAPTER 1

Introduction

1.1 Problem Statement

The identification of shallow rock and the depth of bedrock is an important factor in the design and construction of transportation infrastructure and the foundation of buildings. Rock is typically a great foundation material, however if it is located close to the ground surface and its existence is unknown it can be costly to remove. Therefore, prior testing and the investigation of shallow rock is an essential part in many transportation projects such as road constructions. The most widely accepted method for the identification of shallow rocks and mapping of the surface of bedrock is drilling. However, drilling is a time consuming and destructive method, and in many transportation projects where geotechnical borings are widely spaced along a road alignment, areas of shallow rock can be easily missed.

In contrast, non-destructive evaluation (NDE) techniques, such as seismic refraction, Spectral Analysis of Surface Waves (SASW) or Multi-channel Analysis of Surface Waves (MASW), have been developed for geotechnical surveys. All three techniques are wave propagation methods in which vertical shear wave velocity profiles and elastic moduli of subsurface layers of soil and rock can be estimated. The shear wave velocity profiles can be obtained from the analyses of measured surface wave data, usually generated from a falling weight and measured by an array of two or more geophones. Due to the small amount of basic equipment and simple setup for testing, the three NDE techniques are relatively easy and inexpensive to implement. However, interpretation of the results is not trivial and requires considerable expertise. Specific to this thesis, there has been only a limited amount of research concerning the use of NDE techniques for the identification of shallow rock.

A commercial SASW system has been provided to the University of Rhode Island by Dr. James Kaklamanos of Merrimack College. This provides an opportunity to both investigate shallow rock sites, and also to continue previous research at URI concerning the collection of shear wave velocity data using SASW on coastal beach sites in Rhode Island where coastal erosion occurred.

1.2 Objectives

The first objective of this study is to evaluate the effectiveness of the SASW system for the identification of shallow rock. The shear wave velocity of rock is typically 5-10 times the velocity of soil and may be identifiable in the upper few meters of a soil profile. Four locations at which shallow rock exist will be tested and compared to existing boring logs and previous test results.

In addition to the two main objectives, an ongoing project at the University of Rhode Island (URI) of collecting shear wave velocity data using SASW on coastal beach sites in Rhode Island where coastal erosion occurred is continued in this study. Three beach sites will be examined. At one of these sites, previous SASW tests have been conducted by Groenewold (2015) which gives the opportunity to compare the results and identify possible changes. The aim of the data collection is to obtain insight into erosion at these sites.

1.3 Organization of Thesis

The thesis is organized to give on the one hand an overview about theoretical information of seismic investigation methods and their application in the investigation of shallow rock and on the other hand introduces the procedure and results of tests with the Spectral Analysis of Surface Waves (SASW) system.

Chapter 2 includes a literature review about seismic wave theory and three seismic investigation methods. The SASW system is introduced in this chapter.

Chapter 3 focuses on the application of the three seismic investigation methods for the identification of shallow rock. The commercial SASW system by Olson Instruments, Inc. is discussed in detail in Chapter 4 which includes the equipment and procedure used in the field as well as the software used for data processing. Chapter 5 presents information about the different sites at which tests were conducted. Details about the reason of the selection, the location and the site conditions are provided. The test results are presented in Chapter 6. The last chapter, Chapter 7, summarizes and compares all the results.

CHAPTER 2

Literature Review

2.1 Elastic Wave propagation

When a force is applied to a body, there are resulting stresses (i.e. compression, tension, or shear) and strains (i.e. volumetric, shear) (Bormann et al., 2012). In an elastic material, the relationship between stresses and strain is often linear. Due to their complex mechanical behavior, soils and rocks are generally not described as linear elastic materials. However, for small and rapidly applied forces, a linear elastic constitutive model may be appropriate. In these cases, the applied forces propagate through the material as stress waves. Two different kind of waves are generated, body waves and surface waves. While body waves can propagate in infinite and unbounded mediums, surface waves only exist if a free surface or surface boundary is defined. In the following sections, body and surface waves will be explained in more detail.

2.1.1 Body Waves

Body waves can travel in a homogeneous, infinite and unbounded continuum such as the interior of the earth. Two different kind of body waves exist, primary waves (P-waves) and secondary waves (S-wave). P-waves, also known as compression or longitudinal waves, propagate in compression or extension movements (Bozorgnia and Bertero, 2004). The particle motion of P-waves travel in the same direction as the wave propagation. The particle motion of S-waves, also known as shear or transverse waves, is perpendicular to the direction of propagation. Since fluids and gases are not able to support shear stresses, shear waves only propagate in solid materials. Both wave motions are illustrated in Figure 1.

Figure 1: Deformations produced by compression waves(top) and shear waves (bottom) (Kramer, 1996)

The velocity of propagating body waves depends on the density and the elastic properties of the medium. These parameters can be characterized by the density ρ and Lamé's constants λ and μ (or shear modulus, G). P-waves are the fastest traveling body wave. Since P-waves travel through solids by compression and soils and rocks are nearly incompressible, the waves can propagate at relatively high velocities. Additionally, P-waves travel through pore water due to its incompressibility. In contrast, S-waves can not travel through water and the particle movement is orthogonal to the direction of propagation. Hence, the velocity of the S-waves is slower. The velocities of the P-waves (v_P) and S-waves (v_S) can be obtained using

$$v_P = \sqrt{\frac{\lambda + 2G}{\rho}} \quad (1)$$

$$v_S = \sqrt{\frac{G}{\rho}} \quad (2)$$

2.1.2 Surface Waves

In an elastic half-space, waves can travel along the medium's surface in addition to the propagation within the body of the medium. Surface waves are generated by the interaction between compression and shear waves (P- and S-waves) and propagate along the interface between layers. Since shearing can only occur in solids, at least one of the two media forming the interface has to be a solid material for surface waves to exist. Scientists, including Rayleigh, Love, Stoneley and Scholte, investigated the existence and propagation of surface waves along different boundaries. Rayleigh and Love waves travel along a solid-air interface, Scholte waves are generated on a solid-fluid boundary and Stoneley waves can be found between solid-solid interfaces. The focus of this thesis lies on the surface waves along a solid-air interface and particularly Rayleigh waves.

Rayleigh waves, named after Lord Rayleigh who first investigated them in 1885, result from the interference of P- and S_v -waves (vertical shear waves) with the medium's surface. Figure 2 shows the particle motion associated with Rayleigh waves.

Figure 2: Deformations produced by Rayleigh Waves (top) and Love-waves (bottom) (Kramer, 1996)

Near the surface, the particle motion is elliptical and retrograde with respect to the direction of propagation (Ryden, 2004). Particle motion decreases with increasing depth as can be seen in Figure 3. The energy of Rayleigh waves spreads cylindrically from a point load at the surface, which leads to a reduction of the amplitude with travel distance. However, the rate of geometric attenuation is much lower than for body waves whose energy distribution occurs in a hemi-spherical direction. Therefore, for distances of one to two wavelengths from the source, the wave fields are dominated by Rayleigh waves and body waves can be neglected (Foti et al., 2015). Orthogonal to the propagation direction no energy is transported, which leads to an exponential decay of the displacement field and wave amplitude (Figure 3). Within a depth of one wavelength into the soil, most of the energy associated with surface wave motion dissipates.

Figure 3: Particle motion and amplitude of Rayleigh waves in homogeneous, linear elastic half space (Pei, 2007)

In a vertical homogeneous half space, Rayleigh waves with different wavelengths penetrate into different depths of the subsoil. Waves with larger wavelengths travel deeper into the soil while waves with shorter wavelengths only penetrate in the near surface area Figure 4 (left). Since the material is homogeneous in the vertical direction, the surface wave velocity remains constant for different wavelengths.

Soils are inhomogeneous, and the material properties (e.g. density, shear wave velocity) vary with depth (Pei, 2007). Equivalent to surface waves in homogeneous soils, waves with long wavelengths travel deeper into the soil than waves with short wavelengths. However, since the soil is inhomogeneous the velocity with which each wavelength is propagating depends on the mechanical properties of the respective layer (Figure 4 (right)). Therefore, the different frequency components of the surface wave travel at different velocities, and these are called phase velocities. This phenomenon is called dispersion, and the relationship between frequency and phase velocity of a Rayleigh wave is called a dispersion curve.

Figure 4: Dispersion of surface waves in homogeneous subsoil(left) and inhomogeneous subsoil (right) (Foti et al., 2015)

Different site investigation techniques use the phenomenon of dispersion to gain information about subsurface conditions, and three of those methods will be described here. The first method is seismic refraction which analyzes the travel time of seismic energy after traveling through the ground along refracted ray paths. The other two methods, Spectral Analysis of Surface Waves (SASW) and Multi-Channel Analysis of Surface Waves (MASW), focus on the propagation of surface waves and shear wave velocity profiles.

2.2 Seismic Refraction

The seismic refraction method is the analysis of artificially created seismic waves. It comprises sending an impulse of energy into the ground which leads to a vibration of particles in the direction of seismic wave propagation and a transmission of mechanical energy (Lukic et al., 2013). The propagating body

waves (mainly P-waves) get reflected and refracted at the subsurface interfaces and return to the surface where their arrival time will be recorded by a line or array of geophones laid out at the surface.

The method is based on Snell's law and the fact that the subsurface consists of layers with different elastic properties. P-waves that are approaching an interface at a slanting angle, termed obliquely incident, get reflected and refracted (Figure 5). The refracted ray travels through the lower layer with a change of direction. This phenomena is analogous to the behavior of a light ray obliquely incident on the boundary between for example air and water (Kearey et al., 2002). In Snell's law the ray parameter is defined by $p = \sin\theta_1/v$, where i is the angle of inclination of the ray and v the velocity the ray is traveling with. Since the ray parameter stays constant over the depth, the following assumption can be made:

$$\frac{\sin\theta_1}{v_1} = \frac{\sin\theta_2}{v_2} \quad (3)$$

Figure 5: Snell's law of refraction and reflection (Kearey et al., 2002)

The seismic refraction test uses an impact source to generate the seismic waves and geophones in an array to register them in defined distances. Figure 6

illustrates a schematic diagram of a seismic refraction test in a two-layered model. Additionally, the propagation path of the seismic waves can be seen. The seismic waves can reach the receivers in three ways with different arrival times and travel paths during a test.

Figure 6: Direct and refracted wave rays (Igel, 2005)

The first one is the direct wave which travels straight along the surface with a constant velocity (here v_1). At a distance x the travel time can be computed with:

$$t_{dir} = \frac{x}{v_1} \quad (4)$$

In the second way, the seismic waves get reflected at the interface and travel back to the surface. The calculation of the travel time is only dependent on the velocity of the first layer but slightly more complicated than for the direct wave. It can be determined with the equation of a hyperbola:

$$t_{refl} = \frac{(x^2 + 4h^2)^{1/2}}{v_1} \quad (5)$$

The third wave form which reaches the geophones is the refracted or head wave. First it travels through the upper layer with the velocity v_1 , gets refracted at the interface between the two layers, travels horizontally along the underside of the interface with the velocity of the second layer v_2 and back up to the surface

with the velocity v_1 . This can only happen if the emergence angle θ_2 becomes a particular critical angle θ_c and is equal to 90° . The travel time of a refracted ray is the sum of the travel times of all three paths. It can be expressed as the a refractor travel time plus delay times in which the wave travels down to the interface and back up to the surface. The following equations state two general forms of how the total travel-time of refracted waves can be computed, where h is the depth of the interface.

$$t_{refr} = \frac{x}{v_2} + \frac{2h\cos\theta_c}{v_1} \quad (6)$$

or

$$t_{refr} = \frac{x}{v_2} + t_i \quad (7)$$

where t_i is the intercept time given by:

$$t_i = \frac{2h(v_2^2 - v_1^2)^{1/2}}{v_1v_2} \quad (8)$$

All travel times can be plotted in travel-time curves or time-distance curves. Figure 7 shows plots of the direct and refracted wave. The travel time of the reflected wave is not shown because it is only of minor interest since the distances are so large that the reflected wave has merged with the direct wave (Igel, 2005).

Figure 7: Travel time arrival plot (Kearey et al., 2002)

As mentioned before, the travel time of the refracted wave is divided into two oblique segments. One segment is traveling at v_1 , a lower velocity, and the other one traveling horizontal segment at a higher velocity, v_2 . With increasing distance x , the horizontal ray of the refracted wave traveling at v_2 encounters the possibility to overtake the direct wave traveling at v_1 . Hence, the refracted wave arrives at the receiver before the direct wave. The offset at which the direct waves will be overtaken by the refracted waves is referred to as x_{cross} . In a distance greater than x_{cross} the refracted waves might be recorded first at the receiver (Dentith and Mudge, 2014).

The depth to the interface h can be obtained from the intercept time by extending the branch of the refracting wave to zero offset ($x=0$). The travel time at that location is only dependent on the travel time of the oblique segments and therefore only on the intercept time. By rearranging equation 8, the depth can be obtained:

$$h = \frac{t_i v_1 v_2}{2\sqrt{v_2^2 - v_1^2}} \quad (9)$$

Typical applications for the seismic refraction method include the following (RSK Geophysics, 2013) (Geometrics, 2016):

1. Estimation of depth of bedrock
2. Mapping of depth of water table
3. Prediction and calculation of rock types and quality
4. Identification and mapping of faults or sinkholes

2.3 Spectral Analysis of Surface Waves

The most commonly used surface wave for the SASW method are Rayleigh waves. By measuring the Rayleigh waves with at least two receivers, typically geophones, the SASW method can determine shear wave velocity profiles of geotechnical sites. SASW tests are based on a two-receiver configuration (Foti et al., 2015). The distance between the two geophones and the distance to the impact source have to be specified. The whole process to create shear wave velocity profiles can be divided into three main steps: Acquisition, Signal processing and Inversion processes.

2.3.1 Acquisition of experimental data

In the first step (acquisition) the surface waves will be generated by an impact source, monitored by receivers and the signal data recorded by dynamic signal analyzer. A typical set up can be seen in Figure 8.

Figure 8: SASW set up (Greene, 2011)

Different types of impact sources can be used to generate surface waves, for example sledge hammers, bulldozers and large weights. Each of these sources produces waves with different frequencies and wavelengths. Heavy sledge hammers and falling weights create waves with low frequencies and long wavelengths which travel deep into the subsoil. Small sledge hammers generate waves with higher frequencies and therefore shorter wavelengths which only propagate through the layers near the surface. The propagation of surface waves with different frequencies/wavelengths is illustrated in Figure 9.

Figure 9: SASW set up (Greene, 2011)

For the SASW method, two receivers are used to measure the propagating surface waves. The most commonly used receivers are vertically orientated accelerometers or geophones. The geophones are connected to a dynamic signal analyzer to record the measured data. The geophones have to be set up in a linear array with a certain spacing between each other and the source and the first geophone. Since it is more reliable to measure waves with shorter wavelengths with smaller spacings and waves with longer wavelengths with bigger spacings, the spacings have to be changed within the SASW testing.

Two different set-up configurations for SASW testing exist: 1) common receivers midpoint geometry and 2) common source geometry (Figure 10). For the common receivers midpoint geometry (CRMP), the spacing between the two geophones is adjusted around a fixed point located in the middle of the geophones. In this case the position of both geophones and the source change with increasing spacings. In the common source geometry set-up the source is fixed to one location and only the two geophones change their position with increasing spacing. Previous studies by Nazarian and Stokoe (1985) investigated the effect of the two set-up configurations and suggested that the common receivers midpoint geometry

yields more reliable results. Therefore, the common receivers midpoint geometry was used for all tests in this study.

Figure 10: a) Common receivers midpoint geometry (CRMP) (Goh et al., 1994) and b) Common source geometry (CSR) (Nazarian and Stokoe, 1985)

2.3.2 Signal Processing

From a SASW test a time signal is recorded for each receiver spacing. The receiver closer to the source will record the signal earlier than the receiver located further away which leads to a time difference between them. The generated spectrum of Rayleigh waves consists of a wide range of frequencies which affect different parts of the soil profile (Stokoe et al., 1994). In the second step (processing), the time signals of each receiver spacing are transformed into the frequency domain and the different frequency components are separated using Fourier analysis. For each frequency f , the phase difference $\phi(f)$ between the receivers can be calculated. Following, the travel time $t(f)$ of each frequency between the two receivers can then be obtained by the following equation:

$$t(f) = \frac{\phi(f)}{2\pi f} \quad (10)$$

The phase velocity of the Rayleigh waves depends on the travel time and the distance between the receivers ($\Delta d = d_2 - d_1$) and can be calculated by

$$V_R = \frac{\Delta d}{t(f)} \quad (11)$$

The wavelength of the considered frequency can be obtained by

$$\lambda_R = \frac{V_R}{f} \quad (12)$$

The phase velocity and wavelength are determined for each frequency component of the Rayleigh wave. The results are plotted in the form of a f-k spectrum (frequency vs. wavenumber) and a so called dispersion curve (velocity vs. frequency or velocity vs. wavelength). For each spacing an individual dispersion curve is generated. Subsequently, a single composite dispersion curve from all individual curves is created. Figure 11 and 12 show example plots of dispersion curves. In both figures the top plot displays the individual composite dispersion curves for different spacings each in a different colors and the bottom figure illustrates the single averaged dispersion curve in blue circles.

(a)

(b)

Figure 11: Example dispersion curve with phase velocity [m/s] vs. wavelength [m]
a) composite dispersion curves and b) composite and global dispersion curves

(a)

(b)

Figure 12: Example dispersion curve with phase velocity [m/s] vs. frequency [Hz]
a) composite dispersion curves and b) composite and global dispersion curves

To obtain the shear wave velocity from the phase velocity of the Rayleigh waves a Poisson's ratio has to be assumed. The Poisson's ratio is defined as the "ratio of transverse strain to the axial strain in an elastic material subjected to a uniaxial stress" (Gercek, 2007). The ratio between the velocities of longitudinal waves (or P-waves, V_P) and transversal waves (or S-waves, V_S) can be expressed in terms of the Poisson's ratio.

$$\frac{V_P^2}{V_S^2} = \frac{2(1 - \nu)}{(1 - 2\nu)} \quad (13)$$

where: ν =Poisson's ratio

Due to the dependency of the surface wave or Rayleigh wave velocity on the two body wave velocities, a ratio of surface wave to shear wave velocity can be defined as a function of the Poisson's ratio (Foti et al., 2015). The variation of the V_R/V_S ratio with the Poisson's ratio can be seen in Figure 13. For a Poisson's ratio ranging from 0 to 0.5, the ratio V_R/V_S varies from 0.862 to 0.955. A value of 0.92 was defined by Stokoe et al. (1994) as typical value for ratios between the surface waves velocity and shear wave velocity.

Figure 13: Variation of the ratios V_R/V_S with Poisson ratio (Foti et al., 2015))

2.3.3 Inversion Process

The last step for the method is the inversion process to evaluate the properties of the soil layers. A theoretical dispersion curve is assumed and iteratively adjusted with respect to the layer thickness, density, Poisson's ratio, and shear wave velocity until it fits the experimental dispersion curve.

To obtain the theoretical response of the soil due to an impact, the stiffness matrix approach by Kausel and Roësset (1981) can be applied. This approach can be adopted to homogeneous and layered systems.

In case of a homogeneous profile the subsurface can be represented by a half-space with constant stiffness properties (Stokoe et al., 1994). The stiffness matrix $[K]$ for a half-space can be obtained by the following equation:

$$[K] = 2kG \left\{ \frac{1-s^2}{2(1-rs)} \begin{bmatrix} r & 1 \\ 1 & s \end{bmatrix} + \begin{bmatrix} 0 & -1 \\ -1 & 0 \end{bmatrix} \right\} \quad (14)$$

with

$$r = \sqrt{1 - \frac{V_R^2}{V_P^2}} s = \sqrt{1 - \frac{V_R^2}{V_S^2}} \quad (15)$$

and

k = wave number

G = shear modulus

In this approach the Rayleigh wave velocity is characterized by the quantity V_R and depends only on the shear and compression wave velocity (V_S and V_P) of the half-space. With the help of the stiffness matrix it is possible to relate the horizontal and vertical displacement at the top of the half-space $\{U\}$ to the forces acting on the surface of the half-space $\{P\}$.

$$[K] \{U\} = \{P\} \quad (16)$$

In a inhomogeneous or layered subsurface the stiffness properties vary with depth and layer. Hence, the stiffness matrix relates the displacement at the bottom and top of every layer to an applied load at every interface. The stiffness matrix for only one single layer can be expressed by:

$$[K] = 2kG \begin{bmatrix} [K_{11}] & [K_{12}] \\ [K_{21}] & [K_{22}] \end{bmatrix} \quad (17)$$

Further equations for each element can be found in Stokoe et al. (1994), Kausel and Roësset (1981) and Kausel and Peek (1982). To obtain a dispersion curve out of the approach for homogeneous and layered subsoil, the determinant of the stiffness matrix has to be set to zero and the equation has to be solved for the Rayleigh wave velocity (V_R). Since the Rayleigh wave velocity is mostly dependent on the shear wave velocity, it is a suitable indicator of shear stiffness (Stokoe et al., 1994).

