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Initial Elastic Modulus Degradation Using Pressuremeter and Standard Penetration Test Results at Two Sites

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INITIAL ELASTIC MODULUS DEGRADATION USING PRESSUREMETER AND STANDARD PENETRATION TEST RESULTS AT TWO SITES

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ABSTRACT

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In-situ testing was performed at two sites consisting of pre-bored pressuremeter testing, seismic surface wave testing to develop a shear wave velocity profile, and Standard Penetration testing during the soil boring phase in order to evaluate the feasibility of using large shallow foundations for a project. This study focuses on a comparison of the in-situ direct measurements of soil stiffness obtained from this testing program. The small strain modulus obtained from the seismic surface wave test results is compared to the intermediate strain modulus obtained from both the initial loading pressuremeter modulus and reload pressuremeter modulus. The modulus calculated from blowcount correlations is compared to that of the pressuremeter modulus. The comparisons made from this relatively small data set reveal several trends within the data that are discussed and possible explanations posed. The results of the study are mostly inconclusive due to the small data set. Finally, recommendations are given to further investigate the trends that are revealed.

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CHAPTER 1 INTRODUCTION

1.1 - General

An accurate representation of soil stiffness is essential in order to accurately predict the deformation response of a soil subjected to a change in stress. Soil stiffness is the key parameter on which several types of analyses hinge, including elastic settlement analysis, which will be the focus of this study. Although the importance of accurately representing soil stiffness has been well established in the literature (Yamashita, Jamiolkowski, & Lo Presti, 2000), it is uncommon for direct measurements of soil stiffness to be performed either in the laboratory or in the field for typical geotechnical investigations. In place of the more accurate direct measurements, less reliable and more conservative correlations to penetration tests or index properties are often employed.

There are several reasons why direct measurements of soil stiffness are not regularly performed. Perhaps the strongest reason to rely on correlations in place of a direct measurement is that the methods available for a direct measurement are few, specialized, and are typically cost prohibitive. Also, although direct measurements are preferred for more accurate measurements of soil stiffness, they carry with them some limitations that may lead practicing engineers to believe that they are not a significant enough improvement over traditional methods to warrant their use. Most of these limitations relate to the soil disturbance that is unavoidable and difficult to quantify, particularly when performing these tests in the laboratory, but also when performing them in the field. Finally, the standard of practice in the United States, even for complicated geotechnical engineering problems, does not require the engineer to estimate elastic stiffness parameters for settlement analysis with anything more than a correlated index property and therefore, direct measurements are seen as non-essential.

As stated above, the standard of practice for the majority of geotechnical investigations is to estimate soil stiffness using correlations from index properties. The most common method is to correlate stiffness to Standard Penetration test (SPT) blowcounts. These correlations have been shown to be highly variable, lacking of a direct correlation, and generally conservative (Bellotti, Ghionna, Jamiolkowski, Lancellotta, & Manfredini, 1986). This is likely due to the large amount of scatter in the correlations, indicating a high level of uncertainty in the correlation. Another common practice is to estimate soil stiffness from the Unified Soil Classification System (USCS) soil classification. Ranges of stiffness for each soil classification have been published. Unfortunately, these ranges are very large, indicating the uncertainty and great variability in this correlation.

One obstacle to estimating soil stiffness is that the engineering parameter used to describe stiffness known as the modulus has been shown to be non-linear (not a constant value), even at very small strains (Fahey & Carter, 1992). Modulus at small strains is greater, often dramatically greater, than modulus at intermediate to large strains. Due to the potentially great variability in modulus depending on strain level, using a single modulus value to represent soil stiffness is not preferred. Unfortunately, using a single value is often unavoidable because the accepted methods available to the engineer to estimate settlement require a single value. If a single value is required, it is good practice to select a modulus value at a strain level that is likely to be encountered under working loads.

It has already been stated that the methods for developing the non-linear relationship between soil modulus and strain are typically cost prohibitive. It should also be noted that these methods are also laboratory methods that carry with them certain additional limitations. The most obvious, and most problematic of these limitations is that of soil disturbance due to sampling and transport. It has been shown that the initial modulus (maximum tangent modulus) estimated by laboratory methods such as Resonant Column Torsional Shear testing is often significantly less than those obtained by field measurements and this discrepancy is often related to sample disturbance (Fahey & Carter, 1992). Beyond the effects of soil disturbance, the samples obtained may not be of the correct size to accurately represent the soil conditions at the site. For the reasons stated above, field measurements of modulus are often considered superior to laboratory methods for conventional geotechnical analysis.

This study will analyze data collected at two sites consisting of geophysical testing results, conventional geotechnical soil borings, and pre-bored pressuremeter testing. The geophysical testing and pressuremeter testing provide field measurements of shear modulus at very small and intermediate strains, respectively. The soil borings which include both USCS soil classification testing and SPT blowcounts provide necessary index properties of the soil as well as penetration data which can be used with published correlations to estimate soil stiffness.

The original scope of testing was developed to evaluate the feasibility of using large width (40 feet or greater) shallow foundations for a heavily loaded structure. Therefore, this study will be focused on the key parameter required to determine this type of feasibility, the elastic modulus that would control elastic settlement, as consolidation type settlement was not anticipated at this site.

1.2 - Objectives

The main objective of this study will be to compare elastic modulus values calculated from shear wave velocity measurements obtained from seismic surface wave testing (small strain) and pressuremeter tests (intermediate strain) with an attempt at developing a means for which the intermediate strain modulus can be estimated when the small strain modulus is known. The intermediate strain level obtained from the pressuremeter is assumed to be the most reliable value and the value that the engineer should attempt to obtain for use in an elastic settlement analysis. Once a means of degrading the modulus to intermediate strain levels has been developed, the correlations presented in the literature for correlating standard penetration test results to elastic modulus will be compared to the results of the degradation.

1.3 - Methodology

This study will begin with a discussion of relevant background information about elastic soil modulus and the methods by which it is measured. Next, the methods used for and results of the data collection will be presented. An analysis of the results will then be presented and discussed with the goal of satisfying the previously listed study objectives. Finally, conclusions of the analysis will be presented.