As soon as a matching theoretical dispersion curve to the experimental dispersion curve is obtained, a shear wave velocity profile over the depth corresponding to the assumed shear velocities can be determined. The whole process of the SASW method is illustrated in Figure 14.

Figure 14: Schematic Representation of SASW Measurement Process (Luke and Stokoe, 1998)

2.3.4 Typical Applications

The SASW system can be applied to many different purposes. The following applications are suggested by Olson Instruments Inc. (2012) and (2013) :

1. Determination of pavement system profiles including the surface layer, base and subgrade materials
2. Measuring the in-place shear wave velocity profile of soil and rock
3. Determination of abutment depths of bridges
4. Condition assessment of concrete liners in tunnels, slabs, and other structural

concrete members

2.4 Multi-Channel Analysis of Surface Waves

The Multi-Channel Analysis of Surface Waves (MASW) system was developed by Choon Byong Park at the University of Kansas in 1995 (Park, 1995) and first introduced in the journal Geophysics in 1999 (Park et al., 1999). Similar to the SASW system, it is also used as a seismic survey method to evaluate the elastic properties of the ground for geotechnical engineering purposes (Park Seismic LLC, 2016). To develop shear wave velocity profiles using the MASW system, the same steps used in the SASW approach are followed: 1) Acquisition, 2) Signal processing and 3) Inversion process.

2.4.1 Acquisition

As the name implies, the MASW system uses a series of receivers (usually twelve or more (Park, 1995)). Therefore, the whole range of investigation depth can be covered with just one set-up (Figure 15). The receivers are spaced evenly along an array and the spacings do not have to be changed during testing. Park(1995) defined the following recommendations for the geophone spacings during a MASW test.

The spacing between the geophones (dx) should be smaller than the one tenth of the maximum investigation depth (Z_{max}):

$$dx \leq 0.1Z_{max} \quad (18)$$

Moreover, the first geophone closest to the impact source should have a spacing greater than one half of the maximum desired wavelength (λ_{max}) from the source:

$$dx_1 \geq 0.5\lambda_{max} \quad (19)$$

The total array length of geophones (X) is defined as the distance between the first and the last geophone. The following condition should be considered:

$$X \geq Z_{max} \quad (20)$$

The last recommendation by Park (1995) covers the total required number of seismograph channels (N) to investigate the whole depth of interest in just one survey. If the condition can not be met, more surveys are necessary. In this case the total length of the geophone array should be greater than the depth of investigation.

$$N \geq \frac{X}{dx} \quad (21)$$

Different types of sources can be used to generate the surface waves. The most common ones are impulsive seismic sources like sledge hammers (Figure 15) or vibratory sources like Mini-Vibroseis. In a MASW test the spacing between the receivers is fixed and does not have to be changed during the test. Compared to the SASW system, where only two receivers are used and the spacings have to be changed multiple times, the MASW is a more time efficient method.

Figure 15: Schematic illustration of MASW set-up (Park Seismic LLC, 2016)

2.4.2 Signal Processing

Signal processing in MASW testing consists of the same steps used in the SASW testing. However, the approach to calculate the dispersion curves differs. Here, the transformation theory proposed by Park et al. (1998) is applied.

Similar to SASW testing, the measured data are recorded as time domain signals $u(x, t)$. To transform them into frequency domain signals $U(x, w)$ the Fourier transformation is used:

$$U(x, w) = \int u(x, t) \exp^{iwt} dt \quad (22)$$

Furthermore, the frequency domain signal can be expressed in terms of the phase and amplitude spectrum, $P(x, w)$ and $A(x, w)$. Since the phase spectrum contains information about the attenuation and divergence of the surface wave it can be expressed with regard to $\phi = w/c_w$ where w is the frequency in radians and c_w is the phase velocity. Therefore, the frequency domain signal can be defined as follows:

$$U(x, w) = \exp^{i\Phi x} A(x, w) \quad (23)$$

In the next step the following integration transformation is applied to $U(x, w)$ to obtain $V(w, \phi)$. It can be interpreted as the summation over offset of wavefields of a frequency after applying an offset-dependent phase shift (Park et al., 1998).

$$V(w, \phi) = \int \exp^{-i(\Phi - \phi)} [A(x, w)/|A(x, w)|] dx \quad (24)$$

Dispersion curves can be obtained by transforming $V(w, \phi)$ into $I(w, c_w)$. Figure 16 shows an experimental dispersion curve, generated by Park et al. (1998) with the transformation theory.

Figure 16: Image of a dispersion curve obtained by Park et al. (1998)

2.4.3 Inversion Analysis

An inversion analysis is used to generate a shear velocity profile from the dispersion curve. For this, a theoretical dispersion curve is established. Similar to the SASW method, the shear wave velocity can be obtained from the best fit between the experimental and theoretical dispersion curves. An example shear velocity plot computed by Park (2006) with the software SurfSeis is displayed in Figure 17.

Figure 17: Shear wave velocity plot from MASW test (Park, 2006)

2.4.4 Typical Applications

MASW testing is a seismic survey method, used for geotechnical engineering site classifications. It can also be applied for the following procedures:

1. Void mapping
2. Identification of bedrock
3. Identification of abandoned mine locations
4. Identification of bedrock fracture zones

2.5 Summary

Compared to each other, the three seismic methods, seismic refraction, SASW and MASW, have specific advantages and disadvantages.

Advantages of seismic refraction are its cheap, easy and fast data acquisition and processing. Using the travel time to of the propagating waves in the soil, it is possible to pick up distinct transitions between soil layers with different densities.

Disadvantages are for example the large source-receiver distance that is required and that only the first arrival of the wave is considered.

The main advantage of the SASW system is its simplicity in the data acquisition. The high amount of uncertainty and low accuracy in the data processing and analysis is a big disadvantage of the system.

For the data acquisition with the MASW system only one array of geophone is needed, which makes the collection of data very time sufficient. However, multiple receivers have to be used, which makes the MASW system more expensive. The main advantage of the MASW system is the long experience with multi-channel data processing from oil explorations (Park, 1995). More accurate and faster results can be obtained. Additionally, it is possible to visualize the results in 2-D shear wave velocity maps.

CHAPTER 3

Investigation of Shallow Rock using Seismic Methods

In this chapter, the suitability of seismic refraction, SASW, and MASW for identifying shallow rock is investigated from a review of the literature. Table 1 shows typical values of shear and compression wave velocities, bulk density, and Poisson's ratio for a range of materials relevant to this study (Ryden, 2004).

Table 1: Typical values of shear wave velocity and other properties for a variety geotechnical materials relevant to this study(Ryden, 2004)

Material	V_s (m/s)	V_p (m/s)	Density (kg/m³)	Poisson's ratio
Concrete	1300-2800	2000-4600	2400	0.20
Asphalt*	600-2500	1100-4500	2300-2400	0.20-0.40
Pavement base	250-500	350-800	2100-2300	0.10-0.30
Clay, silt	40-300	100-600	1400-2000	0.40-0.50
Clay,silt (saturated)	40-250	1450	1400-2000	0.45-0.50
Sand	100-500	150-1000	1600-2000	0.15-0.35
Sand (saturated)	80-450	1450	2000-2300	0.45-0.50
Till	300-750	600-1500	1800-2300	0.20-0.40
Till (saturated)	250-700	1400-2000	2100-2400	0.45-0.50
Granite, Gneiss	1700-3500	3500-7000	2200-2600	0.20

*Viscoelastic material

3.1 Identification of Shallow Rock

3.1.1 Seismic refraction

Seismic refraction has been used in several studies for the identification of bedrock, (Hart, 2011) and (Rao et al., 2004).

The aim of the research conducted by Hart (2011) was to investigate the depth of bedrock in Calumet County, Wisconsin using seismic refraction. In previous geotechnical investigations using augers and Geoprobos, a layer of very hard till within the first three feet was misinterpreted as bedrock. Due to the presence of gravel and cobbles in the till layer, hand augers and Geoprobos could not be used

for the determination of bedrock depth.

Seismic refraction tests were performed at ten locations with arrays of 48 geophones with a 10-foot (3 m) spacing. Figure 18 shows an example of the data collected at one array with the source located 240 feet along the array. The vertical motions detected by each geophone can be seen as wiggles in the plot. Additionally, the first arrival times of each layer are marked in red, orange, green and blue lines. An analysis of the data using Seisimagertm by Geomix led to plots as demonstrated in Figure 19. To check the accuracy of the seismic refraction results, backhoe pits were dug at two locations along the array. The differences in the depth of bedrock between the pit and the seismic refraction were only about two feet which supported seismic refraction as a suitable method for the identification of bedrock.

Figure 18: Example of seismic data for Line 2, shot at 240 ft. The first arrival refraction data for the four lithologies are shown by the purple, orange and green, and blue lines (Hart, 2011)

Figure 19: Interpretation of the results of seismic refraction tests at line 13 showing the interpreted stratigraphy (Hart, 2011)

However, Hart (2011) additionally tried to map shallow rock layers with less success. Using the same set-up described above, the method overestimated the depth of bedrock. It is likely that a decrease in geophone spacing would have improved the resolution at shallower depths.

3.1.2 SASW

The SASW system is suitable for soil characterization, but has not been used often for the identification of shallow rock or bedrock. Goh et al. (1994) performed a study relating Rock Quality Designation (RQD) with shear wave velocity which included twenty SASW tests on granitic rock and four SASW tests on a cut hill slope of metasedimentary rock to characterize the rock mass at these sites. In the first step Rayleigh waves were generated, measured and processed. WinSASW 3.1.3. was used for the inversion process and to interpret shear wave velocity profiles (Figure 20). Goh et al. (1994) assumed shear wave velocities of 366 - 610 m/s for highly weathered rock and 610 - 2743 m/s for slightly/moderately weathered rock. In the second step the Rock Quality Designation (RQD) value was correlated with the shear wave velocity of intact specimens in the lab and field

measurements by::

$$RQD(\%) = 100^{1-\delta} \quad (25)$$

where

$$\delta = \left[\frac{(V_{s\mu} - V_{s\beta})^2}{(V_{s\mu} + V_{s\beta})^2} \right]^2 \quad (26)$$

with:

$V_{s\mu}$ = shear wave velocity of intact rock from ultrasonic test (3365 m/s)

$V_{s\beta}$ = shear wave velocity from SASW test

Figure 20: Interpreted shear wave velocity profile at a site in Bukit Tampoi, Dengkil, Selangor (Goh et al., 1994)

3.1.3 MASW

Studies by Miller et al. (1999) and Frei (2012) used the MASW system to identify the depth of bedrock and rock instability zones. In contrast to the SASW method it is possible to generate a laterally continuous 2-D cross-section of the shear wave velocity field.

The objectives of Miller's study was to obtain an accurate mapping of bedrock and the identification of potential fracture zones within bedrock at a site in Olathe, Kansas by using the MASW method. Discontinuities in the bedrock are an important factor in hydrologic characterization of fluid flow. Miller et al. (1999) used two sets of parallel intersecting profile lines in close proximity to existing borings (Figure A.1, Appendix A). The borings were drilled to verify the results and the actual depth of bedrock. 4.5 Hz Geospace GS11D geophones were deployed along the line with spacings of 2 ft. The source was a 12 lb hammer striking a 1 ft² plate. The spacing between the closest receiver and the source was 8 ft and the spacing distance from the source to the farther receiver was 100 ft. Investigation depths of 3 ft to 50 ft could be examined with this set-up.

Figure 21 shows the shear wave velocity profile of line 2. The surface of bedrock ranged between 10 and 15 ft with shear wave velocities of about 1200 ft/s (365 m/s) . At station 2040 the lowest point of the bedrock surface was located. A extreme drop of the shear wave velocity was detected at station 2050 in 20 to 30 ft depth which was interpreted as a paleochannel infilled with weathered bedrock material or a fracture zone by Miller et al. (1999). At the same station an additional drop of shear wave velocity within the first 5 ft was measured. It can be justified with a sewer line buried at this location.

Figure 21: The MASW derived 2-D S-wave velocity field at the Olathe, Kansas site (Miller et al., 1999)

Summarizing, the results from the surface wave data showed a high accuracy with only less than 1 ft of difference to the results determined through drilling. Additionally, the MASW method showed an insensitivity to cultural obstacles and noises created by traffic, electrical and mechanical noise from industrial facilities or drilling.

The focus of the study conducted by Frei (2012) was the detection of rock and soil instability zones using hybrid seismic surveying and the MASW approach. The test site was located in a village in Switzerland, that experienced a sinkhole collapse in a residents' yard. In both test methods the same data acquisition scheme, consisting of a long array of receivers, was used. To cover the area of the sinkhole and be able to detect more instability zones, tests were conducted along three lines (Figure A.1, Appendix A). The length of each line differed and was adjusted depending on the area and depth of interest. The receiver station spacing also depended on the depth of interest and it was recommended not to exceed 1/50 to 1/30. Using these two rules, the number of receivers was determined. In addition to the seismic transects, two wells were drilled to verify the test results.

Their locations can be seen in Figure A.1 (Appendix A).

Figure 22 shows the results of the MASW test at line 1 which intersected the location of the sinkhole. The shear wave velocity of rock was defined as 500 m/s or higher by Frei (2012). The surface of bedrock is marked in red at a depth of around 5 to 12 m below the surface. Layers of lithological mixture of top soil and glacial deposits were overlying the bedrock. At the position of the sinkhole, no bedrock or bedrock with only poor quality was detected. This results were confirmed by a well drilled at the location of the sinkhole and the results of the hybrid seismic surveying.

Figure 22: The MASW derived S-wave velocity field of the line 1 to a depth of approx 12 m (Frei, 2012)

Even though the results showed similarities to the results of the hybrid seis-

mic surveying and the well data, it was recommended by Frei (2012) to calibrate and verify the results of the MASW testing with additional information or different test results. Due to the sensitivity of MASW data to structural variations in the subsurface, considerable experience is required for successful MASW data processing. Hence, use of a combination of two testing methods which can use the same acquired data, like the hybrid seismic surveying and the MASW method, was recommended.

CHAPTER 4

SASW System

In order to perform the tests for this study a commercial SASW system was provided by Professor James Kaklamanos of Merrimack College. The SASW system was developed by Olson Instruments and consists of several basic components. The most important components are the following: 1) NDE-360 platform (spectrum analyzer), 2) a pair of 2 Hz geophones, 3) a pair of 4.5 Hz geophones and 4) an impact source (e.g. sledge hammer). All four components can be seen in Figure 23 and Figure 24. According to Olson Instruments, two different SASW configurations are available with their equipment: SASW-G and SASW-S. SASW-G is the geotechnical system and used to assess material properties of soil and rock. SASW-S is mostly applied to investigate material stiffness and conditions of structures and layer thickness. However, at the time of this study Olson Instruments had not developed the SASW-G system, so only the SASW-S system was used.

(a) 4.5 Hz geophone

(b) 2 Hz geophone

Figure 23: a) 2 Hz geophone and b) 4.5 Hz geophone

(a) NDE platform

(b) Sledge hammer

Figure 24: a) NDE platform and b) small (1 kg) and big (4.5 kg) sledge hammer

Additionally, two software packages are included in the system for post data analysis: WinTFS and WinSASW.

4.1 NDE-360 platform

The NDE-360 platform is used for data acquisition, initial analysis and display of the data in both the time and frequency domains. To perform a SASW test, the platform has to be properly configured. Although only the SASW-S configuration can be used (Figure 25), it can be adjusted so that geotechnical tests can be performed. In the following sections the different settings and parameters will be explained in detail.

Figure 25: Starting screen of NDE platform

The first parameter which can be changed in the SASW system is the gain (Figure 25). The initial gain of the system is set to 1 and can be increased to 10, 100 and 1000, to amplify the signal from the geophone. The values of the gain represent the factor, the original signal gets multiplied with. Typical gains for a SASW-G test on soil are 10-1000 (Olson Instruments Inc, 2012). Depending on the soil type and spacing, the gain has to be modified during the test. Previous studies by Groenewold (2015) suggest a gain of 100 for smaller spacings and 1000 for bigger spacings.

Figure 26 displays the parameter setup screen of the SASW system. Each parameter can be adjusted by touching the corresponding button on the screen. The correct date and time of testing can be changed with the first button on the left. The second button on the left represents the Time/Point or Sampling rate. It describes in which time intervals data is acquired. For example, a sampling rate of $20\mu s$ means that every 20 microseconds data is measured. For SASW-G testing, Olson Instruments Inc. (2012) suggests sampling rates of $100\mu s$ for spacings up to

12 m and sampling rates of 200 or 500 μs for spacings greater than 12 m. Tests conducted by Groenewold (2015) on the other hand, found out that a sampling rate of 500 μs is more reasonable for all spacings. The points per record value represents the number of sampling points for each waveform. Values between 128 and 2048 can be chosen for the SASW testing. A higher number means that more data will be acquired. With the first button on the right it is possible to change the number of records. It is the total number of records measured and averaged for each spacing. The other parameters (spacing, pre-trigger and trigger level) are not relevant for the following tests or the function is not available and do not have to be changed. Table 2 summarizes the relevant parameters and the values used in this study.

Table 2: List of relevant parameters, their possible settings and values used in this study

Parameter	Settings	Values for this study
Gain	10 to 1000	100 and 1000
Time/Point or Sampling Rate	20 μs to 500 μs	500 μs
Points per Record	128 to 2048	2048
Number of Records	1 to 7	4

The channel setup at the bottom of the screen (26) shows how many geophones are connected and which channels will be used. The geophone closer the impact source should be connected to the Trigger Channel (TRIG), which is normally set to channel 1.

Figure 26: Parameter setup on NDE platform

As soon as all parameters are adjusted, the SASW test can be conducted. Once the test surface gets impacted by the source, data will be collected and visualized on the NDE platform screen. Figure 27 shows the screen of the NDE platform during the testing process. The top plot displays the signal in the time domain (amplitude vs. time) measured at the first geophone. The bottom plot shows the phase difference between the two geophones. Each measurement has to be accepted or rejected based on the following two aspects. First, the plot of the phase difference should show a clean saw tooth pattern, as it can be seen in Figure 27. Second, the scale factor of the geophones should lie within a reasonable range. It represents how strong the recorded signal is. A high scale of around 90 % implies that the signal is very strong and may be clipped. If a low scale of around 35% or less was measured, the signal might not have a good quality and background noise was recorded. A scale of around 75% is preferred to get records with clear saw tooth pattern. To achieve the desired scale the strength of the impact or the gain can be adjusted. If both aspects are satisfied, the data should be accepted.

Figure 27: Test screen of the NDE platform

The procedure will be repeated multiple times for each spacing. The number of repetitions is defined by the number of records in the parameter setup. For each spacings an averaged signal is created based on the number of records measured. Therefore, it is desirable to produce similar looking records. The coherence function of the averaged record will be shown after all records of one spacing are accepted. A good correlation with a value close to unity is desired and means that the recorded signal is not affected by background noise (Foti et al., 2015). Figure 28 illustrates a measured signal with good coherence.

Figure 28: NDE platform with a coherence plot in the top figure and an averaged phase angel vs. frequency plot in the bottom figure

4.2 Signal Processing

The recorded signals are processed using two different programs: WinTFS and WinSASW, both provided by Olson Instruments Inc. In the first program, WinTFS, the collected data gets windowed (time domain filtering) and reviewed. Records that show poor quality can be rejected. It is also possible but not mandatory to create dispersion curves for each spacing of the geophones. Afterwards, the reviewed records are imported into the second program, WinSASW. Here, a experimental dispersion curve from the recorded data is generated and a shear velocity profile of the site is estimated by an inversion analysis.

4.2.1 WinTFS

In WinTFS, all collected records from each spacing can be reviewed. Similar to the visual inspection on the NDE platform during the SASW testing, each

record can be accepted or rejected. For each spacing a final averaged record will be produced in the end. Additionally, it is possible to window the data. In this process the parts of the record which are not related to the surface wave and do not carry relevant information should be eliminated. Olson Instruments Inc. (2012) recommends no or exponential windowing for SASW-G testing. The exponential windowing should be used with decay factor of 200 or 500. The decay factor is the exponent of an exponential function and represents how fast the signal decreases and gets cut off. Hence, a higher value leads to a faster decrease and cutoff than a lower value.

The aim of the review is to generate an average record out of similar separate records with a good coherence. Figure 29 shows the window of WinTFS with an exemplary output. The top two plots display one measured signal from each geophone in the time domain. The middle graph shows the coherence function of the averaged signal from all accepted measured signals and the bottom graph illustrated the phase difference. Records which lead to a bad coherence should be rejected and not included in the averaged signal. WinTFS is also able to generate dispersion curves from the averaged signals. However, the averaged signal will be imported into WinSASW for the process.

Figure 29: Example of the interface of WinTFS showing the velocity-time histories of the two geophones (top), the coherence of the recorded geophone signals (middle) and the phase angle as a function of frequency (bottom).

4.2.2 WinSASW

The process to generate a shear wave velocity profile in WinSASW is divided into three main steps. First, irrelevant or scattered data is removed through a process called masking. Second, an combined experimental dispersion curve from the dispersion curves of each spacings is compiled. Third, a theoretical soil profile is assumed and by using an inversion analysis, it gets iteratively adjusted until it's theoretical dispersion curve matches the experimental dispersion curve. With the theoretical soil profile and dispersion curve, a shear wave velocity profile can be determined.

The averaged signals from WinTFS can be uploaded in form of transfer functions into WinSASW. For each spacing a .hyx file has to be uploaded. To be able to distinguish between the different files later on, the files should be named with

respect to the particular spacing. In addition, the spacing between the source and the first geophone as well as between the two geophones has to be specified before the file is loaded into the software. After all files are uploaded, masking of the data can begin.

Masking

The first task to generate a dispersion curve from the imported records is to choose which portions of the phase spectrum (phase angle vs. frequency) will be used for further analysis. This procedure is called masking (i.e. eliminating). Areas within the record get selected or eliminated based on the following criteria:

1. Masking out areas outside the frequency range of interest (based on the geophone spacing, Table 3)

Table 3: Suggested frequency span for different spacings

Spacing [m]	maximum Frequency [Hz]
0.5	1000
1	500
2	250
4	125
6	93.75
8	62.5

2. Masking out areas where the phase angle - frequency plot does not show a clean saw tooth pattern (messy) created by random noise or are significantly undulating (Joh, 1996).
3. Masking out areas with backwards sawtooth pattern (Joh, 1996)
4. Areas with low Gabor spectrum frequency have to be eliminated (Foti et al., 2015) (Figure 30)

Figure 30: Example of data visualized in WinSASW, showing phase angle as a function of frequency (top) and Gabr Spectrum as a function of frequency (bottom). The shaded data in the upper figure is being masked (removed) for further analysis.

5. Masking out the near field, defined by $\lambda \leq 4 * R_d$ where R_d is the radius of the geophone ($R_d = 3.5cm$ for 4.5 Hz geophone and $R_d = 5cm$ for 2 Hz geophones) (Joh, 1996)
6. Masking out area defined by $\lambda \geq 2 * d$ where λ is the wavelength and d is the spacing (Joh, 1996)

Additionally to the mentioned masking criteria, the phase spectrum has to be unwrapped. The plot of the frequency response in WinSASW in Figure 30 shows as a wrapped phase with the phase ranging from $\pm 180^\circ$. However, to calculate the time delay in Equation 10 the so-called unwrapped phase difference is needed. The phase can be unwrapped by adding the correct number of 360° cycles (number of jumps) as shown in Figure 31 (Al-Hunaidi, 1992). The masking

process is conducted separately for each receiver spacing.

Figure 31: Wrapped and unwrapped phase spectra (Al-Hunaidi, 1992)

The process of masking and the identification of jumps is the part of the post-processing which has the highest impact on the evaluation of a dispersion curve and therefore on the calculation of the shear wave velocity profile. A high amount of personal judgment and experience is necessary for the masking process which can lead to a source of mistakes and uncertainties.

Dispersion Curve

The second step of the post-processing is the calculation of a composite experimental dispersion curve for each geophone spacing. The following equation using the unwrapped phase spectrum and the receiver spacing is applied:

$$V_R = f * \lambda = f * \frac{d}{\phi/360^\circ} \quad (27)$$

WinSASW plots the composite dispersion curve for each receiver spacing in a different colors or pattern. A dispersion curve which does not match into the course of the other curves can be removed. Figure 32 shows an example of composite experimental dispersion curves.

Figure 32: Example of composite dispersion curves (phase velocity [m/s] vs. wavelength [m])

The last step before the inversion analysis is the computation of a representative dispersion curve. An averaging algorithm from Joh (1996) is used by WinSASW to generate the representative dispersion curve from the composite experimental dispersion curves. The averaging algorithm is based on a moving average (Press et al., 1996) and a polynomial best-fit analysis. The trend of the composite dispersion curve is characterized by averaged points. The number of points and the distribution along the curve is based on areas along the composite dispersion curves which show a high density of points and a good match between the different curves. Within WinSASW, there are two different methods for developing the averaged experimental dispersion curve: a global average dispersion curve and an array average dispersion curve. Both can be used for the inversion analysis. While the global average dispersion curve consists of only one average dispersion curve, the array average dispersion curve uses the individual dispersion curves computed for each receiver spacing. Example plots of each representative dispersion curve are illustrated in Figures 33 and 34. The global average dispersion curve in Figure 33 is plotted in blue and shows the general trend of the composite dispersion curves plotted in gray. In Figure 34 the array average dispersion curves

are illustrated in different colors depending on the receiver spacing. The composite dispersion curves are also plotted in gray.

Figure 33: Example of composite dispersion curves (grey) and the global dispersion curve (blue). Phase velocity has units of m/s and wavelength has units of m.

Figure 34: Example of composite dispersion curves (grey) and the array dispersion curves (multiple colors and patterns). Phase velocity has units of m/s and wavelength has units of m.

Inversion Analysis

The inversion analysis in WinSASW to evaluate the shear wave velocity profile from the phase velocity dispersion curve determined in SASW testing is based on the maximum likelihood method (Joh, 1996). In the procedure the best match between the experimental and a theoretical dispersion curve is to be determined. The theoretical dispersion curve is created by the response of a

soil model to an impact established with dynamic stiffness matrix method. A description of the method can be found in Chapter 2.3.3.

The first step of the inversion procedure is to define a soil profile which characterizes the investigated depth of the test site. Two different approaches were considered for the layering of the starting profile in this study. In the first one, at least 10 layers with increasing layer thickness from the top to the bottom were considered. The layer thickness ranged from 0.1 to 0.8 m depending on the investigation depth. Figure 35 shows an example of this layering approach. The second approach used in this study considers an initial layer thickness of 0.5 m. Depending on the depth of interest, the number of layers can be smaller than 10. The last layer represents the half space of the dynamic stiffness and has to be set to a value thicker than the maximum wavelength for both approaches. Once the initial layer thickness is set, values for the P-Wave Velocity, S-Wave Velocity, density, Poisson's Ratio and a damping factor have to be defined for each layer. In both approaches constant values for every parameter and over the whole depth of interest are specified. A P-Wave Velocity of 0, S-Wave Velocity of 150 m/s, density of 1900 kg/m³, Poisson's Ratio of 0.3333 and damping factor of 0.02 is recommended by Olson Instruments Inc (2013). An example of the WinSASW window showing the input table for the initial parameters is displayed in Figure 35.

Parameters for Forward Modeling Analysis

Analysis Type: ☒ 2D ☒ 3D ☐ Enhanced 3D

F. of Displ. Bulb: 1.00
SubLayers: 16

Soil Profile No. of Layers: 15

	Thickness	P-Wave Vel.	S-Wave Vel.	Density	Poisson's R.	Damping F.
L6	0.2000	0.0000	150.0000	1900.0000	0.3333	0.0200
L7	0.3000	0.0000	150.0000	1900.0000	0.3333	0.0200
L8	0.3000	0.0000	150.0000	1900.0000	0.3333	0.0200
L9	0.3000	0.0000	150.0000	1900.0000	0.3333	0.0200
L10	0.4000	0.0000	150.0000	1900.0000	0.3333	0.0200
L11	0.4000	0.0000	150.0000	1900.0000	0.3333	0.0200
L12	0.4000	0.0000	150.0000	1900.0000	0.3333	0.0200
L13	0.5000	0.0000	150.0000	1900.0000	0.3333	0.0200
L14	0.5000	0.0000	150.0000	1900.0000	0.3333	0.0200
L15	400.0000	0.0000	150.0000	1900.0000	0.3333	0.0200
Dfit	0.0000	0.0000	150.0000	1900.0000	0.3333	0.0200

Apply All Apply All Apply All Apply All Apply All Apply All

Insert a Layer before Layer 10
Delete Layer.....

Copy from INV 0 Clear All

Figure 35: First parameter setup for inversion analysis

In the same window, the option to choose an analysis type is given. Two types are available, 2-dimensional and 3-dimensional. In the 2D analysis the wave front is assumed to be planar. For the 3D analysis a cylindrical wavefront of surface waves and a hemispherical wavefront of body waves is assumed. The 3D assumption considers the modes of all stress waves and therefore is recommended by Joh (1996) for the analysis of SASW tests. In this study the 3D analysis type was considered.

In the second step the experimental dispersion curve has to be selected. It can be chosen between a inversion analysis based on a global averaged dispersion curve or an array averaged dispersion curve. As mentioned before, the global averaged dispersion curve consists only of one averaged best-fit curve, while the array averaged dispersion curve is a collection of the best-fit curves for each receiver spacing. Each dispersion curve is suitable under different conditions. In case of scattered data in the dispersion curve resulting from significant lateral variability

at the test site, a global inversion analysis should be selected. The wide-banded data gets averaged in one best-fit global dispersion curve. If the layers of the soil profile show significant changes in the stiffness, a averaged inversion analysis leads to more accurate shear wave velocity profiles (Joh, 1996). In addition, the needed computational power and time to run the inversion has to be considered. Since the average dispersion curve is a composition of multiple global dispersion curves the needed computational time is significantly longer.

The third step of the inversion analysis is to determine starting model parameters with the best match to the experimental dispersion curve. The starting model parameters are an initial guess that should be a reasonable estimate of the expected material properties at the site. The scheme to develop the starting model parameters in form of a preliminary shear wave velocity profile, is divided up into two phases (Joh, 1996). In the first phase, a soil profile is assumed with a number of layers based on the number of data points of the experimental dispersion curve. For this step a global averaged dispersion curve is employed as the experimental dispersion curve, even if an array inversion will be performed. The layer thicknesses of the phase 1 profile are calculated with the corresponding wavelength of the dispersion data point and a depth-to-wavelength ratio. For each defined layer a shear wave velocity is determined using the dynamic stiffness matrix. In the second phase, another soil profile with a reduced amount of layers is assumed. The thickness of the layers increase with depth, identical to the first approach of the soil profile in the first step of the inversion analysis. The shear wave velocities for the layers of the phase 2 profile are the weighted averaged velocities of the phase 1 profile. The resulting preliminary shear wave velocity is subsequently used to perform the forward modeling analysis and determine the phase velocities for the theoretical dispersion curve corresponding to the frequencies of the experi-

mental dispersion curve data. To evaluate the goodness of the theoretical curve a root-mean-square (RMS) error is calculated. The whole process is repeated with changing depth-to-wavelength ratios. The number of repetitions and the range of the ratio can be defined in WinSASW. In this study ten repetitions and a range of 0.34 to 0.65 for the depth-to-wavelength ratio were specified for every starting model parameter calculation. The soil profile resulting from the theoretical dispersion curve which shows the best alignment with the experimental dispersion curve and has the lowest RMS error is set as the new starting profile for the inversion analysis.

The actual inversion analysis is performed in the fourth step. Within the inversion analysis multiple parameters and factors can be adjusted. The two most important model parameters which can be varied to reduce the RMS error are the shear wave velocity and the layer thickness. In each inversion one of the two parameters or both at the same time can be varied. To achieve good results only one model parameter should be modified in the beginning until a reasonable profile is generated. The amount of iterations in one inversion run can be defined in WinSASW. At the end of each run the soil profile of the iteration with the least RMS error and best match between the two dispersion curves is set as the new starting model profile. As soon as the error is not decreasing anymore and no better match of the two dispersion curves can be determined, the second model parameter can be adjusted and then both parameters can be adjusted together. To improve the results, four other parameters can be adjusted manually: density, Poisson's ratio, investigation depth and uncertainty factor.

The value of the density can be changed in correlation to the shear wave velocity. Stokoe et al. (2005) recommends a unit weight of 18.9 kN/m^3 (1927 kg/m^3) for shear wave velocities smaller than 610 m/s which accords to the value

set as a starting model parameter. For shear velocities between 610 and 914 m/s, Stokoe et al. (2005) suggests unit weights of about 19.7 kN/m^3 (2000 kg/m^3) and 20.4 kN/m^3 (2100 kg/m^3) for shear wave velocities greater than 914 m/s.

The Poisson's ratio should be changed if the soil layer is below the water table. However, Stokoe et al. (2005) points out that the change does only have a minor effect on the shear wave velocity. The investigation depth can be adjusted based on the depth resolution analysis. The depth resolution analysis provides a sense of how well the model is resolved and determines the maximum possible investigation depth which can be examined for given experimental data (Joh, 1996). In WinSASW for each inversion a profile of the model parameter resolution in each layer is plotted. Figure 36 shows an example of the resolution over the depth for one inversion. In this example, the last two layers have a low resolution and can be eliminated for the model profile. The inversion will then be performed with a decreased amount of layers.

Figure 36: Example of a resolution plot over depth

The last parameter which can be changed is the uncertainty factor of the shear wave velocity or thickness. Joh (1996) defines the uncertainty factor as the ratio of the standard deviation of the model parameter to the standard deviation of the experimental data. The default value in the software is 0.2 for the uncertainty factor of the shear wave velocity and 0.05 for the thickness.

The inversion analysis is repeated until the lowest value of RMS error is reached, preferably under 10, and the experimental and theoretical dispersion match.

CHAPTER 5

Site Descriptions and Test set up

This chapter presents a description of the sites tested for this study. Figure 37 shows a map of Rhode Island with the locations of all the test sites.

Figure 37: Map of Rhode Island with the locations of all test site of this study (Google Maps)

The test sites for the identification of shallow rock were chosen based on the knowledge of existing rock within the first few meters. The first site, Middleton Building at the Bay Campus of the University of Rhode Island, has been used for previous SASW tests (Groenewold, 2015). It was included in this study to gain experience with the equipment and to compare to previous results. The two sites at the Baker Pines Road Bridge and the Weaver Hill Road Bridge were chosen in consultation with the Rhode Island Department of Transportation. Three additional sites on the URI main campus were added as test sites to this study. Since the shear wave velocities for rock differ in the literature, a test at a known rock

outcrop was performed in Jamestown, Rhode Island. The sites are described in more detail below.

5.1 Beavertail State Park, Jamestown, RI

A SASW test was performed on the rocky cliffs at Beavertail State Park, located at the southern tip of Jamestown. Figure 38 shows the position of the midpoint with the coordinates about 41.27042° North and -71.2352° West and Figure 44 shows the test being performed. The cliffs consist of pure and mostly weathered rock.

Figure 38: Test location at Beavertail State Park in Jamestown, RI

Due to small amount of flat and intact areas of rock, the SASW test only was conducted with the 4.5 Hz geophones with receiver spacings of 0.5, 1, 1.5, 2, 2.5,

3 and 3.5 m. The 1.5 kg hammer was used as the source.

Figure 39: Test set up at Beavertail State Park in Jamestown, RI

5.2 Middleton Building, URI

The Middleton building is located at the Narragansett Bay Campus of the University of Rhode Island. The tests were conducted on a grassy area next to the Middleton building (Figure 40). The location was chosen due to its easy access and the possibility to compare the results to previous SASW test results conducted at URI (Groenewold, 2015).

Figure 40: Test site adjacent to the Middleton Building (Google Maps)

Two tests were performed at this test site. The first test was conducted on March 31st, 2016 and the second on April 19th, 2016. In both cases the 4.5 Hz geophones were used to test smaller spacings up to 6 m. The 2 Hz geophones were only used during the second test for spacings up to 8 m. The 4.5 Hz geophones were provided with a spike (Figure 23 a)) which made it easier to fix them to the ground. At the bottom of the 2 Hz geophones a flat surface was mounted (Figure 23 b)).

A 4 kg sledge hammer (Figure 24) was used as an impact source. The previous

study by Groenewold (2015) recommended the use of a steel plate covered with a rubber pad as a striker plate on the ground (Figure 41).

Figure 41: Steel plate and rubber mat used for the testing

In the first test on the 31st of March, the 4.5 Hz and 2 Hz geophones were used. Tests with the 4.5 Hz geophones were conducted for spacings of 0.5m, 1m, 1.5m, 2m, 3m, 4m, 5m and 6m, while the 2 Hz geophones were installed for spacings of 5m, 6m, 7m and 8m. In addition, the location of the source was reversed in the second part of the test, so that the test could be performed in forward and reverse direction (Figure 42). Since the volume of soil tested does not change between the two parts of the test, the reversed testing simply helps to evaluate the accuracy of the results.

Figure 42: Scheme of the forward and reverse testing with the CRMP geometry (Aouad et al., 1993)

The second test at the Middleton Building was also performed with 4.5 Hz and 2 Hz geophones, considering the same spacings as in the second test (0.5 m - 8 m). Tests in the forward and reversed directions were performed. The tests in the forward directions were repeated five times for each spacings with the purpose to evaluate experimental error between repeated tests.

The testing parameters of both tests are summarized in the Tables 5 to 8 in Chapter 6.

5.3 Baker Pines Road Bridge

The Baker Pines Road Bridge can be found along the highway I-95 north of Richmond, Rhode Island. For the construction of the bridge, eight borings were conducted in 1965 (Figure 43). The boring logs are provided in Appendix B. All logs show bedrock or granite layers within the first 4 to 10 feet (1.2 - 3 m). A preliminary site investigation showed that the locations marked red in Figure 44 were suitable as test sites. Both sites are located within the area of boring 596-5 and close to I-95.

Figure 43: Location of Borings at Baker Pines Road Bridge (DOT)

Figure 44: Locations of sites tested at Baker Pines Road Bridge (Google Maps)

Detailed pictures of the two selected test sites are displayed in Figure 45 and Figure 46. The SASW test at the first site was performed on May 31st, 2016 and on May 19th, 2016 on the second site. At the first test site, the ground surface consisted of sand and some gravel. Within the first centimeters of soil a dense layer was found which made it impossible to use the spike for the 4.5 Hz geophones. Instead flat plates which are included in the SASW package were mounted to the bottom of the 4.5 Hz geophones. To secure a good connection between the geophones and the ground, white plastic bags filled with sand, were put on top of the geophones (Figure 45 b)). The 4.5 Hz geophones were arranged with spacings of 0.5m, 1m, 1.5m, 2m, 3m, 4m, 5m and 6m and the 2 Hz geophones with spacings of 5m, 6m, 7m and 8m. Both the small and the large sledge hammer were used. The choice of the source depended on multiple factors, e.g. the spacing and the gain. Table 9 describes details of each spacing.

The second site is a grassy area (Figure 46). Here, the spikes could be used again for the 4.5 Hz geophone. Similar to the first test at the Baker Pines Road Bridge, the 2 Hz geophones were covered with sand bags. A summary of the parameters are in Table 10.