CHAPTER 2 BACKGROUND

2.1 - General

The stiffness of a soil is represented by an engineering parameter termed a modulus. There are several different types of modulus that can be measured for soils, including the elastic modulus, shear modulus, constrained modulus, and bulk modulus. Each type of modulus is appropriate for different types of analyses and is determined in different ways. For the purposes of this study, the elastic modulus, which is the modulus of a soil in triaxial compression (Briaud, 2001), will be the modulus that is preferred because the elastic modulus is the modulus most typically used in standard deformation analyses. The elastic modulus is also the modulus that is most commonly reported from the results of the pressuremeter test. The definition of elastic modulus is given in the following equation:

$$E = \sigma/\varepsilon$$

Where E is the elastic modulus, σ is the level of axial stress, and ε is the level of axial strain. The elastic modulus equation above assumes that the soil is isotropic and homogeneous within each soil layer assigned.

In engineering practice, there are several definitions of the elastic modulus, the most common of which are the initial tangent modulus and secant modulus. Figure 2.1 is a graphical representation of the definition of both the initial tangent elastic modulus and the secant elastic modulus, which is variable according to the strain level. Other definitions of modulus that can be reported are the unload modulus, reload modulus, and cyclic modulus (Briaud, 2001).

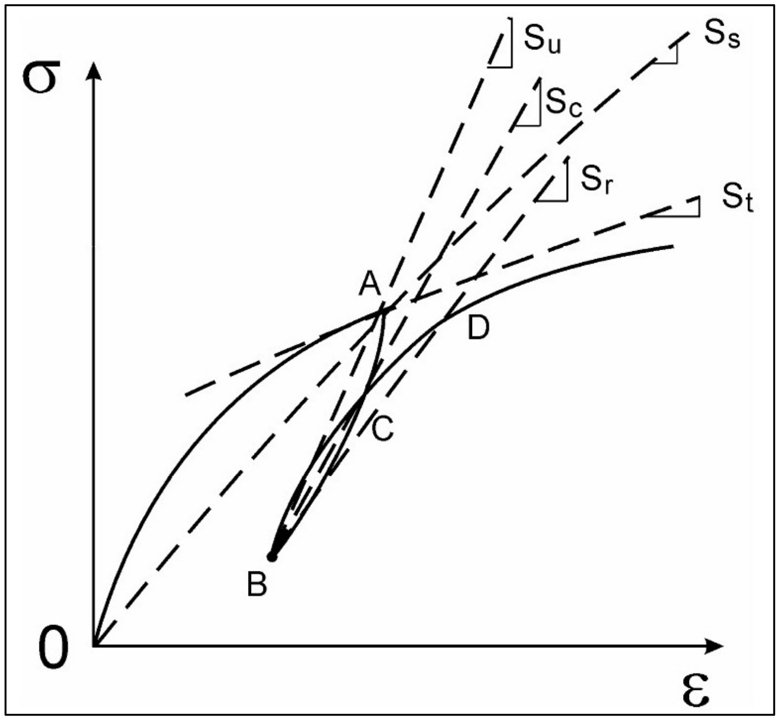

Figure 2-1: Definition of Soil Modulus (Briaud, 2001)

As Figure 2-1 implies, the selection of a single value of elastic modulus for a soil can be difficult. Not only is the relationship non-linear, it may not be readily apparent as to which definition of elastic modulus to use. Moreover, a stress vs. strain plot is often unavailable and therefore the engineer must understand the characteristics of the modulus they are using based on how it was estimated.

2.2 - Elastic Modulus from the Pressuremeter Test

The pre-bored pressuremeter is an in place test procedure consisting of positioning a cylindrical probe at depth into a pre-bored hole and then inflating the probe with either air or fluid while measuring the amount of fluid (assumed incompressible) introduced to the system and the resulting pressure in the probe (Sabatini, Bachus, Mayne, Schneider, & Zettler, 2002). These two measurements along with the probe geometry provide the information required to develop an in place stress-strain relationship for the soil at the location of the test. Figure 2.2 shows a diagram depicting the principles of the pressuremeter test.

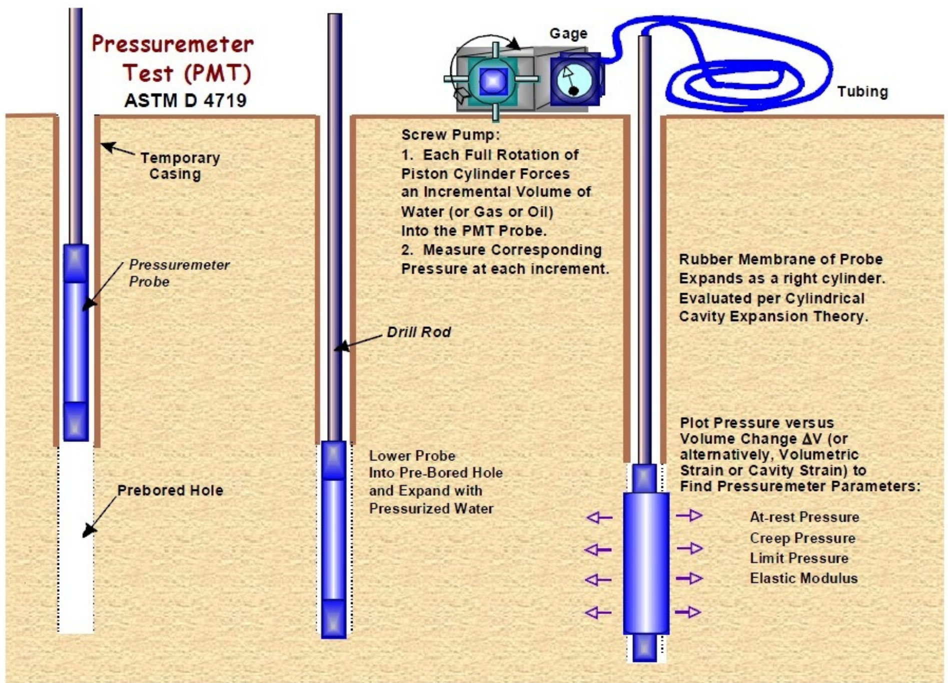

Figure 2-2: Description of Pressuremeter Test (Mayne, Barry, & DeJong, 2002)

The pressure measured in the hydraulic line as well as the radial expansion of the probe provides the information required to develop what is known as the pressuremeter curve. This curve consists of the radial stress vs. percent radial expansion. Figure 2-3 presents a typical pressuremeter curve with one unload-reload loop. The modulus values obtained from this curve would typically be the tangent modulus on the linear portion of the initial loading curve and the reload modulus taken as tangent on the reload portion of the curve.