Figure 45: Test site 1 a) looking north and b) looking south (Google Maps)

Figure 46: Test site 2 a) looking east and b) looking south (Google Maps)

5.4 Weaver Hill Road Bridge

The Weaver Hill Road Bridge is also located along highway I-95, north of West Greenwich, Rhode Island. Similar to the Baker Pines Road Bridge, eight borings were conducted before the construction of the bridge in 1965. The locations of the

borings are illustrated in Figure 47. The boring logs provided by the Department of Transportation can be found in Appendix B. In all cases granite layers were found within the first 10 feet (3 m).

Figure 47: Location of Borings at the Weaver Hill Road Bridge (DOT)

Figure 48: Locations of sites tested at the Weaver Hill Road Bridge (Google Maps)

Two locations close to the bridge were suitable for SASW testing and are marked in red in Figure 48. Both sites had an easy accessibility and an even surface. The first location was west of the bridge and is a small unpaved path parallel to I-95. The second location was a grassy area east of the bridge and lies in between I-95 and RI-3. Detailed pictures of both sites are shown in Figure 49.

Figure 49: a) Test site 1 west of the bridge and b) Test site 2 east of the bridge

The SASW tests were conducted on May 19th, 2016 at the first location and on May 25th, 2016 at the second location. The set-up at both locations was very similar to the set-up of the test at the Baker Pines Road bridge. The 4.5 Hz, with spikes, and the 2 Hz geophones (with sand bags to ensure good contact between the ground and the surface) were used in both tests. The 4.5 Hz geophones were placed at spacings of 0.5m, 1m, 1.5m, 2m, 3m, 4m, 5m and 6m and the 2 Hz geophones were placed at spacings of 5m, 6m, 7m and 8m. The values of the applied gain and the choice of the source, small or big sledge hammer, are summarized in Table 11 and 12 for each spacing.

5.5 URI main campus

The main campus of the University of Rhode Island is located in South Kingston, Rhode Island. The consulting company GZA conducted twelve borings in the engineering quad of the URI main campus in May 2016 for the design of a new engineering building. Three of the borings, GZ-4, GZ-7 and GZ-8, showed shallow rock and were additionally suitable and accessible for SASW testing. The

location of the three borings can be seen in Figure 50. The boring logs for the three sites were provided by GZA and can be found in the Figures B.6 to B.8 (Appendix B). To ensure that a unbiased test was performed, no information regarding the depth of bedrock or the type and thickness of the layers were provided during data collection or processing. The tests were conducted on June 17th, 2016 at GZ-4, June 27th, 2016 at GZ-8 and June 31st, 2016 at GZ-7. After the shear wave velocity profiles were finalized, they were compared to the boring logs. The three selected test sites are going to be described in more detail in this section.

Figure 50: Boring locations of GZA on the URI main campus (GZA)

5.5.1 GZ-4

The test site GZ-4 is a grassy area next to Crawford Hall and the Powerhouse Road on the URI main campus. The boring of GZA was conducted in one of the outside corners of the lawn area and is visible in Figure 51 a) as a light spot. Multiple trees, one hydrant and three covers of a water main can be found on the lawn area. The location of the array of geophones, illustrated in red in Figure 51 a), was chosen in a way to avoid any interference with these objects. Figure 51 b) shows the position of the 4.5 Hz geophones during a test.

Figure 51: a) Test site GZ-4 on the URI main campus and b) Test site GZ-4 with the set-up of the 4.5 Hz geophones

The major parts of the testing area consisted of a sandy and grassy surface. Close to the Crawford Hall a path made out of sand and gravel ran parallel to the building. Along this path only the source but no geophone was positioned. Hence, 4.5 Hz geophones with spikes could be used for the entire test. Identical to the previous tests, sand bags were used to apply weight on the 2 Hz geophones to ensure a good geophone-ground contact. The test set-up was based on the CRMP geometry with the midpoint located in the middle of the grassy area. The source

and the first geophone moved closer to the Crawford Hall with increasing receiver spacing, while the second geophone moved in the opposite direction, closer to the parking lot. Receiver spacings of 0.5, 1, 1.5, 2, 3, 4, 5 and 6 m were applied for the 4.5 Hz geophones and spacings of 5, 6, 7 and 8 m for the 2 Hz geophones. The applied test parameter such as the gain are summarized in Table 13 for each spacing.

5.5.2 GZ-7

The test site GZ-7 lied on a grassy area in the middle of the Engineering Squad. The boring by GZA was conducted in the top left corner of the lawn area and can be seen in Figure 52 a) as a lighter patch on the grass. The array of geophones for this study ran beside the boring location and is marked in red in Figure 52 a). Figure 52 b) shows the position of the 4.5 Hz geophones during a test with a spacing of 0.5 m. Within the testing area no trees or other objects are located. The only interference which could have an effect on the test results is a gully which can be found to the left of the end of the approximate array of geophones in Figure 52 a).

Figure 52: a) Test site GZ-7 on the URI main campus and b) Test site GZ-7 with the set-up of the 4.5 Hz geophones

The set-up is also based on the CRMP geometry with the midpoint right next to the boring location of GZA (Figure 102 b)). 4.5 Hz geophones with a spike were used and set up in spacings of 0.5, 1, 1.5, 2, 3, 4, 5 and 6 m. The 2 Hz geophones were weighted with sand bags in positioned in spacings of 5, 6, 7 and 8 m. Table 14 summarizes the applied test parameter for each spacing.

5.5.3 GZ-8

The last test site on the URI main campus, GZ-8, was also located in the middle of the engineering quad and close to Wales Hall (Figure 106). The location of the boring conducted by GZA was situated in between two trees without enough space for a SASW test in between. Hence, the position of the array of geophones was moved closer to the sidewalk (Figure 106 b)).

Figure 53: a) Test site GZ-8 on the URI main campus and b) Test site GZ-8 with the set-up of the 2 Hz geophones

As in the other two test on the URI main campus, the CRMP geometry with the array of geophones running parallel to Wales Hall and a midpoint adjacent to the boring location has been used. The test was performed with spacings of 0.5, 1, 1.5, 2, 3, 4, 5 and 6 m for the 4.5 Hz geophones and 5, 6, 7 and 8 m for the 2 Hz geophones. Due to a grassy and sandy surface, 4.5 Hz geophones with attached spikes could be used. The applied sandbags on the 2 Hz geophones can be seen in Figure 106 b). All information and parameters used for this test are summarized in Table 15.

CHAPTER 6

Test results

In this chapter the results from the SASW tests conducted at the four different locations are presented. For some test sites boring logs or results from previous studies were provided and are compared to the results of this study. In addition, at two test sites the effect of testing in the forward and reverse directions and the repeatability of the results are investigated.

SASW tests for the identification of rock were conducted at four different sites: Middleton Building on the URI Bay Campus, Baker Pines Road Bridge, Weaver Hill Road Bridge and on the engineering quad on URI Main Campus. A rock outcrop was also tested to obtain an unambiguous measure of the shear wave velocity of intact rock.

6.1 Beavertail State Park, Jamestown, RI

The aim of the SASW test on the rocky cliffs in Jamestown was to identify typical shear wave velocities for rock. The SASW tests were performed only with a rubber mat placed directly on the rock. The input parameters used for the test are summarized in Table 4.

Table 4: Summary of testing parameters for the test at Beavertail State Park, Jamestown RI

Test #	Spacing [m]	Source	Geophone [Hz]	NDE file	Gain	Scale	Disp.- Curve
1	0.5	SS	4.5	24	1000	69%	No
2	1	SS	4.5	25	1000	76%	Yes
3	1.5	SS	4.5	26	1000	78%	Yes
4	1.5	SS	4.5	27	1000	72%	Yes
5	2	SS	4.5	28	1000	74%	Yes

6	2.5	SS	4.5	29	1000	76%	Yes
7	3	SS	4.5	30	1000	74%	Yes
8	3.5	SS	4.5	31	1000	74%	Yes

with: SS = small sledgehammer (1 kg) and BS = big sledgehammer (4.5 kg)

In the first step of the data processing, the signals were reviewed and windowed in the program WinTFS, using an exponential cut filter with a decay of 500. The file NDE 24 with a spacing of 0.5 m did not show a good averaged signal or coherence and was eliminated from further analysis. The other seven files were imported into the program WinSASW. After the masking process, composite dispersion curves for every spacings were determined. They can be seen in Figure 54 (phase velocity vs. wavelength) and Figure 55 (phase velocity vs. frequency). Based on the composite dispersion curve a averaged global dispersion curve was calculated (Figure 56, blue solid circles).

Figure 54: Composite dispersion curve (phase velocity [m/s] vs. wavelength [m]) for the test performed at Beavertail State Park, Jamestown RI

Figure 55: Composite dispersion curve (phase velocity [m/s] vs. frequency [Hz]) for the test performed at Beavertail State Park, Jamestown RI

In the next step the soil profile for the inversion analysis was defined. The depth of investigation depends on the wavelengths of the global dispersion curve and can be assumed to be half of the maximum wavelength. This approach is based on the fact that most of the particle motion occurs at depths less than one-half of the wavelength (Stokoe et al., 2005). In this case the maximum wavelength was 7 m which leads to an investigation depth of 3.5 m.

The layering of the starting soil profile was defined in two different ways (Chapter 4.2.2). For both approaches, a global inversion was performed. However, in this section only the results of the first approach, increasing layer thickness with depth is described.

In the global inversion process, a theoretical dispersion curve with the lowest RMS error and best match to the experimental dispersion curve was calculated. In Figure 56 the theoretical dispersion curve is illustrated in red empty circles and the experimental dispersion curve in solid blue circles.

Figure 56: Global representative dispersion curve (solid blue circles) and theoretical dispersion curve (empty red circles) (phase velocity [m/s] vs. wavelength [m]) for the test performed at Beavertail State Park, Jamestown RI

The computed shear wave velocity profile is shown in Figure 57. The shear wave velocities range from 360 m/s in the first layer to 2200 m/s at a depth of about 2 m. A clear difference in the course of the three profiles for sand, gravel and rock is visible. Based on the results, shear wave velocities greater than 500 m/s are assumed to be rock in the following tests.

Figure 57: Shear wave velocity profile for the test performed at Beavertail State Park, Jamestown RI

6.2 Middleton Building, URI Bay Campus, Narragansett

A SASW test was performed at the Middleton Building in a previous study by Groenewold (2015). Figure 58 shows the shear wave velocity profile from that study. The shear wave velocity profile indicates dense soil layers in the first 4 m with velocities between 200 and 600 m/s, overlying soil with velocities up to 800 m/s. Due to the shear wave velocity values obtained from the SASW test at Jamestown and from the literature review (Table 1), rock layers can be assumed starting at a depth of about 4 m. The results of the tests conducted in this study

on March 31 and April 13, 2016 will be compared to the results of Groenewold (2015).

Figure 58: Test result of Groenewold (2015) at the Middleton Building, URI Bay Campus

6.2.1 March 31, 2016

The SASW test on March 31st, 2016 was performed in the forward and reverse direction. The results of both tests will be discussed separately and then compared.

The testing parameters that were used in the forward test are summarized in Table 5. In the first step of the post-processing, the data was reviewed in the program WinTFS. An exponential cut filter with a decay of 500 was used in the windowing process. All 13 recordings showed signals with good quality, hence all

were used for further analysis.

Table 5: Summary of testing parameters for the forward test on March 31st, 2016 at the Middleton Building, URI Bay Campus

Test #	Spacing [m]	Source	Geophone [Hz]	NDE file	Gain	Scale	Disp.- Curve
1	0.5	SS	4.5	188	100	80%	Yes
2	1	SS	4.5	189	100	85%	No
3	1.5	SS	4.5	190	100	85%	Yes
4	2	SS	4.5	191	100	85%	No
5	2	SS	4.5	192	1000	80%	Yes
6	3	SS	4.5	193	1000	75%	Yes
7	4	SS	4.5	194	1000	83%	Yes
8	5	SS	4.5	195	1000	81%	Yes
9	6	SS	4.5	196	1000	75%	Yes
10	5	SS	2	197	1000	78%	Yes
11	6	SS	2	198	1000	79%	Yes
12	7	SS	2	199	1000	71%	Yes
13	8	SS	2	200	1000	72%	Yes

with: SS = small sledgehammer (1 kg) and BS = big sledgehammer (4.5 kg)

In the second step the software WinSASW was used to mask the signals and to calculate a composite dispersion curve for each spacing. Signals, that did not conform to the masking criteria or resolved in a dispersion curve which did not match with the other dispersion curves, were excluded for the calculation of the representative averaged dispersion curve. For this test only the NDE files 189 and 191 were excluded (Table 5). The calculated composite dispersion curves for all the other files are shown in Figure 59 (phase velocity vs. wavelength) and Figure 60 (phase velocity vs. frequency). Each spacing is illustrated in a different color.

Figure 59: Composite dispersion curve (phase velocity [m/s] vs. wavelength [m]) for the forward test on March 31, 2016 at the Middleton Building, URI Bay Campus

Figure 60: Composite dispersion curve (phase velocity [m/s] vs. frequency [Hz]) for the forward test on March 31, 2016 at the Middleton Building, URI Bay Campus

A global dispersion curve was determined from the composite dispersion curves and is illustrated in Figure 62 in solid blue circles. The maximum wavelength of the global dispersion curve is 14 m which leads to an approximate investigation depth of around 7 m (half of the maximum wavelength).

Inversions with two different starting layer thicknesses, which were determined with the two proposed approaches from Chapter 4.2.2, were performed. In the following only the results of the approach with increasing layer thickness over depth in the starting soil profile will be discussed. The results of the other approach can be seen in Figure 116 (a) in Chapter 7. The global inversion analysis was conducted

until the best match between the theoretical and experimental dispersion curve was found. An array inversion analysis was not considered due to a high spreading between the composite dispersion curves at a wavelength of about 2 to 3 m (Figure 61). The result of the best match is displayed in Figure 62 with the theoretical dispersion curve in empty red circles and the experimental dispersion curve in solid blue circles.

Figure 61: Array representative dispersion curve (phase velocity [m/s] vs. wavelength [m]) for the forward test on March 31, 2016 at the Middleton Building, URI Bay Campus

Figure 62: Global representative dispersion curve (solid blue circles) and theoretical dispersion curve (empty red circles) (phase velocity [m/s] vs. wavelength [m]) for the forward test on March 31, 2016 at the Middleton Building, URI Bay Campus

The shear wave velocity profile corresponding to the determined theoretical dispersion curve is shown in Figure 63. Additionally, typical shear velocity profiles for soft sand, silt and clay as well as for dense gravel can be seen as references (Lin et al., 2014).

In the first 3 m the shear wave velocity ranged from 100 to 500 m/s, which are typical values for dense gravel or till. In the deeper layers the velocity increases to a value up to 950 m/s.

Figure 63: Shear wave velocity profile for the forward test on March 31, 2016 at the Middleton Building, URI Bay Campus

The SASW test on March 31st, 2016 at the Middleton Building was repeated in the reverse direction. The midpoint of the CRMP geometry stayed the same, but the source and the first geophone moved in the opposite direction. The testing parameter of the reverse test are summarized in Table 6.

Table 6: Summary of testing parameters for the reverse test on March 31st, 2016 at the Middleton Building, URI Bay Campus

Test #	Spacing [m]	Source	Geophone [Hz]	NDE file	Gain	Scale	Disp.-Curve
1	0.5	SS	4.5	201	100	82%	Yes
2	1	SS	4.5	202	100	80%	No
3	1.5	SS	4.5	203	100	80%	Yes
4	2	SS	4.5	204	100	70%	Yes
5	3	SS	4.5	205	100	30%	Yes
6	3	SS	4.5	206	1000	72%	Yes
7	4	SS	4.5	207	1000	82%	Yes
8	5	SS	4.5	208	1000	72%	Yes
9	6	SS	4.5	209	1000	79%	Yes
10	5	SS	2	210	1000	71%	Yes
11	6	SS	2	211	1000	80%	Yes
12	7	SS	2	212	1000	84%	No
13	8	SS	2	213	1000	82%	Yes

with: SS = small sledgehammer (1 kg) and BS = big sledgehammer (4.5 kg)

The software WinSASW was used to mask the data and calculate a composite dispersion curve for each spacing. In the Figures 64 (phase velocity vs. wavelength) and 65 (phase velocity vs. frequency) the composite dispersion curves are presented in different colors. As seen in Table 6, only the data from the file NDE 202 was not used for that calculation and therefore is not shown in Figures 64 and 65.

Figure 64: Composite dispersion curve (phase velocity [m/s] vs. wavelength [m]) for the reverse test on March 31, 2016 at the Middleton Building, URI Bay Campus

Figure 65: Composite dispersion curve (phase velocity [m/s] vs. frequency [Hz]) for the reverse test on March 31, 2016 at the Middleton Building, URI Bay Campus

The composite dispersion curves were used to determine a global dispersion curve (Figure 66 in solid blue circles). The investigation depth of interest was set to 7.5 m, based on the maximum wavelength of 15 m from the global dispersion curve. The two approaches from Chapter 4.2.2 were used to estimate the starting soil profile. The results of the first approach are going to be discussed in this section. Using the experimental dispersion curve and the starting soil profile a global inversion analysis was conducted. Figure 66 shows the best match of the experimental dispersion curve (solid blue circles) and the computed theoretical dispersion curve (empty red circles).

Figure 66: Global representative dispersion curve (solid blue circles) and theoretical dispersion curve (empty red circles) (phase velocity [m/s] vs. wavelength [m]) for the reverse test on March 31, 2016 at the Middleton Building, URI Bay Campus

In Figure 67 the corresponding shear wave velocity profile is shown. At a depth of 0.5 m a spike in the shear wave velocity was found with a value of 500 m/s.

Figure 67: Shear wave velocity profile for the reverse test on March 31, 2016 at the Middleton Building, URI Bay Campus

6.2.2 April 13, 2016

The SASW test on April 13th was performed in the forward and reverse direction. Additionally, in the forward test five data sets were recorded for each spacing to demonstrate the experimental error.

The results of the forward test will be discussed first. Table 7 shows the testing parameters for all five records of each spacing. All files showed a good signal and coherence during the review in WinTFS and were used for the further analysis in WinSASW.

Table 7: Summary of testing parameters for the forward test on April 13, 2016 at the Middleton Building, URI Bay Campus

Test #	Spacing [m]	Source	Geophone [Hz]	NDE file	Gain	Scale	Disp.-Curve
1	0.5	BS	4.5	214	100	86%	Yes
2	0.5	BS	4.5	215	100	80%	No
3	0.5	BS	4.5	216	100	81%	No
4	0.5	BS	4.5	217	100	82%	No
5	0.5	BS	4.5	218	100	83%	No
6	1	BS	4.5	219	100	70%	Yes
7	1	BS	4.5	220	100	68%	No
8	1	BS	4.5	221	100	70%	No
9	1	BS	4.5	222	100	67%	No
10	1	BS	4.5	223	100	70%	No
11	1.5	BS	4.5	224	100	72%	Yes
12	1.5	BS	4.5	225	100	73%	No
13	1.5	BS	4.5	226	100	75%	No
14	1.5	BS	4.5	227	100	75%	No
15	1.5	BS	4.5	228	100	75%	No
16	2	BS	4.5	229	100	77%	Yes
17	2	BS	4.5	230	100	72%	No
18	2	BS	4.5	231	100	80%	No
19	2	BS	4.5	232	100	80%	No
20	2	BS	4.5	233	100	78%	No
21	3	BS	4.5	234	1000	72%	Yes
22	3	BS	4.5	235	1000	79%	No
23	3	BS	4.5	236	1000	75%	No
24	3	BS	4.5	237	1000	75%	No
25	3	BS	4.5	238	1000	75%	No
26	4	BS	4.5	239	1000	75%	No
27	4	BS	4.5	240	1000	78%	No
28	4	BS	4.5	241	1000	76%	No
29	4	BS	4.5	242	1000	72%	No
30	4	BS	4.5	243	1000	77%	Yes
31	4	BS	4.5	244	1000	70%	No
32	5	BS	4.5	245	1000	80%	Yes
33	5	BS	4.5	246	1000	77%	No
34	5	BS	4.5	247	1000	74%	No
35	5	BS	4.5	248	1000	75%	No
36	5	BS	4.5	249	1000	74%	No

37	6	BS	4.5	250	1000	78%	Yes
38	6	BS	4.5	251	1000	75%	No
39	6	BS	4.5	252	1000	78%	No
40	6	BS	4.5	253	1000	77%	No
41	6	BS	4.5	254	1000	70%	No
42	5	BS	2	255	1000	75%	No
43	5	BS	2	256	1000	76%	No
44	5	BS	2	257	1000	77%	No
45	5	BS	2	258	1000	79%	No
46	5	BS	2	259	1000	77%	Yes
47	6	BS	2	260	1000	77%	No
48	6	BS	2	261	1000	75%	No
49	6	BS	2	262	1000	77%	Yes
50	6	BS	2	263	1000	80%	No
51	6	BS	2	264	1000	77%	No
52	7	BS	2	265	1000	76%	Yes
53	7	BS	2	266	1000	74%	No
54	7	BS	2	267	1000	77%	No
55	7	BS	2	268	1000	76%	No
56	7	BS	2	269	1000	76%	No
57	8	BS	2	270	1000	70%	Yes
58	8	BS	2	271	1000	71%	No
59	8	BS	2	272	1000	75%	No
60	8	BS	2	273	1000	75%	No
61	8	BS	2	274	1000	74%	No

with: SS = small sledgehammer (1 kg) and BS = big sledgehammer (4.5 kg)

In the program WinSASW, the signals were masked and composite dispersion curves were calculated. The five dispersion curves for each spacing showed in most cases good alignments and similar trends. Figure 68 illustrates the dispersion curves for the spacings 0.5 (solid circles), 3 (crosses), 5 (solid squares) and 8 m (empty squares). At spacings of 0.5 m and 8 m the dispersion curves follow exactly the same line. For the other two spacings, small deviations between the curves were determined. However, the general trend is still comparable.

Figure 68: Comparison of 5 composite dispersion curves (phase velocity [m/s] vs. wavelength [m]) for the spacings 0.5, 3, 5 and 8 m, April 13, 2016 at the Middleton Building, URI Bay Campus

Due to the high amount of data files only the record from each spacing which showed the best result after the masking process and satisfied the most masking requirements was used to determine the composite dispersion curves. The following files were chosen: NDE 214, 219, 224, 229, 234, 243, 245, 250, 259, 262, 265 and 270. Figures 69 and 70 show the resulting composite dispersion curves.

Figure 69: Composite dispersion curve (phase velocity [m/s] vs. wavelength [m]) for the forward test on April 13, 2016 at the Middleton Building, URI Bay Campus

Figure 70: Composite dispersion curve (phase velocity [m/s] vs. frequency [Hz]) for the forward test on April 13, 2016 at the Middleton Building, URI Bay Campus

A global averaged dispersion curve from the composite dispersion curves was determined in the next step. Figure 71 shows the global dispersion curve in solid blue circle. A maximum wavelength of the dispersion curve of about 14 m was captured which resolves in a approximate investigation depth of 7 m. The two different processes to determine the starting soil profile have been described in Chapter 4.2.2. Here, the results of the second method will be discussed in detail. The estimated match of the theoretical (empty red circles) and experimental (solid blue circles) dispersion curves with the global inversion analysis is shown in Figure 71.

Figure 71: Global representative dispersion curve (solid blue circles) and theoretical dispersion curve (empty red circles) (phase velocity [m/s] vs. wavelength [m]) for the forward test at April 13th, 2016 at the Middleton Building, URI Bay Campus

The corresponding shear wave velocity profile is presented in Figure 72. In the first 1.5 m the shear wave velocities range from 100 to 450 m/s. A significant decrease of the shear wave velocities can be seen in deeper layers. At a depth of 1.5 to 7 m shear wave velocities of only 25 to 125 m/s were found. These velocities are typical for very soft material like clay (Table 1).

Figure 72: Shear wave velocity profile for the forward test on April 13, 2016 at the Middleton Building, URI Bay Campus

The SASW test in the reverse direction was conducted with only one repetition for each spacing. Table 8 shows the parameters for this test.

Table 8: Summary of testing parameters for the reverse test on April 13, 2016 at the Middleton Building, URI Bay Campus

Test #	Spacing [m]	Source	Geophone [Hz]	NDE file	Gain	Scale	Disp.- Curve
1	0.5	BS	4.5	275	100	80%	Yes
2	1	BS	4.5	276	100	75%	Yes

3	1.5	BS	4.5	277	100	74%	Yes
4	2	BS	4.5	278	100	72%	No
5	3	BS	4.5	279	1000	74%	Yes
6	4	BS	4.5	280	1000	77%	No
7	5	BS	4.5	281	1000	76%	Yes
8	6	BS	4.5	282	1000	75%	No
9	5	BS	2	283	1000	73%	No
10	6	BS	2	284	1000	68%	No
11	7	BS	2	285	1000	76%	No
12	8	BS	2	286	1000	44%	No

with: SS = small sledgehammer (1 kg) and BS = big sledgehammer (4.5 kg)

The review of the data in WinTFS showed that the records of the second geophone of the files NDE 282 to 286 were very weak (Figure 73 (right)). These five data files were excluded from further analysis and not imported in WinSASW.

Figure 73: Recorded signal at the first and second geophone of the file NDE 285 at the Middleton Building, URI Bay campus

The remaining files were imported and masked in WinSASW. Additional files were eliminated from the further processing based on masking criteria described in Chapter 4.2.2. Table 8 shows which files were used for the determination of composite dispersion curves. The composite dispersion curves are shown in Figures 74 and 75.

Figure 74: Composite dispersion curve (phase velocity [m/s] vs. wavelength [m]) for the reverse test on April 13, 2016 at the Middleton Building, URI Bay Campus

Figure 75: Composite dispersion curve (phase velocity [m/s] vs. frequency [Hz]) for the reverse test on April 13, 2016 at the Middleton Building, URI Bay Campus

The global dispersion curve was calculated from the composite dispersion curves and is illustrated in Figure 76. Since only dispersion curves from smaller spacings were considered the maximum wavelength of the averaged dispersion curve was approximately 9 m. The approximate investigation depth is therefore limited to 4.5 m.

Two starting model profiles were created, based on the approaches described in Chapter 4.2.2. In this section the results of the first approach will be evaluated.

A global inversion analysis was performed until the best match between the theoretical and experimental dispersion curve was determined. Figure 76 shows the

two dispersion curves with the best alignment.

Figure 76: Global representative dispersion curve (solid blue circles) and theoretical dispersion curve (empty red circles) (phase velocity [m/s] vs. wavelength [m]) for the reverse test on April 13, 2016 at the Middleton Building, URI Bay Campus

The shear wave velocity profile for the test in the reverse direction can be found in Figure 77. The shear wave velocities range from 100 m/s in the first layer to 350 m/s at a depth of 2.5 m. At a depth of about 2.5 m the shear wave velocity decreases significantly. Generally, the found velocities are typical for very dense soils.

Figure 77: Shear wave velocity profile for the reverse test at April 13th, 2016 at the Middleton Building, URI Bay Campus

6.3 Baker Pines Road Bridge

Two locations were tested at the Baker Pines Road Bridge. The sites are located in the area of the north abutment of the bridge and close to the boring location 596-5 conducted by the Department of Transportation of Rhode Island in 1965. The corresponding boring log which was provided by the Department of Transportation of Rhode Island can be seen in the Figure B.2 (Appendix B). It shows a 2 ft (0.8 m) thick layer of top soil, overlying a 7 ft thick layer of medium

to fine sand and then granite. The granite is marked as decomposed, very coarse and weathered.

6.3.1 Location 1

The testing parameters for the test are summarized in Table 9.

Table 9: Summary of testing parameters for the test at location 1 at the Baker Pines Road Bridge

Test #	Spacing [m]	Source	Geophone [Hz]	NDE file	Gain	Scale	Disp.- Curve
1	0.5	SS	4.5	375	100	78%	Yes
2	1	SS	4.5	376	100	73%	Yes
3	1.5	SS	4.5	377	100	75%	Yes
4	1.5	BS	4.5	378	100	73%	Yes
5	2	BS	4.5	379	100	77%	Yes
6	2	BS	4.5	380	100	77%	Yes
7	3	BS	4.5	381	1000	77%	Yes
8	3	SS	4.5	382	1000	74%	Yes
9	4	SS	4.5	383	1000	75%	Yes
10	5	SS	4.5	384	1000	73%	Yes
11	6	SS	4.5	385	1000	78%	Yes
12	5	SS	2	386	1000	74%	Yes
13	6	SS	2	387	1000	71%	Yes
14	7	BS	2	388	1000	74%	Yes
15	8	BS	2	389	1000	73%	Yes

with: SS = small sledgehammer (1 kg) and BS = big sledgehammer (4.5 kg)

The composite dispersion curves produced after the masking process in Win-SASW are displayed in the Figures 78 (phase velocity vs. wavelength) and 79 (phase velocity vs. frequency). As it can be seen in Table 9, a composite dispersion curve for every file was calculated. The composite dispersion curve of each spacing was used for the determination of the representative averaged dispersion curve. A global dispersion curve was calculated and can be seen in Figure 80 in blue solid circles.

Figure 78: Composite dispersion curve (phase velocity [m/s] vs. wavelength [m]) from location 1 at the Baker Pines Road bridge

Figure 79: Composite dispersion curve (phase velocity [m/s] vs. frequency [Hz]) from location 1 at the Baker Pines Road bridge

The maximum wavelength of the global dispersion curve was around 12 m and therefore, a investigation depth of 6 m was assumed. The starting soil profile for this investigation depth was set up in two different ways as described in Chapter 4.2.2. The global inversion analysis was conducted for both approaches. Figure 81 shows the matches between the theoretical (empty red circles) and the experimental (solid blue circles) for a) the first approach and b) the second approach. In addition to the good agreement, a low RMS error was computed for both theoretical dispersion curves.

(a)

(b)

Figure 80: Global representative dispersion curve (solid blue circles) and theoretical dispersion curve (empty red circles) (phase velocity [m/s] vs. wavelength [m]) for the location location 1 at the Baker Pines Road bridge for a) increasing thickness for the starting soil profile and b) uniform initial thickness for the starting soil profile

The shear wave velocity profiles for both cases can be seen in Figure 81. The boring log from the DOT specified the first 0.8 m as topsoil. The two profiles show a range of shear wave velocities from 100 to 400 m/s for this layer, which can be characterized as topsoil. The following layer was identified as sand by

the DOT which would assume shear wave velocities of about 100 to 400 m/s. Compared to Lin et al (2014), the second approach (Figure 81 b)) shows typical shear wave velocities for medium soft sands in this layer, while the shear wave velocities determined with first approach (Figure 81 a)) show typical values for gravel. The last layer of weathered granite was identified in both profiles. The shear wave velocities increase to values of 700 to 900 m/s which indicate rock.

Figure 81: Shear wave velocity profile of location 1 at the Baker Pines Road bridge for a) first approach for the starting soil profile and b) second approach for the starting soil profile

6.3.2 Location 2

The SASW test at the second location next to the Baker Pines Road Bridge was conducted with the equipment and parameters shown in Table 10. The data

processing in WinTFS showed, that the coherence and quality of all data files is sufficient enough to import them into WinSASW for further analysis.

Table 10: Summary of testing parameters for the test location 2 at the Baker Pines Road Bridge

Test #	Spacing [m]	Source	Geophone [Hz]	NDE file	Gain	Scale	Disp.- Curve
1	0.5	SS	4.5	318	100	72%	Yes
2	1	SS	4.5	319	100	77%	Yes
3	1	SS	4.5	320	100	75%	Yes
4	1.5	SS	4.5	321	100	74%	Yes
5	1.5	SS	4.5	322	100	74%	Yes
6	2	BS	4.5	323	100	74%	No
7	2	BS	4.5	324	100	73%	No
8	3	SS	4.5	325	1000	72%	Yes
9	3	SS	4.5	326	1000	71%	Yes
10	4	SS	4.5	327	1000	74%	Yes
11	5	SS	4.5	328	1000	71%	Yes
12	6	SS	4.5	329	1000	70%	No
13	5	SS	2	330	1000	78%	No
14	5	SS	2	331	1000	73%	No
15	5	SS	2	332	1000	76%	Yes
16	6	SS	2	333	1000	72%	Yes
17	7	SS	2	334	1000	74%	Yes
18	7	SS	2	335	1000	74%	Yes
19	8	SS	2	336	1000	65%	Yes
20	8	SS	2	337	1000	75%	No

with: SS = small sledgehammer (1 kg) and BS = big sledgehammer (4.5 kg)

The masking process in WinSASW led to the execution of the following files, NDE 323, 324, 329, 330, 331 and 337, because they did not fulfill the requirements defined in Chapter 4.2.2. On the base of the remaining data files, composite dispersion curves were determined. In Figures 82 and 83 the composite dispersion curves for the different spacings can be seen.

Figure 82: Composite dispersion curve (phase velocity [m/s] vs. wavelength [m]) for location 2 at the Baker Pines Road bridge

Figure 83: Composite dispersion curve (phase velocity [m/s] vs. frequency [Hz]) for location 2 at the Baker Pines Road bridge

In the next step, the global dispersion curve was estimated for the inversion analysis. This can be seen in Figure 84 in solid blue circles. The maximum wavelength of the dispersion curve of 13 m led to the assumption of a approximate investigation depth of 6.5 m. The set up of the starting soil profile followed the same scheme as location 1 and is divided up into two approaches (Chapter 4.2.2). In the section only the results of the first approach will be considered. The theoretical dispersion curve of the inversion with the smallest RMS error is displayed in Figure 84.

Figure 84: Global representative dispersion curve (solid blue circles) and theoretical dispersion curve (empty red circles) (phase velocity [m/s] vs. wavelength [m]) for location 2 at the Baker Pines Road bridge

Location 2 at Baker Pines Road Bridge also lied within the area of the boring location 596-5 of the DOT. Hence, the computed shear wave velocity profile of Figure 85 will be compared to the same soil profile as location 1. The first two layers of topsoil and sand, up to a depth of 3 m, can be identified in the shear wave velocity profile with velocities of 100 to 300 m/s. In the following layer the shear wave velocity increased, which agrees with the weathered granite defined in the boring log. However, at a depth of 5 m the shear wave velocity profiles reached a velocity of 1500 m/s which is a typical value for unweathered rock. At a depth of 5.5 m the shear wave velocity decreased again to typical shear wave velocities of weathered rock (700 m/s). The jump at 5 m occurred probably due to experimental errors and uncertainties.

Figure 85: Shear wave velocity profile for location 2 at the Baker Pines Road bridge

6.4 Weaver Hill Road Bridge

RIDOT conducted several borings for the construction of the Weaver Hill Road Bridge in East Greenwich, Rhode Island. The two test sites of this study were located close to two of the eight boreholes. For the first location the boring WH-3 and for the second location the boring WH-8 were considered for comparison.

6.4.1 Location 1

Boring log WH-3 (Figure B.4, Appendix B) showed a first layer of top soil, followed by a layer of sand and fine gravel. At a depth of about 10 ft (3 m) the

top of rock, weathered and seamy granite, was determined.

The SASW test was conducted with the parameters summarized in Table 11.

Table 11: Summary of testing parameters for the test at location 1 at the Weaver Hill Road Bridge

Test #	Spacing [m]	Source	Geophone [Hz]	NDE file	Gain	Scale	Disp.- Curve
1	0.5	SS	4.5	338	100	73%	Yes
2	1	SS	4.5	339	100	77%	Yes
3	1	BS	4.5	340	100	75%	Yes
4	1.5	BS	4.5	341	100	75%	Yes
5	1.5	BS	4.5	342	100	75%	Yes
6	2	SS	4.5	343	1000	75%	No
7	3	SS	4.5	344	1000	73%	Yes
8	4	BS	4.5	345	1000	74%	Yes
9	4	BS	4.5	346	1000	72%	Yes
10	5	BS	4.5	347	1000	79%	No
11	5	BS	4.5	348	1000	76%	Yes
12	6	BS	4.5	349	1000	73%	Yes
13	6	BS	4.5	350	1000	73%	Yes
14	5	BS	2	3531	1000	72%	Yes
15	5	BS	2	352	1000	75%	Yes
16	6	BS	2	353	1000	72%	Yes
17	6	BS	2	354	1000	75%	Yes
18	7	BS	2	355	1000	75%	Yes
19	7	BS	2	356	1000	70%	Yes
20	8	BS	2	357	1000	73%	No
21	8	BS	2	358	1000	73%	No
22	8	BS	2	359	1000	74%	No

with: SS = small sledgehammer (1 kg) and BS = big sledgehammer (4.5 kg)

In the first step the data was reviewed and windowed in the program WinTFS using an exponential cut filter with a decay of 500. The computed composite dispersion curves are displayed in the Figures 86 and 87.

Figure 86: Composite dispersion curve (phase velocity [m/s] vs. wavelength [m]) at location 1 at the Weaver Hill Road Bridge

Figure 87: Composite dispersion curve (phase velocity [m/s] vs. frequency [Hz]) at location 1 at the Weaver Hill Road Bridge

The second step included the calculation of a global dispersion curve from the composite dispersion curves. Figure 88 shows the global dispersion curve with a maximum wavelength of 13 m. Due to Stokoe et al. (2005), the investigation depth can be assumed as half the maximum wavelength which is 6.5 m in this case.

The procedure to determine the starting soil profile was explained in Chapter 4.2.2. The results of the first approach will be described in this section while the results of the second approach can be found in Figure 116 (d) in Chapter 7. The global inversion analysis resulted in the theoretical dispersion curve shown in Figure 88.

Figure 88: Global representative dispersion curve (solid blue circles) and theoretical dispersion curve (empty red circles) (phase velocity [m/s] vs. wavelength [m]) at location 1 at the Weaver Hill Road Bridge

The boring log which was provided by the DOT shows weathered granite at a depth of 3 m. Typical shear wave velocities for weathered rock of 500 m/s or higher were found at a depth of 5 m in this study. The layers above were identified as dense sand to dense gravel with shear wave velocities of 150 to 450 m/s. The layering specified by the DOT characterizes the layers above the granite as layers of sand and fine gravel. Lower shear wave velocities would have been expected in the computed profiles to confirm that.

Figure 89: Shear wave velocity profile for the test at location 1 at the Weaver Hill Road Bridge

6.4.2 Location 2

Boring log WH-8 (Figure B.5, Appendix B) shows a similar soil profile to the one from WH-3. At about 10 ft (3 m) weathered and seamy granite was found with layers of top soil and sand above.

The test site was located between I-95 and RI-3, two high-traffic streets. At larger spacings the gain had to be set to a higher value to be able to record a signal. However, higher gains also caused the system to trigger from vibrations created by passing cars and trucks. Figure 90 (a) shows such a signal. Due the

high number of passing cars it was not possible to record four similar signals for one spacing with a gain set to 1000. As you can see in Table 12 the test had to be stopped at 6 m. A review of the recordings for larger spacings on the site showed that those recordings are not usable for further post-processing and analysis.

Figure 90: Example of a triggered signal caused by a passing car or truck at the location 2 at the Weaver Hill Road Bridge

Table 12: Summary of testing parameters for the test at location 2 at the Weaver Hill Road Bridge

Test #	Spacing [m]	Source	Geophone [Hz]	NDE file	Gain	Scale	Disp.- Curve
1	0.5	SS	4.5	360	100	75%	Yes
2	0.5	SS	4.5	361	100	74%	Yes
3	1	SS	4.5	362	100	73%	Yes
4	1	SS	4.5	363	100	73%	No
5	1.5	BS	4.5	364	100	74%	Yes
6	1.5	BS	4.5	365	100	74%	No
7	2	BS	4.5	366	100	76%	Yes

8	2	BS	4.5	367	100	72%	Yes
9	3	BS	4.5	368	100	71%	Yes
10	3	BS	4.5	369	100	71%	No
11	4	BS	4.5	370	100	42%	Yes
12	5	SS	4.5	371	1000	75%	Yes
13	5	SS	4.5	372	1000	73%	Yes
14	6	SS	4.5	373	1000	74%	Yes

with: SS = small sledgehammer (1 kg) and BS = big sledgehammer (4.5 kg)

The computed dispersion curves of the remaining data are demonstrated in Figures 91 and 92.