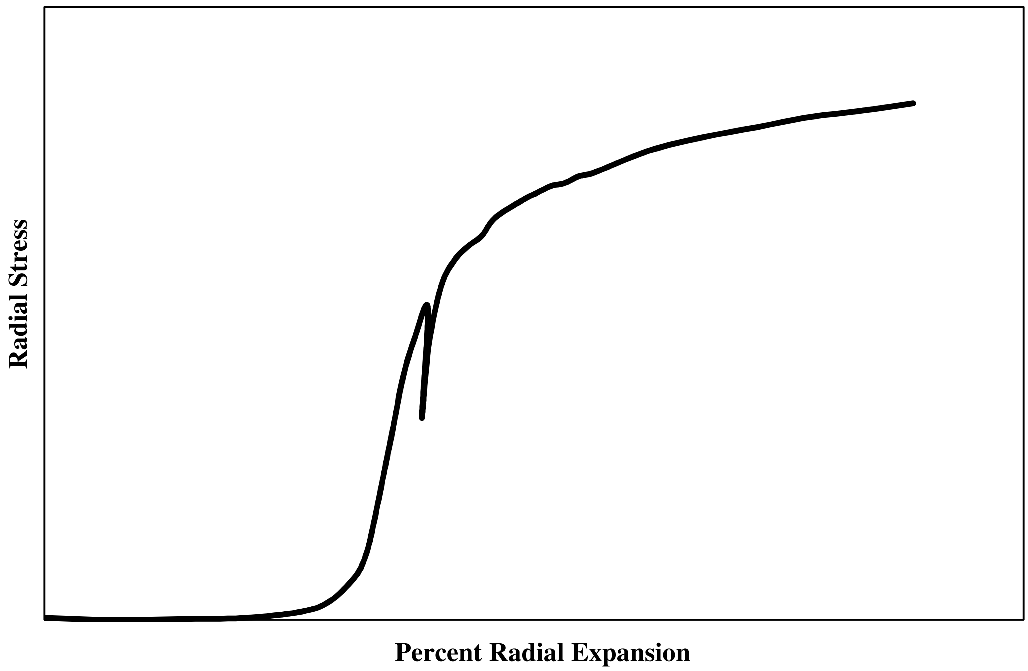

Figure 2-3: Typical Pressuremeter Curve with Unload-Reload Loop

The pressuremeter curve can be converted to show the radial stress vs. cavity strain at the borehole wall as shown in Figure 2-4 (Briaud, The Pressuremeter, 1992).

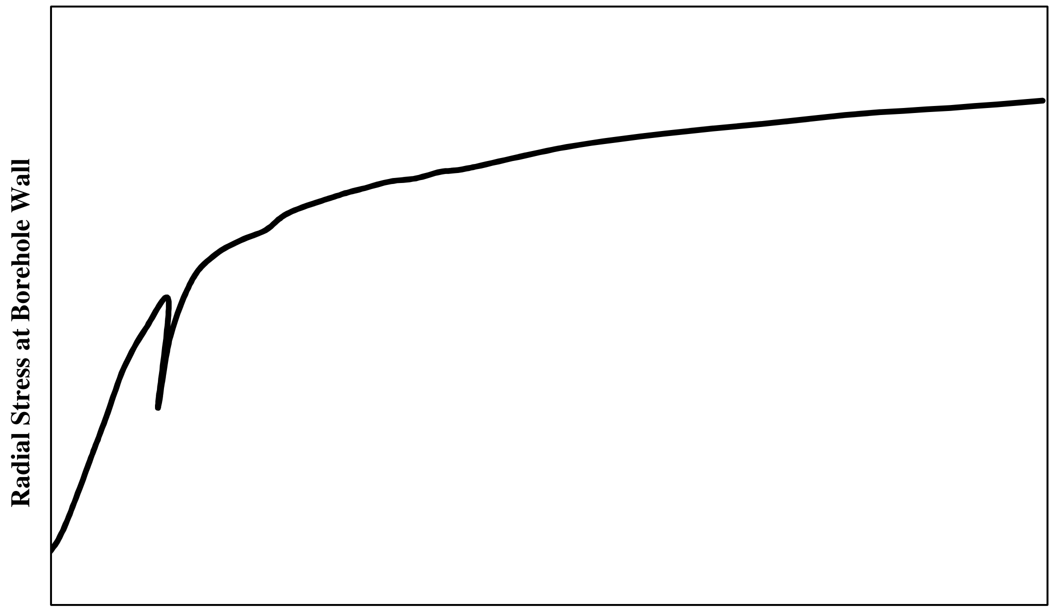

Figure 2-4: Typical Stress-Strain Curve from Pressuremeter Curve

As shown in the figures above, the elastic modulus is calculated from the linear segment of the initial loading portion of the pressuremeter curve using the following expression:

$$E = 2(1+\nu)(V_0+V_m)(\Delta P/\Delta V)$$

Where E is the pressuremeter elastic modulus, ν is the Poisson's ratio which is generally assumed to be 0.33 for pressuremeter tests (Briaud, The Pressuremeter, 1992), V_0 is the theoretical volume of the uninflated probe, V_m is the corrected volume increase from the initial volume, ΔP is the corrected pressure increase in the linear portion of the curve, and ΔV is the corrected volume increase in the linear portion of the curve. The initial loading shear modulus can also simply be taken as half of the initial slope of the pressuremeter curve. Similar theory can be used to calculate either the shear or elastic reload modulus from the pressuremeter curve as well.

The initial loading tangent modulus measured from the pressuremeter is known to be a relatively low modulus (Briaud, The Pressuremeter, 1992). Although the test produces a relatively low modulus, it is commonly accepted that this relatively low initial loading modulus is still less conservative than the traditional methods of correlating modulus using penetration test results and soil index properties.