Figure 91: Composite dispersion curve (phase velocity [m/s] vs. wavelength [m]) for the test at location 2 at the Weaver Hill Road Bridge

The composite dispersion curves were used to determine a global dispersion curve as the representative averaged dispersion curve. It is displayed in Figure 93 in solid blue circles. A global inversion analysis was conducted, considering an approximate investigation depth of 4 m due to a maximum wavelength of 8 m of the experimental dispersion curve. The starting soil profile was specified in two ways, as already described in the sections before. The following results are based on the approach for the starting soil parameter with increasing layer thickness over the depth. Figure 93 shows the theoretical dispersion curve (empty red circles) conducted in the global inversion analysis. The results from the other approach

are presented in Figure 116 (e) in Chapter 7.

Figure 92: Composite dispersion curve (phase velocity [m/s] vs. frequency [Hz]) for the test at location 2 at the Weaver Hill Road Bridge

The best matching theoretical dispersion from Figure 93 led to the shear wave velocity profile in Figure 94. According to the boring log provided by the DOT, the first 3 m consists of topsoil and layers of sand and fine gravel. The shear wave velocities in that depth range between 100 and 200 m/s which can be characterized as medium dense sand and dense gravel. At a depth of about 2.5 m the shear wave velocities increase and velocities of 550 to 800 m/s were determined. This identification is in agreement with the boring log which shows weathered granite at a depth of about 3 m. Generally, the shear wave velocity profile shows the same trend as the boring log.

Figure 93: Global representative dispersion curve (solid blue circles) and theoretical dispersion curve (empty red circles) (phase velocity [m/s] vs. wavelength [m]) for the test at location 2 at the Weaver Hill Road Bridge

Figure 94: Shear wave velocity profile for the test at location 2 at the Weaver Hill Road Bridge

6.5 Blind Prediction of Depth to Rock at the URI Main Campus

SASW tests at three different locations on the Engineering quad on the URI main campus were performed. The locations were chosen to coincide with the locations of borings performed in May, 2016 by GZA Geoenvironmental, Inc. as part of the geotechnical investigation for a new engineering building. The boring logs were reviewed by Dr. Chris Baxter and the locations of borings GZA-4, GZA-7 and GZA-8 were chosen for SASW testing. However, the boring logs were not shared with the author until after the shear wave profiles were generated. As

such, these tests constitute a blind prediction of the ability of SASW techniques to identify shallow rock.

6.5.1 GZA-4

The boring log identified as GZA-4 showed bedrock at a depth of about 9.3 ft (2.8 m), overlaid by sand and fill consisting of fine to medium sand and some gravel. The top layer with a thickness of 0.5 ft (0.15 m) consisted of dense fine sand. At a depth of 4 to 6 ft (1.2 to 1.8 m) a lower Standard Penetration Test (SPT) value than in the layers above and below was measured, which indicates a less dense or loose material.

The testing parameters which were used during the test at GZA-4 are listed in Table 13.

Table 13: Summary of testing parameters for the test at the boring location GZA-4 at the URI Main Campus

Test #	Spacing [m]	Source	Geophone [Hz]	NDE file	Gain	Scale	Disp.- Curve
1	0.5	SS	4.5	452	100	77%	Yes
2	0.5	SS	4.5	453	100	71%	Yes
3	1	SS	4.5	454	100	74%	No
4	1	SS	4.5	455	100	72%	No
5	1.5	SS	4.5	456	100	73%	No
6	1.5	SS	4.5	457	100	76%	Yes
7	2	SS	4.5	458	100	76%	Yes
8	2	SS	4.5	459	100	76%	Yes
9	3	SS	4.5	460	1000	71%	Yes
10	3	SS	4.5	461	1000	75%	No
11	4	SS	4.5	462	1000	77%	Yes
12	4	SS	4.5	463	1000	75%	Yes
13	5	SS	4.5	464	1000	75%	Yes
14	5	SS	4.5	465	1000	70%	Yes
15	6	SS	4.5	466	1000	75%	Yes
16	6	SS	4.5	467	1000	74%	Yes
17	5	SS	2	468	1000	72%	Yes

18	5	SS	2	469	1000	72%	Yes
19	6	SS	2	470	1000	74%	Yes
20	6	SS	2	471	1000	76%	No
21	7	SS	2	472	1000	73%	Yes
22	7	SS	2	473	1000	77%	Yes
23	8	SS	2	474	1000	76%	No
24	8	SS	2	475	1000	73%	Yes
25	8	SS	2	476	1000	75%	No

with: SS = small sledgehammer (1 kg) and BS = big sledgehammer (4.5 kg)

For each spacing a dispersion curve was calculated. However, seven dispersion curves, NDE 454, 455, 456, 461, 471, 474 and 476 were not used to calculate the representative averaged dispersion curve because the course of the curve did not fit to the other curves or a better dispersion curve of the same spacing was used. The composite dispersion curves of the approved files are plotted in the Figures 95 (phase velocity vs. wavelength) and Figure 96 (phase velocity vs. frequency). A global averaged dispersion curve of the composite dispersion curves was calculated and can be seen in Figure 97 in solid blue circles.

Figure 95: Composite dispersion curve (phase velocity [m/s] vs. wavelength [m]) for the boring location GZA-4 on URI main campus

Figure 96: Composite dispersion curve (phase velocity [m/s] vs. frequency [Hz]) for the boring location GZA-4 on URI main campus

The global averaged dispersion curve had a maximum wavelength of 16 m which led to a investigation depth for the shear wave velocity profile of about 8 m. Two different starting soil profiles were entered in the software, based on the two approaches proposed in Chapter 4.2.2. In both cases the global inversion analysis was performed. The process was the same as described in the previous sections. The theoretical dispersion curve (empty red circles) with the lowest RMS error and the best match with the experimental dispersion curve is shown in Figure 97 for starting soil profiles with a) increasing thickness over depth and b) constant thickness over depth.

(a)

(b)

Figure 97: Global representative dispersion curve (solid blue circles) and theoretical dispersion curve (empty red circles) (phase velocity [m/s] vs. wavelength [m]) for the boring location GZA-4 on URI main campus for a) first approach for the starting soil profile and b) second approach for the starting soil profile

The Figures 98 a) and b) show the corresponding shear wave velocity profiles to the theoretical dispersion curves. In the first meter of depth, shear wave velocities of 150 to 350 m/s were calculated in both approaches. These velocities correspond to the top soil and fill detected with the boring. At a depth of 1 to 1.5 m the

shear wave velocity dropped to 100 m/s which corresponds to velocities of soft sand (green dashed line) at that depth (Lin et al., 2014). As already mentioned before, the SPT value at a depth of 1.2 to 1.8 m was also smaller than the value half a meter above and below. The drop in the SPT value corresponds to the shear wave velocities and can be explained by a layer of loose sand in that depth. At a depth of 2 m the shear wave velocity increased up to 600 and 800 m/s in both approaches. Drilling was stopped at this depth due to rock, which explains the high velocities. Further information about the soil profile below 2.8 m does not exist. The computed shear wave velocity profile however shows values up to a depth of around 8 m. In the first approach the velocity decreases again to values corresponding to gravel or till while the velocity at the second approach stays constant at a value of 600 m/s. Since the results for the deep layers can not be reviewed, it is not possible to determine which approach shows more accurate shear wave velocity values.

Figure 98: Shear wave velocity profile of location GZA-4 on URI Main Campus for a) first approach for the starting soil profile and b) second approach for the starting soil profile

6.5.2 GZA-7

At the location of the boring GZA-7, a top layer of dense sand with a thickness of 2 ft (0.6 m), followed by a 2 ft thick layer of gravel, a 5 ft (1.5 m) thick layer of medium dense sand and a 5 ft thick layer of till was determined. The boring had to be stopped at a depth of about 14.3 ft (4.4 m). The SASW test was conducted at the same location. Table 14 summarizes the parameters used for the test at GZA-7.

Table 14: Summary of testing parameters for the test at the boring location GZA-7 at the URI Main Campus

Test #	Spacing [m]	Source	Geophone [Hz]	NDE file	Gain	Scale	Disp.- Curve
1	0.5	SS	4.5	428	100	76%	Yes
2	0.5	SS	4.5	429	100	75%	Yes
3	1	SS	4.5	430	100	72%	Yes
4	1	SS	4.5	431	100	75%	No
5	1.5	BS	4.5	432	100	76%	Yes
6	1.5	BS	4.5	433	100	75%	No
7	2	BS	4.5	434	100	72%	Yes
8	2	BS	4.5	435	100	71%	Yes
9	3	SS	4.5	436	1000	72%	Yes
10	3	SS	4.5	437	1000	77%	No
11	4	SS	4.5	438	1000	76%	Yes
12	4	SS	4.5	439	1000	71%	Yes
13	4	SS	4.5	440	1000	73%	Yes
14	5	BS	4.5	441	1000	77%	Yes
15	5	BS	4.5	442	1000	73%	Yes
16	6	BS	4.5	443	1000	74%	Yes
17	6	BS	4.5	444	1000	75%	Yes
18	5	BS	2	445	1000	75%	Yes
19	6	BS	2	446	1000	72%	Yes
20	6	BS	2	447	1000	75%	No
21	7	BS	2	448	1000	75%	No
22	7	BS	2	449	1000	75%	No
23	8	BS	2	450	1000	77%	Yes
24	8	BS	2	451	1000	77%	Yes

with: SS = small sledgehammer (1 kg) and BS = big sledgehammer (4.5 kg)

The composite dispersion curves of the approved files are shown in the Figures 99 (phase velocity vs. wavelength) and Figure 100 (phase velocity vs. frequency). The corresponding global averaged dispersion curve which was determined from the composite dispersion curves is displayed in blue solid circles in Figure 101.

Figure 99: Composite dispersion curve (phase velocity [m/s] vs. wavelength [m]) for the boring location GZA-7 on URI main campus

Figure 100: Composite dispersion curve (phase velocity [m/s] vs. frequency [Hz]) for the boring location GZA-7 on URI main campus

As it can be seen in Figure 101, the maximum wavelength of the global dispersion curve is 10 m and therefore, a investigation depth of 5 m was assumed. Similar to the evaluations in the previous sections, inversions with different starting soil profiles were conducted. In the following only the results from the first approach, increasing thickness of layers with depth, will be discussed. The theoretical dispersion curve of this approach showed a better match to the experimental dispersion curve and the calculated RMS error was smaller. Figure 101 shows the best matching theoretical dispersion curve in empty red circles. The results of the

second approach can be found in Figure 116 (g) in Chapter 7.

Figure 101: Global representative dispersion curve (solid blue circles) and theoretical dispersion curve (empty red circles) (phase velocity [m/s] vs. wavelength [m]) for the boring location GZA-7 on URI main campus

The corresponding shear wave velocity profile for the displayed theoretical dispersion curve is shown in Figure 102. The boring log of GZA indicates topsoil and dense gravel within in the first 1.2 m. The shear wave velocities at the equivalent depth show higher values than typical velocities for dense gravel. At around 1 m the shear wave velocity decreased to values typical for sand. This corresponds to the layer of sand in a depth of 1.2 to 2.7 m identified by the GZA boring. According to the boring log a layer of till follows the sand until further boring was refused at a depth of 4.2 m. The till layer can be seen in the shear wave velocity profile with an increase in velocity at around 2.5 m. The velocity values indicated dense gravel and therefore could also be interpreted as till. However, an additional increase at a depth of 4.2 m which would explain the refusal of the boring was not determined.

Figure 102: Shear wave velocity profile of location GZ-7 on URI Main Campus

6.5.3 GZA-8

The boring at the location GZA-8 had to be stopped at a depth of 20.4 ft (6.2 m) due to bedrock. The layers above consist of a 6 ft (1.8 m) thick layer of topsoil, fill (loose sand and some gravel) and a 14.4 ft (4.4 m) thick layer of till .

The SASW tests were conducted with the parameters summarized in Table 15. The jump in numbering of the NDE files occurred due to a switch of the memory card. As for the other two sites on URI main campus, no data were excluded in WinTFS. Especially in the lower frequencies good signals with high coherence were

obtained.

Table 15: Summary of testing parameters for the test at the boring location GZA-8 at the URI Main Campus

Test #	Spacing [m]	Source	Geophone [Hz]	NDE file	Gain	Scale	Disp.- Curve
1	0.5	SS	4.5	502	100	74%	No
2	0.5	SS	4.5	503	100	4%	No
3	1	SS	4.5	2	100	75%	Yes
4	1	SS	4.5	3	100	72%	Yes
5	1.5	SS	4.5	4	100	73%	No
6	1.5	SS	4.5	5	100	75%	Yes
7	2	SS	4.5	6	100	76%	Yes
8	2	SS	4.5	7	100	74%	Yes
9	3	SS	4.5	8	1000	74%	Yes
10	3	SS	4.5	9	1000	72%	Yes
11	4	SS	4.5	10	1000	75%	Yes
12	4	SS	4.5	11	1000	74%	Yes
13	5	SS	4.5	12	1000	75%	Yes
14	5	SS	4.5	13	1000	72%	Yes
15	6	SS	4.5	14	1000	76%	No
16	6	SS	4.5	15	1000	77%	No
17	5	SS	2	16	1000	72%	Yes
18	5	SS	2	17	1000	72%	Yes
19	6	SS	2	18	1000	76%	Yes
20	6	SS	2	19	1000	75%	Yes
21	7	SS	2	20	1000	72%	Yes
22	7	SS	2	21	1000	74%	Yes
23	8	SS	2	22	1000	74%	No
24	8	SS	2	23	1000	73%	No

with: SS = small sledgehammer (1 kg) and BS = big sledgehammer (4.5 kg)

After masking the records in WinSASW, the following files were disregarded and not used to develop the composite dispersion curves: NDE 502, 503, 4, 14, 15, 22 and 23. The composite dispersion curves determined with the approved files are shown in the Figures 103 and 104.

Figure 103: Composite dispersion curve (phase velocity [m/s] vs. wavelength [m]) for the boring location GZA-8 on URI main campus

Figure 104: Composite dispersion curve (phase velocity [m/s] vs. frequency [Hz]) for the boring location GZA-8 on URI main campus

Based on the composite dispersion curves a global dispersion curve was calculated. Figure 105 shows the experimental global dispersion curve in solid blue circles. The maximum wavelength of the dispersion curve was around 13 m which led to a reasonable depth of interest of about 6.5 m. Inversions based on the starting soil profiles, created with the two approaches from Chapter 4.2.2, were performed. The results, using the first approach with increasing layer thickness over the depth for the starting soil profile, are presented in this section. Figure 105 shows the alignment between the theoretical (empty red circles) and experimental (solid blue circles) dispersion curves. The results of the second approach also

showed a good match and a low RMS error (Figure 116 (h), Chapter 7). However, compared to the boring log from GZA the results of the first approach agree more to the boring log.

Figure 105: Global representative dispersion curve (solid blue circles) and theoretical dispersion curve (empty red circles) (phase velocity [m/s] vs. wavelength [m]) for the boring location GZA-8 on URI main campus

The shear velocity profile (Figure 106) shows similarities to the boring log. The first 1.8 m of the boring log consist of topsoil and fill made of sand and gravel. In the shear wave velocity profile values typical for soft sand as well as dense gravel were identified. However, the jump in the shear wave velocity at a depth of about 1 m does not fit into the general course of the profile. Due the atypical low value, a mistake in the calculation can be assumed. After about 1.5 m the shear wave velocity increases to values of 300 to nearly 900 m/s. At a depth of 7 m shear wave velocities higher than 700 m/s were determined which indicate the existence of rock. In the boring log the boring was refused at a depth of 6.2 m. The soil above was marked as till which due to Ryden (2004) has shear wave velocities of 300 to 750 m/s. Hence, the soil above the determined rock can be characterized as till.

Figure 106: Shear wave velocity profile for location GZ-8 on URI Main Campus

CHAPTER 7

Discussion and Comparison of Shear Wave Velocity Profiles

In this chapter the SASW test results from this study are compared to boring logs or other SASW test results. Additionally, the results using global or array inversion and the influence of the initial layer thickness are analyzed.

7.1 Middleton Building

The results of all SASW tests on March 31 in forward and reverse direction and the SASW test from Groenewold (2015) are shown in Figure 107. In all three tests rock was indicated. The forward test and Groenewold's test show shear wave velocities of 500 m/s or higher at depths of 2 m and 3.5 m. For the reverse test, shear wave velocities of 500 m/s were found in the top layer and at around 2 m. In deeper layers the velocity stayed nearly constant at a value of about 500 m/s.

Figure 107: Shear wave velocity profiles obtained from tests in forward direction (black), reverse direction (red) and by Groenewold (2015) (cyan)

For the SASW test performed on April 13, five signals were recorded for each spacing. Therefore, five dispersion curves and shear wave velocity profiles were calculated for this test. The comparison of the five dispersion curves for each spacing in Figure 68 in Chapter 6 showed a good agreement between the dispersion curves. Figure 108 shows the five shear wave velocities in different colors and the shear wave profiles from the SASW test on March 31st and from Groenewold (2015) in dashed lines.

Figure 108: Shear wave velocity profiles of SASW tests from April 13th (solid lines), March 31st (red dashed line) and Groenewold (2015) (black dashed line)

In contrast to the close agreement of the dispersion curves, the five shear wave velocity profiles do not show any similarities in their trend or values. Rock was only identified with the second and fourth test at a depth of about 2 m and only with the fourth test in deeper layers. In comparison to the test results from March 31st and Groenewold (2015), only the fourth test (solid cyan line) shows similar shear wave velocities at a depth of 3.5 m. It is unclear why, with such close agreement between the dispersion curves, the resulting shear wave velocities are so different,

particularly at deeper depths.

7.2 Baker Pines Road Bridge

The SASW test results at the Baker Pines Road Bridge were compared to the boring log 595-5 provided by the RIDOT. The boring log in Figure 109 a) shows an initial layer of topsoil, followed by a layer of very dense sand and granite at a depth of 2.74 m. In Figure 109 b) the shear wave velocity plots calculated for the two SASW tests, Location 1 (solid black line) and Location 2 (dashed red line), are shown.

Figure 109: a) Boring log 595-5 and b) Shear wave velocity profile at Location 1 (black) and Location 2 (red) at the Baker Pines Road Bridge

The shear wave velocity profile of the Location 1 shows velocities of 400 m/s within the first meter, which do not conform to the loose layer in the boring log. However, during the SASW test very dense topsoil was identified at Location 1 instead of the loose soil indicated in the boring log. In fact, the spikes used to anchor the 4.5 Hz geophones had to be removed because it was not possible to push them into the soil. At both locations rock was identified at depths of around 3 m at the Location 2 and at 3.5 m at the Location 1. Compared to the boring log,

the difference to the actual depth of granite is 0.5 to 1 m. However, in general the shear wave velocity profile matches reasonably well with the stratigraphy shown in the boring log.

7.3 Weaver Hill Road Bridge

The SASW test results from the Weaver Hill Road Bridge were compared to the boring logs provided by the RIDOT. Figure 110 shows the results of the SASW test at the first location and the corresponding boring log WH-3. In the boring log, top of rock was defined at a depth of 3.35 m, overlaid with a very dense layer of Sand and a layer of topsoil (Figure 110 a)). The shear wave velocity profile in Figure 110 b) shows dense to very dense soils in the first 5 m. At a depth of 5 m, a shear wave velocity of 500 m/s (rock) was identified. In comparison to the boring log, the top of rock was found about 2 m deeper with the SASW test. However, again the trend of the shear wave velocity profile (dense sand and gravel overlying rock) is consistent with the description from the boring log.

Figure 110: a) Boring log WH-3 and b) Shear wave velocity profile at Location 1 at the Weaver Hill Road Bridge

The shear wave velocity profile of the second location was compared to the boring log WH-8 (Figure 111). In the boring log a layer of topsoil, followed by a layer of medium dense sand and the top of rock at a depth of 3.35 m was found. Up to a depth of 2.5 m the shear wave velocity profile in Figure 111 b) shows values typical for dense gravel. For the deeper layers shear wave velocity higher than 500 m/s, typical for rock, were determined. In this SASW test the calculated top of rock was shallower than in the boring log, but again the agreement between the log and the shear wave profile is reasonable..

Figure 111: a) Boring log WH-3 and b) Shear wave velocity profile at Location 2 at the Weaver Hill Road Bridge

7.4 Blind Prediction of Depth to Rock at the URI Main Campus

The three SASW tests performed on the URI main campus were compared to the corresponding boring logs provided by GZA Geoenvironmental, Inc.

Figure 112 a) shows the boring log GZA-4 and Figure 112 b) the shear wave profile determined in this study. The medium dense to dense layers, as well as the loose layers of fill are reflected in the shear wave velocity profile. At a depth of around 2.5 m typical shear wave velocities for rock (900 m/s) were found. The boring log shows a very dense sand layer at that depth followed top of rock at a depth of 2.8 m. The top of rock in the shear wave velocity profile was therefore

identified in shallower depth. Even though the following layers can be expected to consist of rock, the shear wave velocity decreases in the profile. It is not clear whether the very low velocity of 90 m/s at 1 m and the high velocity of 900 m/s at 2.5 m are reasonable. Most of the velocities suggest dense to very dense soils, which is consistent with the log.

Figure 112: a) Boring log GZA-4 and b) Shear wave velocity profile at the boring GZA-4 on the URI main campus

The boring log and shear wave velocity profile for the location GZA-7 can be seen in Figure 113. The shear wave velocity profile shows higher velocities of 100 to 350 m/s within the first meter which can be related to the medium dense gravel and sand layers shown in the boring log. The decrease of the shear wave velocity to a depth of 2.5 m and the following increase of the velocity corresponds

to the sand and till layers in the boring log. At a depth of 4.2 m the boring was terminated due to rock. Since the shear wave profile did not reach further than 3.5 m, no rock was identified with the SASW test. At this location, the blind SASW prediction does not match well with the boring log. In particular, the shear wave velocity suggests looser material than indicated in the boring. Of some concern is the very thin "spike" at a depth of 0.4 m and the early reversals in the shear wave velocity at the initial depths.

Figure 113: a) Boring log GZA-7 and b) Shear wave velocity profile at the boring GZA-7 on the URI main campus

The third SASW test on the URI main campus was performed at the same location as the boring GZA-8. The shear wave velocity profile and the boring log are shown in Figure 114. The boring log shows a layer of topsoil, followed by

layers of medium dense and loose fill and very dense till. At a depth of 6.2 m the exploration was stopped due to bedrock. The shear wave velocities to a depth of 1 m correspond to the layers in the boring log. At a depth of 1 to 1.5 m the shear wave velocity decreases to very low values which could indicate the loose fill layer but have to be questioned due to the exceptionally low velocities. Uncertainties in the data processing or inversion could have led to these values. The shear wave velocities in the depth of 2 to 6 m can be classified as till. At a depth of about 6 m, the shear wave velocity increase to values higher than 500 m/s which corresponds to the existence of rock. At this location, the trend of the shear wave velocity profile (medium dense overlying loose sand overlying very dense sand and gravel/till) is in good agreement with the boring log. However the actual velocities may not be reasonable, particularly the low values of 20 m/s at a depth of 1 m.

Figure 114: a) Boring log GZA-8 and b) Shear wave velocity profile at the boring GZA-8 on the URI main campus

In Table 16 the identified depths of rock are summarized for each test site. In addition to the SASW test results calculated in this study, the depths of bedrock found by drilling or previous SASW tests can be seen. The accuracy of the SASW test results are described qualitatively in the last column.

Table 16: Summary of bedrock depth identified at the five test sites

Test site		Boring log/ pre- vious SASW test	Global Inversion		Array Inver- sion	Accuracy
			Increasing initial layer thickness	Constant initial layer thick- ness		
Middleton Building March 31, 2016	forward direc- tion	3.7m	2m & 3.5m	1.5m & 3.3m	3m	reasonable
	reverse direc- tion	3.7m	0.5m & 2.1m	3.5m & 7m	not per- formed	poor
Middleton Building April 13, 2016	forward direc- tion	3.7m	no rock	no rock	not per- formed	poor
	reverse direc- tion	3.7m	no rock	0.5m to 1m	not per- formed	poor
Baker Pines Rd Bridge	Location 1	2.74m	3.7m	4m	not per- formed	poor
	Location 2	2.74m	3.2m	2m & 3.5m	not per- formed	poor
Weaver Hill Rd Bridge	Location 1	3.35m	5m	0.5m, 1m & 2.5m	not per- formed	poor
	Location 2	3.35m	2.5m	2.25m	not per- formed	poor
URI main campus	GZA-4	2.8m	2.5m	2m	3.1m	reasonable
	GZA-7	4.2m	no rock	no rock	not per- formed	poor
	GZA-8	6.2m	6m	3m & 5m	not per- formed	reasonable

7.5 Global vs. Array Approach for Modeling the Dispersion Curve

The inversion analysis for two SASW tests, Middleton Building and GZA-4, were performed with both the global and the array inversion, to investigate possible differences in the resulting shear wave velocity profiles.

Figure 115 a) shows the shear wave velocity profiles for the Middleton Building and Figure 115 b) the profiles for the location GZA-4 with the results from the global inversion in solid black lines and the results from the array inversion in dashed red lines. The two profiles calculated with the two different methods show similar trends with small differences for both SASW tests. At the Middleton building, rock was identified at a depth of about 3.5 m for both inversion methods. At the location GZA-4 the depth of rock differs by 0.5 m between the two inversion methods.

(a)

(b)

Figure 115: Shear wave velocity profile calculated with the global inversion (black) and array inversion (red) for tests at a) Middleton Building on March 31, 2016 and b) GZA-4 on URI main campus

According to the shear wave velocity profiles, there is no major difference in the accuracy of the result of the global or array inversion. Considering the match between the experimental and theoretical dispersion curves and the RMS error, the global inversion showed better results with a better match and lower RMS error. Additionally, the calculation time of the global inversion was significant lower than with the array inversion.

7.6 Influence of the Initial Layer Thickness

Two different approaches were considered for the definition of the initial layer thickness in the starting soil profile. In the first approach the layer thickness increases with depth and in the second approach the initial layer thickness is set to constant value of 0.5 m. To investigate the influence of the initial layer thickness on the shear wave velocity profile, inversions with both approaches were performed for every test. Figure 116 shows the shear wave velocity profiles for both approaches for every test.

(a) Middleton Building

(b) Baker Pines Rd Bridge, Loc 1

(c) Baker Pines Rd Bridge, Loc 2 (d) Weaver Hill Rd Bridge, Loc 1

(e) Weaver Hill Rd Bridge, Loc 2 (f) URI main campus, GZA-4

(g) URI main campus, GZA-7

(h) URI main campus, GZA-8

Figure 116: Shear wave velocity profile with increasing initial layer thickness of depth (black) and constant initial layer thickness over depth (red) for every SASW test

The comparison of the shear wave velocity profiles of the two approaches for the Middleton Building (Figure 116 a)), Baker Pines Road Bridge, Location 1 (Figure 116 b)) and boring location GZA-4 (Figure 116 f)) showed only small differences and a general similar trend of the profiles. Rock was identified within the same depth for each inversion at these three tests.

At the other six tests sites, layers with very different shear wave velocities were determined with the two different initial layer thickness approaches. Especially the shear wave velocity profiles of the Location 1 at the Weaver Hill Road Bridge (Figure 116 d)) disagreed over most parts of the investigation depth. The approach with constant layer thickness over depth led to shear wave velocity profiles in three cases, Location 2 at the Baker Pines Road Bridge, Location 1 at the

Weaver Hill Road Bridge and boring location GZA-8 on URI main campus, that showed layers of rock which could not be identified in the corresponding boring log. Except for the results of the boring location GZA-4 on the URI main campus, the shear profiles obtained with an starting soil profile with increasing layer thickness over depth resolved in more accurate results with a better matching experimental and theoretical dispersion curve, lower RMS error and better accordance to the provided boring logs.

CHAPTER 8

Conclusion

The primary objective of this thesis was to determine the efficiency of the SASW test for the identification of shallow rock. SASW tests were conducted at three different locations where rock was identified in previous studies or with different surveying methods. A commercial SASW system manufactured by Olson Instruments, Inc., consisting of two pairs of geophones (2Hz and 4.5Hz), a data acquisition system (NDE 360 platform), and softwares for filtering and masking the data and developing the dispersion curves and performing the inversions (WinTFS and WinSASW), were used.

For the test sites at Baker Pines Road Bridge, Weaver Hill Road Bridge and on the URI main campus boring logs were provided by the RIDOT or a geotechnical consulting company. The shear wave velocity profiles computed for this site were compared to the given logs. The results show, that it is difficult to determine the exact material and thickness of each layer with the SASW system, but a general trend which agrees to the provided information is visible. The SASW test on the URI main campus were performed without any knowledge about the layering. As it can be seen in Chapter 6, the results for all three tests on the URI main campus are comparable the informations of the boring logs.

Two different approaches for determining the starting soil profile for the inversion analysis were introduced. The first one (using initial layers of increasing thickness with depth) was based on previous studies (e.g. Joh, 1996) and the second one (using an initial layer thickness of 0.5 m) related to a typical layout of boring logs or soil profiles. The shear wave profiles determined with the first approach resolved in more accurate results in the majority of the tests. Since the

second approach did not improve the calculation time, the first approach should be used in future studies.

In summary, it was possible to determine rock layers with the SASW system. The process to reach the shear velocity requires considerable experience and personal judgment and especially the masking procedure is a source of mistakes and uncertainty.

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Appendix A

Figure A.1: Situation map of the three hybrid seismic / MASW lines (Frei, 2012)

Baker Pines Road Bridge

Figure B.1: Boring locations at the Baker Pines Road Bridge

American Drilling & Boring Co., Inc.									
54 WATER STREET EAST PROVIDENCE, R I									
TO <u>E.P. Snow Assoc.</u>					ADDRESS <u>Prov., RI</u>				
PROJECT NAME <u>I-95 Baker Pines to Austin Farm</u>					LOCATION <u>Richmond-Exeter, RI</u>				
REPORT SENT TO <u>above</u>					PROJ. NO. <u>F.A. I-95-1(11)9</u>				
SAMPLES SENT TO <u>"</u>					OUR JOB NO. <u>65-121</u>				

GROUND WATER OBSERVATIONS				CASING		SAMPLER	CORE BAR	SURFACE ELEV. <u>293.26</u>
At <u>13'</u>	after <u>1/4</u> Hours	Type			<u>S/S</u>	<u>S/T</u>	DATE STARTED <u>7/6/65</u>	
cas. <u>10'</u>		Size I.D.	<u>2 1/2"</u>	<u>18/8"</u>	<u>1-3/8"</u>		DATE COMPL. <u>7/6/65</u>	
At <u>13'4"</u>	after <u>1/2</u> Hours	Hammer Wt	<u>300#</u>	<u>140#</u>	BIT		BORING FOREMAN <u>C.L. Koehler</u>	
cas. <u>10'</u>		Hammer Fall	<u>24"</u>	<u>30"</u>	<u>Diamond</u>		INSPECTOR	
								SOILS ENGR.

LOCATION OF BORING:												
DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler			Moisture Density or Consist	Strata Change Elev.	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE		
				From 0-6"	6-12"	12-18"				No	Pen	Rec
10	1	0'-1'6"	D	1	2	2	dry loose		Brown TOP SOIL, Loam, Bldr's.	1	18"	12"
	18							2'-6"				
	23								Light brown medium to fine SAND, trace silt & coarse to fine gravel			
	97											
	200	5'-6'6"	D	18	29	38	dry v.d.			2	18"	16"
	275											
	300											
	412							9'-0"				
	650	8'-10'	D**	53	87		min./ft	10'-0"	Decomposed GRANITE	3	12"	-
		10'-15'	C				1		Wash shows Decomposed Bnk to Gray-white GRANITE, very coarse & weathered		60"	0"
20							1					
							1					
							1					
							1	15'-0"				
		15'-20'	C				1		Gray-white GRANITE Very seamy Medium hard	C	60"	15"
							1					
							1					
							1					
		20'-25'	C				1 1/2		"	C	60"	50"
							1 1/2					
						1 1/2	25'-0"					
						1 1/2		Bottom of Boring 25'-0"				

GROUND SURFACE TO <u>10'</u>		USED <u>2 1/2"</u>		CASING: THEN <u>C to 25'</u>	
Sample Type	Proportions Used	140 lb Wt. x 30" fall on 2" O.D. Sampler		SUMMARY:	
D=Dry C=Cored W=Washed	trace 0 to 10%	Cohesionless Density	Cohesive Consistency	Earth Boring <u>10'</u>	
UP=Undisturbed Piston	little 10 to 20%	0-10 Loose	0-4 Soft 30+ Hard	Rock Boring <u>15'</u>	
TP=Test Pit A=Auger V=Vane Test	some 20 to 35%	10-30 Med. Dense	4-8 M/Stiff	Samples <u>3</u>	
UT=Undisturbed Thinwall	and 35 to 50%	30-50 Dense	8-15 Stiff	HOLE NO <u>596-5</u>	
		50+ Very Dense	15-30 V-Stiff		

Figure B.2: Boring log of the hole number 596-5 at the Baker Pines Road Bridge

Weaver Hill Road Bridge

Figure B.3: Boring locations at the Weaver Hill Road Bridge

UNIVERSITY ENGINEERING SERVICES, INC.										SHEET <u>1</u> OF <u>1</u>		
76 Errol Street SOIL SAMPLING & TESTING Warwick 5, R.I.										DATE <u>9-24-65</u>		
PERFORMED FOR: <u>E. P. Snow Associates</u>										HOLE NO. <u>WH-3</u>		
PROJECT NAME: <u>Providence, Rhode Island</u>										LINE/STA <u>2-73</u>		
PROJECT NO: <u>1-80-1 (11)</u>										OFFSET <u>WT 40'</u>		
OUR JOB NO: <u>1001</u>												
SURFACE ELEVATION <u>132.80</u>		CASING TYPE <u>3/8"</u>		SAMPLER <u>3/8"</u>		CORE BAR <u>3/8"</u>		DATE STARTED <u>9-24-65</u>				
GROUND WATER DEPTH <u>No Data</u>		SIZE I.D. <u>3 1/8"</u>		HAMMER WT <u>600#</u>		BIT <u>140#</u>		DATE COMPLETED <u>9-24-65</u>				
		HAMMER FALL <u>24"</u>				DIAMOND <u>30"</u>		BORING FOREMAN <u>C. Carlson</u>				
								INSPECTOR <u>H. Potter</u>				
								SOILS ENGR.				
LOCATION OF BORING: <u>Inter. East Greenwich</u>												
DEPTH	Casing Blows Per Foot	Sample Depths From - To	Type of Sample	Blows Per 6" On Sampler			Moisture Density or Consistency	Strata Change Elev.	SAMPLE			SOIL IDENTIFICATION Remarks include color, gradation, type of soil, etc., Rock color, type, condi- tion, hardness, drilling time, etc.
				0-6	6-12	12-18			NO.	PEN	REC.	
0								330.60				Top Soil
5		4' 5" - 6"	D	13	10	14	Dry Very Dense		1	10	10	BR P-C Sand and fine gravel
10		8' - 10' - 0"	D	12	10	10	"	321.60	1	10	10	
			C	1	7							Top of Rock
				2	9							3m Gray weathered massy granite
				3	9							
				4	10							
				5	8			316.60				
												Bottom of Boring 16'
GROUND SURFACE TO				USED		CASING		THEN		TOTALS		
SAMPLE TYPE				Proportions Used		140 lb wt X 30' fall on 2" O.D. Sampler		Cohesive Consistency		Earth Boring		
D= Dry C= Cored W= Washed				Trace 0 to 10%		0-10 Loose		0-4 Soft 31+ Hard		11		
UP= Undisturbed Piston				Little 11 to 20%		11-30 Med Dense		5-8 Medium Stiff		Rock Coring		
TP= Test Pit A= Auger V= Vane Test				Some 21 to 35%		31-50 Dense		9-15 Stiff		8		
UT= Undisturbed Shelby				And 36 to 50%		51+ Very Dense		16-30 Very Stiff		Samples		
										HOLE NO. <u>WH-3</u>		

PROPOSED BOTTOM OF FOOTING NORTH ABUTMENT
SOUTHBOUND ROADWAY - EL. 330.00

Figure B.4: Boring log of the hole number WH-3 at the Weaver Hill Road Bridge

URI Main Campus

GZA-4

TEST BORING LOG
GZA GeoEnvironmental, Inc. Engineers and Scientists						Ballinger URI New Engineering Building Kingston, Rhode Island				EXPLORATION NO.: GZ-4 SHEET: 1 of 1 PROJECT NO: 34214.00 REVIEWED BY: D. Baxter					
Logged By: A. DiGiantommaso Drilling Co.: New England Boring Foreman: J. Stokes						Type of Rig: Truck Rig Model: Failing Strata Star Drilling Method: Drive-Wash		Boring Location: See Plan Ground Surface Elev. (ft.): 208 Final Boring Depth (ft.): 9.5 Date Start - Finish: 6/3/2016 - 6/3/2016				H. Datum: V. Datum:			
Hammer Type: Safety Hammer Hammer Weight (lb.): 140 Hammer Fall (in.): 30 Auger or Casing O.D./I.D Dia (in.): 4"						Sampler Type: SS Sampler O.D. (in.): 2.0 Sampler Length (in.): 24 Rock Core Size:		Groundwater Depth (ft.)							
								Date		Time		Water Depth		Stab. Time	
Depth (ft)	Casing Blows/ Core Rate	Sample					SPT Value	Sample Description and Identification (Modified Burmister Procedure)	Remark	Field Test Data	Depth (ft.)	Stratum Description	Elev. (ft.)		
		No.	Depth (ft.)	Pen. (in)	Rec. (in)	Blows (per 6 in.)									
5		S-1	0-2	24	9	9 20 39 22	59	S-1 : Very dense, brown, fine SAND, some Silt (Dry)			0.5	TOPSOIL	207.5		
		S-2	2-4	24	2	39 9 5 9	14	S-2 : Medium dense, brown, fine SAND, some coarse Gravel, trace Silt (Dry)	1						
		S-3	4-6	24	7	8 5 2 3	7	S-3 : Loose, brown, fine to Medium SAND some Gravel, little Silt, Ceramic Fragments (Wet)	2			FILL			
		S-4	6-8	24	8	20 7 7 15	14	S-4 : Loose, black, fine to medium SAND, some Ceramic Fragments, little Silt (Wet)	3						
		S-5	8-9.3	16	8	38 72 100	R	S-5 : Very dense, brown, fine to coarse SAND, some Gravel, trace Silt (Wet)			8	GRAVELLY SAND WITH SILT	200.0		
10								End of exploration at 9.5 feet.			9.5		196.5		
15															
20															
25															
30															
REMARKS 1 - Rock stuck in tip of spoon may have presented additional recovery. 2 - Fine gravel visible in wash as casing is cleaned. 3 - Yellow fine gravel sized ceramic encountered.															
See Log Key for explanation of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.												Exploration No.: GZ-4			