Five reasons that the initial loading modulus measured by the pressuremeter test is generally considered to be relatively low were presented in Briaud, 1992. First, relatively large strains, on the order of 2% to 5%, are induced on the soil over the range at which the modulus is calculated. Second, due to the manner in which the soil is loaded, a portion of the soil is in tension, and the modulus measured is an average of the modulus of the soil in both tension and compression. It is known that soils are relatively weak in tension, and this will therefore reduce the measured modulus. Third, there is disturbance that is developed while preparing the borehole wall. Fourth, the equation to calculate the modulus is based on that of an infinitely long cylinder. A probe with a smaller length to diameter ratio will result in more conservative modulus. Fifth, the pressuremeter tests the horizontal modulus, which is not usually as high as the vertical modulus.

2.3 - Elastic Modulus from Shear Wave Velocity

When seismic energy is transmitted within a soil, the energy travels in seismic waves. Seismic waves can be either body waves or surface waves and there are multiple types of both (Sabatini, Bachus, Mayne, Schneider, & Zettler, 2002). The types of body waves are shear waves and compression waves and a diagram showing their propagation patterns is shown on Figure 2.6.

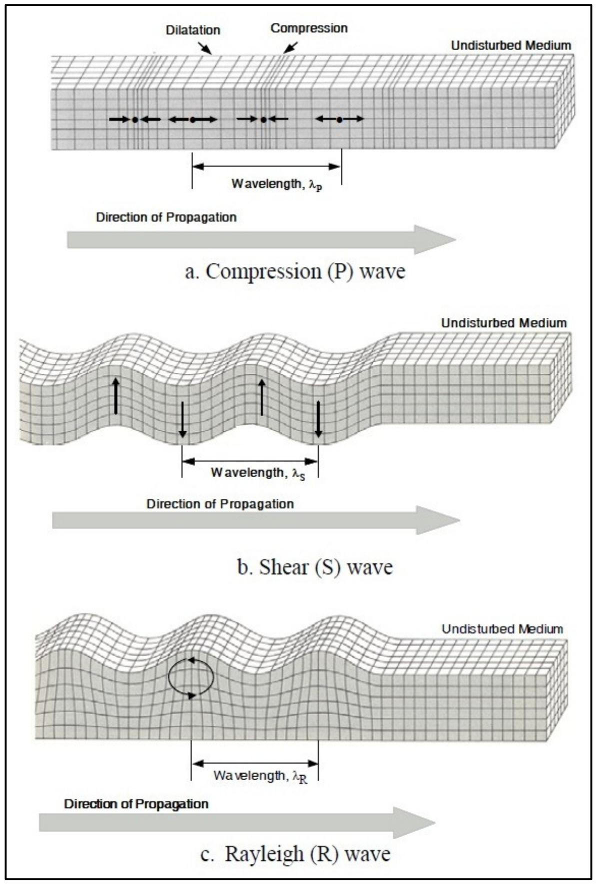

Figure 2-5: Modes of Wave Propagation (Stokoe, Joh, & Woods, 2004)

One important feature of body waves is that they travel at a constant speed within a medium and that speed is dependent on the stiffness of that medium. Compression wave velocities are dependent on the soil density and the constrained modulus of the soil. The velocity at which the shear waves travel is dependent on the density of the soil and the shear modulus, or soil skeleton stiffness of the soil.

There are many ways in which the shear wave velocity of a soil can be measured. The field tests that can determine shear wave velocities are known as geophysical methods. The geophysical methods can vary from surface methods such as refraction and reflection to subsurface methods such as downhole, crosshole, and seismic cone penetrtormeter. Methods for measuring shear wave velocities in soil that has recently been developed are the surface wave methods. The surface wave methods measure Rayleigh waves, a type of surface wave, and through an inversion process that utilizes the dispersive properties of surface waves, can develop a shear wave velocity profile of a soil (Louie, 2001). Common surface wave methods are the Refraction Microtremor (ReMi), Spectral Analysis of Surface Waves (SASW), and the Multi-Channel Analysis of Surface Waves (MASW) methods. A diagram showing the propagation patterns of Rayleigh waves is shown on Figure 2-6.

An important limitation of the surface wave methods is that they do not produce the detailed layering than can be obtained from a downhole test or seismic CPT test. Relatively thin layers that have different stiffness properties than the majority of the profile will likely not be detected by these methods. This results in a shear wave velocity that has been shown to be relatively accurate at averaging the shear wave velocity over large depths, but may not work well to represent the shear wave velocity at every point in the soil profile. The traditional and most common application for surface wave methods is the development of the average shear wave velocity in the upper 100 feet of the soil profile. Using the shear wave velocity profile measured from surface wave testing for the modeling of soil stiffness is not common, partly because of the above mentioned limitations. Despite the limitations, surface wave testing is the easiest method of obtaining shear wave velocities of a soil.

The shear modulus as obtained from the shear wave velocity of the soil is defined in the equation below:

$$G_{ss} = \rho_T * (V_s)^2$$

Where G_{ss} is the maximum (small strain) shear modulus, ρ_T is the total mass density, and V_s is the shear wave velocity. The shear modulus as calculated using a shear wave velocity is defined as the maximum shear modulus because testing has shown that the shear modulus reaches its maximum and is relatively linear at the strain levels produced by shear waves (Holtz, Kovacs, & Sheahan, 2011). The strain levels associated with shear waves are generally taken to be on the order of 10^{-6} (Iwasaki & Tatsuoka, 1977).

The most commonly accepted and utilized methods for performing settlement analysis require the use of an elastic modulus, not a shear modulus. The maximum shear modulus can readily be converted to elastic modulus using the relationship below:

$$E_{ss} = 2 * G_{ss}(1+\nu)$$

Where E_{max} is the maximum elastic modulus, G_{ss} is the maximum shear modulus, and ν is the Poisson's ratio. At very small strains the Poisson's ratio of soil has been shown to typically vary from 0.1 to 0.2 (Mayne, Barry, & DeJong, 2002).

2.4 - Elastic Modulus from Published Correlations

An intermediate strain level elastic modulus can be approximated by several methods. The two most commonly used methods are the correlations with SPT blowcounts and the correlations based on soil type. The correlations utilized in this study are presented in Tables 2-1 and 2-2.

Table 2-1: Estimating Elastic Modulus from SPT Blowcounts (AASHTO, 2011)

Soil Type	E_{spt} (ksi)
Silts, sandy silts, slightly cohesive mixtures	$0.056N1_{60}$
Clean fine to medium sands and slightly silty sands	$0.097N1_{60}$
Coarse sands and sands with litte garvel	$0.139N1_{60}$
Sandy gravel and gravels	$0.167N1_{60}$

Table 2-2: Estimating Elastic Modulus from SPT Blowcounts (McGregor & Duncan, 1998)

Soil Type	E_{spt} (kPa)
Sand	$500(N_{60}+15)$
Gravelly Sand and Gravel	$600(N_{60}+6)+2000$

Where $N1_{60}$ is the SPT blowcount corrected for hammer energy transfer efficiency and an overburden pressure of 1 ton per square foot, N_{60} is the SPT blowcount corrected only for hammer energy transfer efficiency, and E_{int} is the intermediate strain modulus. The correlation in Table 2-1 is presented in the AASHTO LRFD Bridge Design

Code (AASHTO, 2011) and the correlation in Table 2-2 is presented in the SPT manual commonly used in geotechnical practice (McGregor & Duncan, 1998). Note that no correlations are presented in the above tables for clay soils. Where the pressuremeter test was performed within clay soils, not correlation of SPT blowcounts was performed as it is generally assumed in geotechnical practice that SPT blowcounts are not sufficient to estimate the elastic modulus of clay soils.

CHAPTER 3 DATA COLELCTION AND DATA PROCESSING

3.1 - General Overview

The testing was performed in the northern portion of the Las Vegas Valley. Testing Site 1 was located in the unpaved median of an existing highway while Testing Site 2 was located in a rough graded area between a parking lot and an access road. The general surface geology at the two sites consists of recent alluvium deposits and older alluvium deposits. Nearby areas have also been mapped as fine grained spring and marsh deposits (Bell, Ramelli, & Caskey, 1998).

The testing sites were located approximately 1200 feet apart and are shown on Figure 3-2.

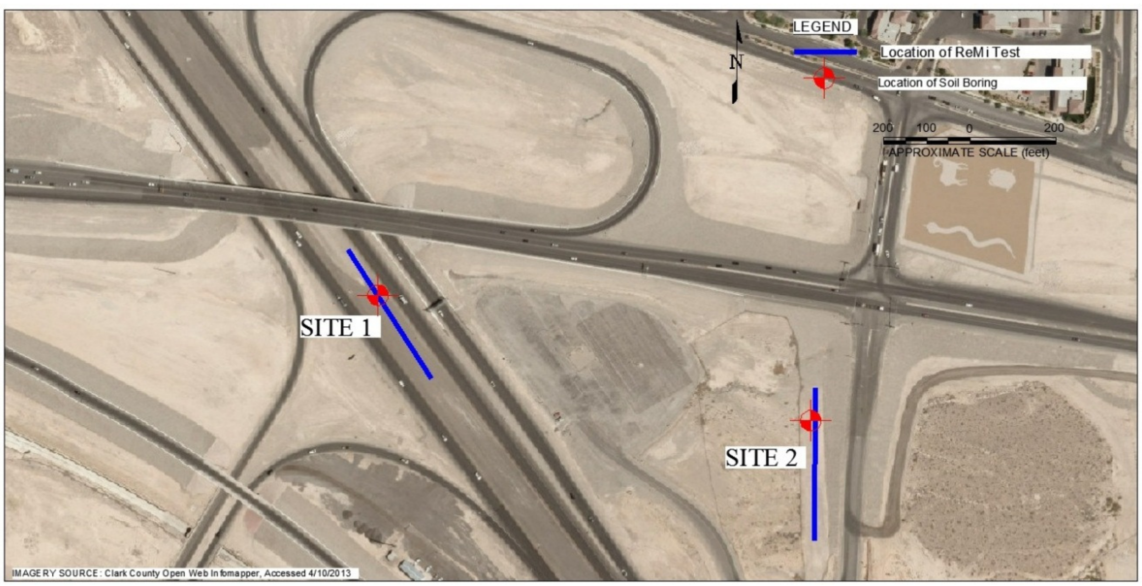

Figure 3-1: Testing Location Map

Testing was performed at both sites in three phases consisting of geotechnical soil borings, including soil sampling and Standard Penetration tests, pre-bored Pressuremeter testing, and geophysical testing to develop a shear wave velocity profile of each site. The testing was performed as part of a State highway transportation project for the development of preliminary recommendations for the design of bridge foundations. Consequently, the boring locations were selected by the State of Nevada.

The original scope of exploration was developed to evaluate the feasibility of using shallow foundations for heavily loaded structures where elastic settlement was assumed to control the foundation design due to the assumed depth to groundwater and the relatively coarse grained site geology.

3.2 - Soil Borings and SPT blowcounts

The soil borings were each drilled to a depth of approximately 120 feet with soil samples obtained in 2½ foot increments in the upper 20 feet, 5 foot increments from a depth of 20 feet to 80 feet, and 10 foot increments below 80 feet. Mud rotary methods were used for drilling with a Diedrich D120 drill rig equipped for soil sampling.

Standard Penetration test blowcounts were obtained at the majority of the sampling intervals using an unlined, 1-3/8 inch inside diameter in conformance with ASTM D1586. The sampler was driven with a 140 pound, hydraulically actuated, automatic trip hammer free-falling a distance of 30 inches. The number of blows required to drive the sampler 6 inches was recorded in the field until either the sampler was driven 18 inches or more than 50 blows was required to drive the sampler through one of the three 6 inch intervals. The hammer was calibrated to an energy transfer efficiency of 78 percent.

Laboratory testing was performed on the samples recovered from the SPT tests, as well as the undisturbed samples obtained from other sampling methods. The laboratory testing program was primarily directed toward USCS soil classification and consisted of grain size distribution analysis (ASTM C117 and C136), Atterberg Limits testing (ASTM D4318), in place Moisture content (ASTM D2216) and in place Dry Density testing (ASTM D2937).

USCS soil classification, corrected and uncorrected SPT blowcounts, moisture content and in place density data obtained during testing for both sites is presented in Tables 3-1 and 3-2.