Figure B.6: Boring log of the location GZA-4 on the URI main Campus

GZA-7

TEST BORING LOG
GZA GeoEnvironmental, Inc. <i>Engineers and Scientists</i>						Ballinger URI New Engineering Building Kingston, Rhode Island				EXPLORATION NO.: GZ-7 SHEET: 1 of 1 PROJECT NO: 34214.00 REVIEWED BY: D. Baxter											
Logged By: A. DiGianfrancesco Drilling Co.: New England Boring Foreman: J. Stokes						Type of Rig: Truck Rig Model: Falling Strata Star Drilling Method: Drive-Wash		Boring Location: See Plan Ground Surface Elev. (ft.): 216 Final Boring Depth (ft.): 14.3 Date Start - Finish: 6/6/2016 - 6/6/2016				H. Datum: V. Datum:									
Hammer Type: Safety Hammer Hammer Weight (lb.): 140 Hammer Fall (in.): 30 Auger or Casing O.D./I.D Dia (in.): 4"						Sampler Type: SS Sampler O.D. (in.): 2.0 Sampler Length (in.): 24 Rock Core Size:		Groundwater Depth (ft.) <table><tr><th>Date</th><th>Time</th><th>Water Depth</th><th>Stab. Time</th></tr><tr><td></td><td></td><td></td><td></td></tr></table>						Date	Time	Water Depth	Stab. Time				
Date	Time	Water Depth	Stab. Time																		
Depth (ft)	Casing Blows/ Core Rate	Sample					SPT Value	Sample Description and Identification (Modified Burmister Procedure)	Remark	Field Test Data	Stratum Description	Elev. (ft.)	Depth (ft.)	SPT Value							
		No.	Depth (ft.)	Pen. (in)	Rec. (in)	Blows (per 6 in.)															
5		S-1	0-2	24	19	5 11 19 14	30	S-1 : Medium dense, brown, fine SAND, little Gravel, trace Silt, trace Root (Dry)					0.5		215.5						
		S-2	2-4	24	8	20 17 13 8	30	S-2 : Gray, ROCK FRAGMENTS (Dry)													
		S-3	4-6	24	7	16 6 5 6	11	S-3 : Medium dense, brown, fine to medium SAND, little Gravel, some Silt (Wet)													
		S-4	6-8	24	4	8 4 6 7	10	S-4 : Medium dense, brown, fine to medium SAND, some Silt, little Gravel, (Wet)													
10		S-5	9- 10.8	22	10	14 21 25 100	46	S-5 : Dense, gray, fine to coarse SAND and Gravel, little Silt (Wet)	1				9		207.0						
15		S-6	14- 14.3	4	3	100		S-6 : Dense, gray, fine to coarse SAND, some Gravel, little Silt (Wet)					14.3		201.7						
								End of exploration at 14.3 feet.													
20																					
25																					
30																					
REMARKS 1 - Gravel in tip of spoon.																					
See Log Key for explanation of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.												Exploration No.: GZ-7									

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Figure B.7: Boring log of the location GZA-7 on the URI main Campus

GZA-8

TEST BORING LOG
GZA GeoEnvironmental, Inc. <i>Engineers and Scientists</i>							Ballinger URI New Engineering Building Kingston, Rhode Island				EXPLORATION NO.: GZ-8 SHEET: 1 of 1 PROJECT NO: 34214.00 REVIEWED BY: D. Baxter																																																																																																																																																																																						
Logged By: A. DiGiantommaso Drilling Co.: New England Boring Foreman: J. Stokes							Type of Rig: Truck Rig Model: Faling Strata Star Drilling Method: Drive-Wash			Boring Location: See Plan Ground Surface Elev. (ft.): 221 Final Boring Depth (ft.): 20.4 Date Start - Finish: 6/7/2016 - 6/7/2016			H. Datum: V. Datum:																																																																																																																																																																																				
Hammer Type: Safety Hammer Hammer Weight (lb.): 140 Hammer Fall (in.): 30 Auger or Casing O.D./I.D Dia (in.): 4"							Sampler Type: SS Sampler O.D. (in.): 2.0 Sampler Length (in.): 24 Rock Core Size:			Groundwater Depth (ft.)																																																																																																																																																																																							
										Date		Time		Water Depth		Stab. Time																																																																																																																																																																																	
<table><thead><tr><th rowspan="2">Depth (ft.)</th><th rowspan="2">Casing Blows/ Core Rate</th><th colspan="6">Sample</th><th rowspan="2">Sample Description and Identification (Modified Burmister Procedure)</th><th rowspan="2">Remark</th><th rowspan="2">Field Test Data</th><th rowspan="2">Stratum Description (ft.)</th><th rowspan="2">Stratum Elev. (ft.)</th></tr><tr><th>No.</th><th>Depth (ft.)</th><th>Pen. (in.)</th><th>Rec. (in.)</th><th>Blows (per 6 in.)</th><th>SPT Value</th></tr></thead><tbody><tr><td rowspan="4">5</td><td></td><td>S-1</td><td>0-2</td><td>24</td><td>15</td><td>3 11 10 6</td><td>21</td><td>S-1 : Medium dense, brown, fine to medium SAND, little Gravel, trace Silt (Dry)</td><td></td><td></td><td>0.5</td><td>TOPSOIL</td><td>220.5</td></tr><tr><td></td><td>S-2</td><td>2-4</td><td>24</td><td>13</td><td>6 4 4 3</td><td>8</td><td>S-2 : Loose, brown, fine to coarse SAND, some Gravel, little Silt (Dry)</td><td></td><td></td><td></td><td>FILL</td><td></td></tr><tr><td></td><td>S-3</td><td>4-6</td><td>24</td><td>9</td><td>9 5 4 11</td><td>9</td><td>S-3 : Loose, fine to coarse SAND, little Gravel, little Silt (Moist)</td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td>S-4</td><td>6-8</td><td>24</td><td>18</td><td>21 20 39 48</td><td>59</td><td>S-4 : Very dense, gray, fine SAND, trace Silt (Moist)</td><td></td><td></td><td>6</td><td></td><td>215.0</td></tr><tr><td rowspan="2">10</td><td></td><td>S-5</td><td>9-11</td><td>24</td><td>16</td><td>26 29 29 41</td><td>58</td><td>S-5 : Very dense, gray, fine to coarse SAND, some Gravel, little Silt (Moist)</td><td></td><td></td><td></td><td>TILL</td><td></td></tr><tr><td></td><td>S-6</td><td>14-16</td><td>24</td><td>15</td><td>25 25 49 43</td><td>74</td><td>S-6 : Very dense, gray, fine to coarse SAND, and Gravel, little Silt (Moist)</td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="2">20</td><td></td><td>S-7</td><td>19- 20.4</td><td>17</td><td>10</td><td>43 75 100</td><td>R</td><td>S-7 : Very dense, gray, fine to medium SAND, and Gravel, little Silt (Moist)</td><td></td><td></td><td>20.4</td><td></td><td>200.6</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>End of exploration at 20.4 feet.</td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="2">25</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="2">30</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr></tbody></table>														Depth (ft.)	Casing Blows/ Core Rate	Sample						Sample Description and Identification (Modified Burmister Procedure)	Remark	Field Test Data	Stratum Description (ft.)	Stratum Elev. (ft.)	No.	Depth (ft.)	Pen. (in.)	Rec. (in.)	Blows (per 6 in.)	SPT Value	5		S-1	0-2	24	15	3 11 10 6	21	S-1 : Medium dense, brown, fine to medium SAND, little Gravel, trace Silt (Dry)			0.5	TOPSOIL	220.5		S-2	2-4	24	13	6 4 4 3	8	S-2 : Loose, brown, fine to coarse SAND, some Gravel, little Silt (Dry)				FILL			S-3	4-6	24	9	9 5 4 11	9	S-3 : Loose, fine to coarse SAND, little Gravel, little Silt (Moist)							S-4	6-8	24	18	21 20 39 48	59	S-4 : Very dense, gray, fine SAND, trace Silt (Moist)			6		215.0	10		S-5	9-11	24	16	26 29 29 41	58	S-5 : Very dense, gray, fine to coarse SAND, some Gravel, little Silt (Moist)				TILL			S-6	14-16	24	15	25 25 49 43	74	S-6 : Very dense, gray, fine to coarse SAND, and Gravel, little Silt (Moist)						20		S-7	19- 20.4	17	10	43 75 100	R	S-7 : Very dense, gray, fine to medium SAND, and Gravel, little Silt (Moist)			20.4		200.6								End of exploration at 20.4 feet.						25																											30																										
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Figure B.8: Boring log of the location GZA-8 on the URI main Campus

Appendix C

Beach sites

Three beach sites have been added to this study to continue an ongoing collection of shear wave velocity data on coastal beach sites in Rhode Island where coastal erosion occurred.

URI Bay Campus beach

The first site is the beach at the URI Bay Campus in Narragansett facing the Narragansett bay. In 1690's the area of the URI Bay Campus Beach was called South Ferry and was used as a pier for a ferry service to Jamestown (Rhode Island Historical Preservation Commission, 1999).

The SASW test was conducted on May 12th, 2016. The position of the array of geophones used for the test is shown in Figure C.1.

Figure C.1: Test site at the URI Beach (Google Maps)

The array ran parallel and in a distance of about 4 m to the Narragansett Bay. The CRMP geometry, with a south moving source and first geophone and a north moving second geophone, was used. The spacing for the 4.5 Hz geophones

were set to 0.5, 1, 1.5, 2, 3, 4, 5 and 6 m and for the 2 Hz geophones to 5, 6, 7 and 8 m. Further informations about the applied test parameters can be found in Table C.1.

Matunuck Beach, RI

The Matunuck Beach in Matunuck, Rhode Island is one of many significant erosion zones at the shoreline of Rhode Island. Great amounts of sand have been washed away in the last years which does not only cause a problem for residents living right next to the beach but also endangers the accessibility of the only road to get in and out of Matunuck. Several strategies have been proposed to prevent further erosion. To investigate if the coastal erosion can be monitored with the SASW system, the Matunuck Beach was added to the collection of beach test sites. The first SASW test on Matunuck Beach was conducted on June 5th, 2016. The test site was located in front of a sand notch at the east end of the beach (Figure C.2). The array ran parallel the to the shoreline and was set up based on the CRMP geometry (Figure C.3). In the test receiver spacings of 0.5, 1, 1.5, 2, 3, 4, 5 and 6 m for the 4.5 Hz geophones and to 5, 6, 7 and 8 m for the 2 Hz geophones were used. Table C.2 summarizes the applied test parameter fore each spacing.

Figure C.2: Test site at the Matunuck Beach (Google Maps)

Figure C.3: Test site at the Matunuck Beach with 4.5 Hz geophones (facing west)

Misquamicut Beach, Westerly, RI

The last test on a beach site was performed at the Misquamicut Beach in Westerly, Rhode Island on June 8th, 2016. Misquamicut Beach was replenished by the U.S. Army Corps of Engineering in 2013 and experienced significant erosion in the following years. The beach consists of outwash with medium to coarse sands and gravel and fine sands, silts and clays. On October 3rd, 2015, Groenewold (2015) conducted a SASW test on Misquamicut Beach with the first geophone located at the coordinates 41.322667° North and 71.805001° West. To be able to compare the results, the test for this study was conducted at the same location. In Figure C.18 the marker shows the position of the midpoint of the CRMP geometry method and the red line represents the array of geophones. The midpoint was located close (about 13 m) away from the Entrance 4, at the coordinates 41.322656° North and 71.805026° West. The linear array ran parallel to the Atlantic Ocean and a fence separating the parking lot and the beach. Spacings of 0.5, 1, 1.5,

2, 3, 4, 5 and 6 m were used for the tests with 4.5 Hz geophones and 5, 6, 7 and 8 m for the tests with 2 Hz geophones. The set-up for a test with the 2 Hz geophones is displayed in Figure C.5. The applied test parameter for each spacings are summarized in Table C.3.

Figure C.4: Test site at the Misquamicut Beach (Google Maps)

Figure C.5: Test site at the Misquamicut Beach with 2 Hz geophones

Test results

At the beach sites shear velocities of soft to dense sand are expected. In the literature, for example Foti (2015), Ryden (2004) and Stokoe et al. (2005), shear wave velocities of 100 to 500 m/s are assumed for sand. An estimation of the shear wave velocities of soft sand, silt and clay according to Lin et al. (2014) will be included in the results.

URI Bay Campus Beach, Narragansett

The Beach at the URI Bay Campus has not been investigated with the SASW system before and therefore, no soil profiles or shear wave velocity data exist. The testing parameters and equipment of the first SASW test on the URI Bay Campus Beach can be seen in Table C.1.

Table C.1: Summary of testing parameters for the test beach at the URI Bay Campus

Test #	Spacing [m]	Source	Geophone [Hz]	NDE file	Gain	Scale	Disp.- Curve
1	0.5	SS	4.5	287	100	71%	No
2	1	SS	4.5	288	100	75%	No
3	1.5	SS	4.5	289	100	75%	No
4	2	BS	4.5	290	100	77%	Yes
5	3	BS	4.5	291	100	76%	Yes
6	4	BS	4.5	292	1000	72%	Yes
7	5	SS	4.5	293	1000	82%	Yes
8	6	SS	4.5	294	1000	78%	Yes
9	5	SS	2	295	1000	79%	Yes
10	6	SS	2	296	1000	75%	Yes
11	7	SS	2	297	1000	76%	Yes
12	8	BS	2	298	1000	76%	No

with: SS = small sledgehammer (1 kg) and BS = big sledgehammer (4.5 kg)

The recorded data was processed and reviewed in WinTFS. No data had to be excluded from further analysis in WinSASW. Nevertheless, after the masking process in WinSASW the NDE files 287, 288, 289 and 298 had to be taken out of the calculation of the composite dispersion curves. The Figures C.6 and C.7 show the composite dispersion curves determined with the usable data.

Figure C.6: Composite dispersion curve (phase velocity [m/s] vs. wavelength [m]) of the URI Bay Campus Beach

Figure C.7: Composite dispersion curve (phase velocity [m/s] vs. frequency [Hz]) of the URI Bay Campus Beach

The composite dispersion curves were used to compute the global averaged dispersion curve with a maximum wavelength of 14 m. Due to Stokoe et al. (2005), an investigation depth half of the maximum wavelength can be assumed, hence 7 m.

Two inversion analysis were performed for the calculated global dispersion curve. The difference between the two analysis was the starting soil profile. The specification of the starting soil profile was based on two approaches which are explained in Chapter 4.2.2. In the following only the results of the first case will be considered.

The theoretical dispersion curve was computed with the global inversion analysis. In Figure C.8 the theoretical dispersion curve is displayed in empty red circles. Additionally, the experimental dispersion curve can be seen (solid blue circles) to show the match between the two curves.

Figure C.8: Global representative dispersion curve (solid blue circles) and theoretical dispersion curve (empty red circles) (phase velocity [m/s] vs. wavelength [m]) of the URI Bay Campus Beach

The resulting shear wave velocity of the theoretical dispersion curve and typical shear wave velocities for soft sands and dense clay are demonstrated in Figure C.9. Differently than expected, the profile does not only show shear wave velocities typical for sand. At a depth of 2 m the velocity increases up to a value of 800 m/s. These are velocities typical for very dense soils or rock. Since no other references or records exist, the result could not be validated.

Figure C.9: Shear wave velocity profile of URI Bay Campus Beach

Matunuck Beach, RI

As well as at the beach at the URI Bay campus, no shear wave velocity records or soil information exist for the Matunuck Beach in Rhode Island. The SASW test to investigate the shear wave velocity profile was conducted with the parameters listed in Table C.2.

Table C.2: Summary of testing parameters for the test at the Matunuck beach

Test #	Spacing [m]	Source	Geophone [Hz]	NDE file	Gain	Scale	Disp.- Curve
1	0.5	SS	4.5	390	100	74%	Yes
2	1	SS	4.5	391	100	73%	Yes
3	1.5	SS	4.5	392	100	74%	Yes
4	2	BS	4.5	393	100	76%	Yes
5	3	BS	4.5	394	100	76%	No
6	4	SS	4.5	395	1000	72%	No
7	5	SS	4.5	396	1000	75%	Yes
8	6	SS	4.5	397	1000	73%	Yes
9	5	SS	2	398	1000	75%	Yes
10	5	SS	2	399	1000	74%	Yes
11	6	SS	2	400	1000	77%	Yes
12	6	SS	2	401	1000	74%	Yes
13	7	SS	2	402	1000	73%	Yes
14	8	BS	2	403	1000	78%	No
15	8	BS	2	404	1000	72%	No

with: SS = small sledgehammer (1 kg) and BS = big sledgehammer (4.5 kg)

The WinTFS processed data showed a high coherence and good quality. Therefore, all data files could be imported into WinSASW for further analysis. However, after masking the records four files, NDE 394, 395, 403 and 404, were excluded from following calculations. The other eleven data files were used to determine composite dispersion curves which are illustrated in the Figures C.10 and C.11.

Figure C.10: Composite dispersion curve (phase velocity [m/s] vs. wavelength [m]) for the Matunuck Beach

Figure C.11: Composite dispersion curve (phase velocity [m/s] vs. frequency [Hz]) for the Matunuck Beach

A global dispersion curve was calculated from the composite dispersion curves. With a maximum wavelength of 12 m, the investigation depth comes 6 m (global dispersion curve (solid blue circles) in Figure C.12). The starting soil profile for this depth was again specified based on two concepts discussed in Chapter 4.2.2. For both cases a theoretical dispersion curve was determined using the global inversion analysis. The results of the first concept had the smaller RMS error and a better match between the theoretical and experimental dispersion curve (Figure C.12).

Figure C.12: Global representative dispersion curve (solid blue circles) and theoretical dispersion curve (empty red circles) (phase velocity [m/s] vs. wavelength [m]) for the Matunuck Beach

In Figure C.13 the shear wave velocity for the Matunuck Beach can be seen. The velocities range from 50 to 250 m/s which are typical shear wave velocities for soft to dense sand. The trend of the profile matches to the shear velocity profile of sand according to Lin et al. (2014).

Figure C.13: Shear wave velocity profile of Matunuck Beach

Misquamicut Beach, Westerly, RI

SASW tests have been conducted at the Misquamicut Beach by Groenewold (2015) in October 2015. The shear wave velocity profile (Figure C.14), consists of velocities from about 80 to 270 m/s which indicates the shear wave velocity range of sand.

Figure C.14: Shear wave velocity profile of Misquamicut Beach by Groenewold (2015)

The SASW test for this study has been performed at exact the same location and the parameters and equipment from Table C.3.

Table C.3: Summary of testing parameters for the test at the Misquamicut beach, Westerly

Test #	Spacing [m]	Source	Geophone [Hz]	NDE file	Gain	Scale	Disp.- Curve
1	0.5	SS	4.5	405	100	75%	Yes
2	0.5	SS	4.5	406	100	76%	Yes
3	1	SS	4.5	407	100	77%	No
4	1	SS	4.5	408	100	73%	Yes
5	1.5	BS	4.5	409	100	73%	Yes

6	1.5	BS	4.5	410	100	72%	Yes
7	2	BS	4.5	411	100	73%	Yes
8	2	BS	4.5	412	100	72%	Yes
9	3	SS	4.5	413	1000	74%	Yes
10	3	SS	4.5	414	1000	72%	No
11	4	SS	4.5	415	1000	76%	Yes
12	4	SS	4.5	416	1000	71%	No
13	5	SS	4.5	417	1000	77%	Yes
14	6	SS	4.5	418	1000	76%	Yes
15	6	SS	4.5	419	1000	73%	Yes
16	5	SS	2	420	1000	74%	Yes
17	5	SS	2	421	1000	76%	No
18	6	SS	2	422	1000	71%	No
19	6	SS	2	423	1000	72%	No
20	7	SS	2	424	1000	73%	Yes
21	7	SS	2	425	1000	74%	Yes
22	8	BS	2	426	1000	78%	Yes
23	8	BS	2	427	1000	74%	Yes

with: SS = small sledgehammer (1 kg) and BS = big sledgehammer (4.5 kg)

The recorded data was reviewed in WinTFS to make a preliminary decision if the data is usable for the inversion analysis. For the test at the Misquamicut Beach, every data file could be used for further processing. The NDE files were imported in WinSASW, masked and again reviewed. Data which did not match in the general trend were excluded. The remaining data files were used to compute the composite dispersion curves (Figures C.15 and C.16).

Figure C.15: Composite dispersion curve (phase velocity [m/s] vs. wavelength [m]) for the Misquamicut Beach

Figure C.16: Composite dispersion curve (phase velocity [m/s] vs. frequency [Hz]) for the Misquamicut Beach

In the next step, the global dispersion curve was determined. Figure C.17 shows the dispersion curve computed from the composite dispersion curves in solid blue circles. The maximum wavelength captured is about 10 m which gives an approximate investigation depth of 5 m. The procedures for setting up the starting soil profiles for the two considered approaches have been described in Chapter 4.2.2. For this test, the focus lied on the results of the first concept due to a much smaller RMS error. The computed theoretical dispersion curve can be found in Figure C.17.

Figure C.17: Global representative dispersion curve (solid blue circles) and theoretical dispersion curve (empty red circles) (phase velocity [m/s] vs. wavelength [m]) for the Misquamicut Beach

The comparison of the shear wave velocity profile from Groenewold (2015) and the profile computed in this study (Figure C.18) show a similar trend of the two profiles. In both cases the velocities range between 50 and 270 m/s. In the top layers the velocities computed in this study were a little bit higher which can be caused by compaction due to human activities at the beach.

Figure C.18: Shear wave velocity profile of Misquamicut Beach

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