Table 3-1: Results of Soil Borings at Test Site 1

Soil Layer	Top Depth (feet)	Bottom Depth (feet)	USCS Classification	Average Uncorrected N for Layer*	N_{60}	$N1_{60}$	Moisture Content (%)	Total Density (pcf)
1	0	4.5	GM	38	46	51	8	-
2	4.5	6.5	ML	34	41	33	9	120
3	6.5	8.5	SM	25	30	19	11	-
4	8.5	13.5	GC	72	86	48	8	147
5	13.5	15.5	SM	51	61	31	11	137
6	15.5	19	GM	36	43	20	12	-
7	19	21	SC	25	30	15	27	-
8	21	24	CL	-	-	-	24	-
9	24	28	CH	18	22	10	23	-
10	28	30	SC-SM	-	-	-	-	-
11	30	31	CALICHE	-	-	-	-	-
12	31	40	SC	40	48	19	15	-

*Refusal blowcounts not included in average.

Table 3-2: Results of Soil Borings at Test Site 2

Soil Layer	Top Depth (feet)	Bottom Depth (feet)	USCS Classification	Average Uncorrected N for Layer*	N_{60}	$N1_{60}$	Moisture Content (%)	Total Density (pcf)
1	0	2	GM	-	-	-	-	-
2	2	5	SC-SM	45	54	51	7	-
3	5	6.5	SC	78	94	69	12	124
4	6.5	8	CL	46	55	44	13	-
5	8	12	SM	34	41	25	16	138
6	12	14.5	SC-SM	17	20	11	20	125
7	14.5	15.5	GM	87	104	57	2	-
8	15.5	20	SM	73	88	44	7	141
9	20	30	SC	37	44	20	16	127
10	30	35	CL	31	37	16	18	-
11	35	40	CH	25	30	12	22	-

*Refusal blowcounts not included inaverage.

The field blowcounts were corrected for hammer energy transfer efficiency (N_{60}) and overburden stress ($N1_{60}$) by the procedures shown below (Das, 2006).

$$N_{60} = \frac{N\eta_H\eta_B\eta_S\eta_R}{60} \qquad\qquad N1_{60} = C_N N_{60}$$

Where N Is the number of blows required to drive the sampler 12 inches, η_H is the hammer efficiency, η_B is the correction for borehole diameter, η_S is the correction for sampler type, η_R is the correction for sampling rod length, and C_N is the overburden correction factor. The overburden correction factor used is defined below (Youd & Idriss, 2001).

$$C_N = \left(\frac{P_a}{\sigma'_{vo}}\right)^{0.5}$$

Where P_a is the atmospheric pressure and σ'_{vo} is the effective overburden stress at the sample depth.

Data obtained from the grain size distribution analyses and Atterberg Limits tests are presented in Tables 3-3 and 3-4. These values include the percent (by weight) of gravel, sand, and fines (percent passing the number 200 sieve), the liquid limit, and the plasticity index.

Table 3-3: Results of Laboratory Tests at Test Site 1

Soil Layer	Top Depth (feet)	Bottom Depth (feet)	USCS Classification	Percent Gravel	Percent Sand	Percent Fines	Liquid Limit (%)	Plasticity Index (%)
1	0	4.5	GM	50	34	16	-	-
2	4.5	6.5	ML	0	29	71	-	-
3	6.5	8.5	SM	28	46	27	-	-
4	8.5	13.5	GC	58	25	17	44	21
5	13.5	15.5	SM	31	43	26	49	18
6	15.5	19	GM	49	36	15	-	-
7	19	21	SC	0	54	46	110	78
8	21	24	CL	5	34	61	45	28
9	24	28	CH	0	46	54	61	39
10	28	30	SC-SM	29	34	36	24	7
11	30	31	CALICHE	-	-	-	-	-
12	31	40	SC	26	44	30	50	26

Table 3-4: Results of Laboratory Tests at Test Site 2

Soil Layer	Top Depth (feet)	Bottom Depth (feet)	USCS Classification	Percent Gravel	Percent Sand	Percent Fines	Liquid Limit (%)	Plasticity Index (%)
1	0	2	GM	-	-	-	-	-
2	2	5	SC-SM	30	39	31	24	6
3	5	6.5	SC	28	44	28	27	8
4	6.5	8	CL	9	33	57	24	9
5	8	12	SM	31	33	36	-	-
6	12	14.5	SC-SM	23	42	35	24	7
7	14.5	15.5	GM	57	31	13	-	-
8	15.5	20	SM	24	48	28	-	-
9	20	30	SC	11	49	40	35	17
10	30	35	CL	5	21	73	36	19
11	35	40	CH	1	38	62	53	32

3.3 - Pressuremeter Data Collection and Processing

Pressuremeters utilize either an air or hydraulic pressure system and can be either self-boring or require pre-boring. A TexAM hydraulic control unit and a single cell, long NX probe pre-bore type pressuremeter manufactured by Roctest was utilized for pressuremeter testing for this study. The pressuremeter specifications are given in Table 3-5.

Table 3-5: Pressuremeter Specifications

Probe Diameter (cm)	Membrane Length (cm)	Theoretical Volume (cm3)	Total Probe Length (cm)
7	46	1770	117

The pressuremeter testing was performed in general conformance to ASTM D4719. The testing was conducted as a volume controlled test (ASTM D4719, Procedure B) where pressure readings were obtained at designated volumes. The units utilized on the testing equipment were cubic centimeters for volume and bar for pressure. Volume and membrane calibrations were performed periodically throughout testing. Corrections were performed to the raw data for membrane resistance, volume losses and hydraulic head in the fluid supply line. As part of the testing procedure, once the change in pressure from one volume increment to the next decreased from that above it, the soil was deemed to have begun to fail and an unload-reload loop was performed in order to obtain and unload-reload modulus.

The pressuremeter testing was performed in a pre-bored hole drilled after and within a ten foot radius of the initial soil boring. The pre-bored hole was drilled with mud rotary methods using a 3½ inch drill bit. Once the desired depth for testing had been reached, a 2 15/16 inch drill bit was utilized to drill an additional 3 feet. The three inch outside diameter pressuremeter was then lowered into the hole and pushed into the final three feet such that the probe was positioned very tightly to reduce disturbance and softening of the borehole wall. Where possible, a head of mud was maintained to the top of the hole to better simulate overburden pressure.

A total of 6 successful pressuremeter tests were performed for the project, three at each site, ranging from 7.3 to 33.6 feet in depth below ground surface.

The shear and elastic modulus values obtained during pressuremeter testing along with the required assumptions to calculate them are presented in Table 3-6.

Table 3-6: Summary of Pressuremeter Test Results

Site Number	Depth (feet)	ν	E_{PMT} (ksf)	$E_{R,PMT}$ (ksf)	G_{PMT} (ksf)	$G_{R,PMT}$ (ksf)
1	7.3	0.33	1060	1455	398	547
	17.3	0.33	135	460	51	173
	28	0.33	530	805	199	303
2	7.9	0.33	765	2090	288	786
	11.7	0.33	730	2145	274	806
	33.6	0.33	505	1450	190	545

The shear modulus, as described in Chapter 2, was taken as half the slope of the linear portion of the stress-strain curve. Also shown in Table 3-6 are the shear and elastic modulus values obtained from the reload portion of the stress-strain curve. The stress-strain curves for each test are presented on Figures 4-1 through 4-6.

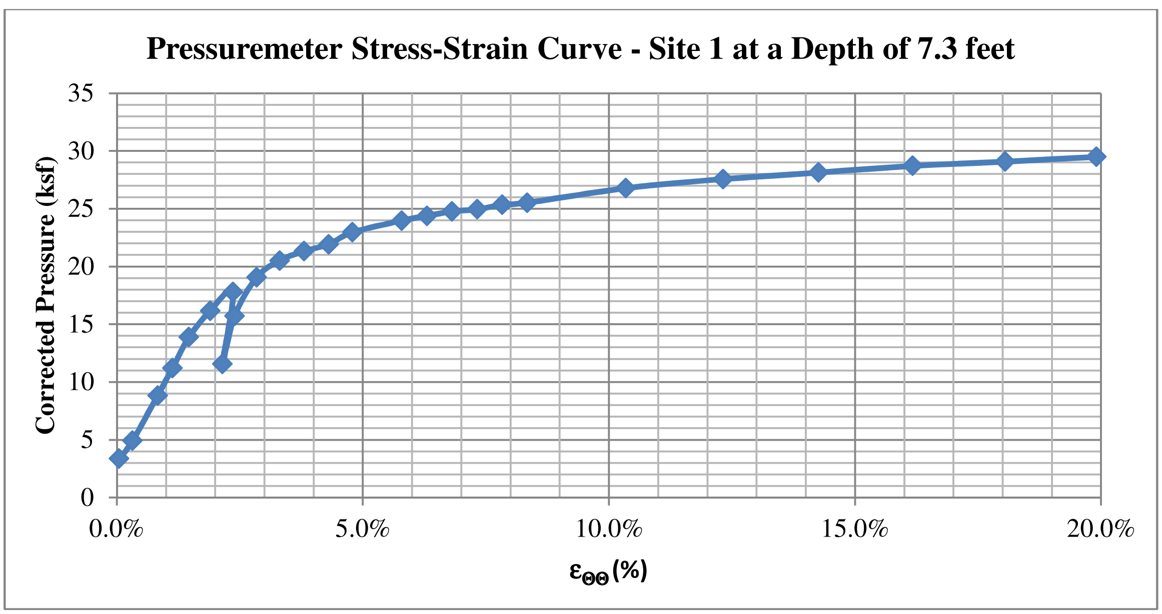

Figure 3-2: Pressuremeter Stress-Strain Curve for Site 1 at 7.3 feet

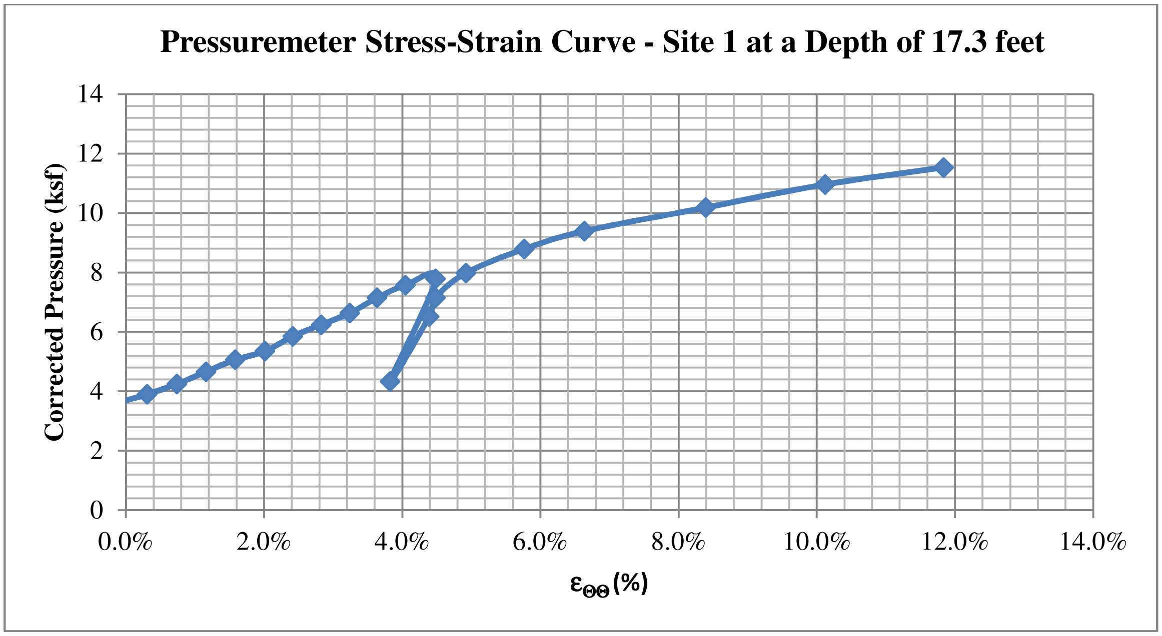

Figure 3-3: Pressuremeter Stress-Strain Curve for Site 1 at 17.3 feet

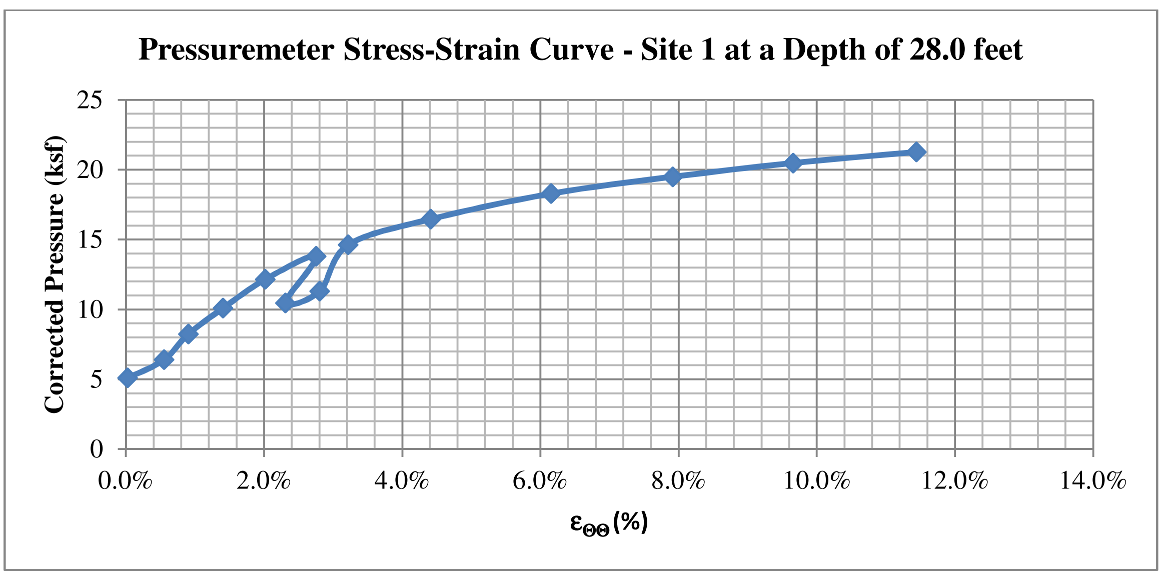

Figure 3-4: Pressuremeter Stress-Strain Curve for Site 1 at 28.0 feet

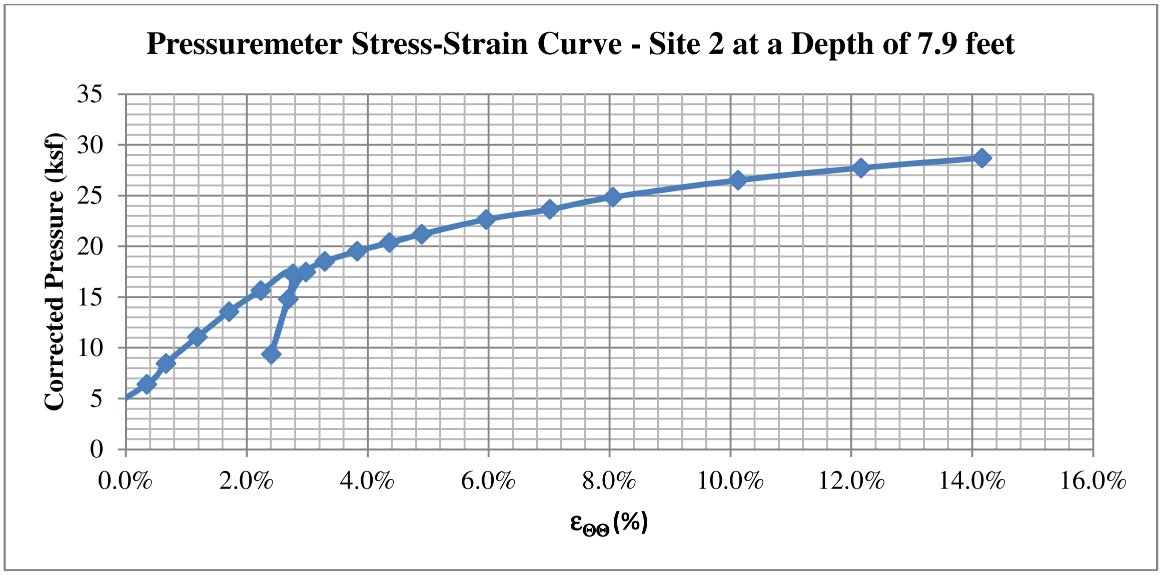

Figure 3-5: Pressuremeter Stress-Strain Curve for Site 2 at 7.9 feet

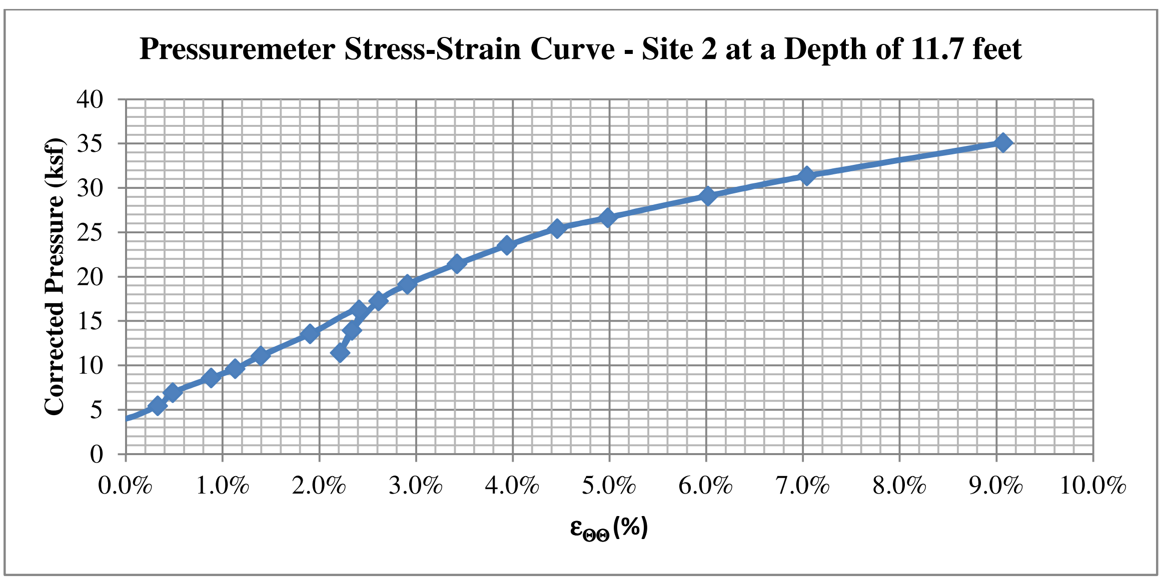

Figure 3-6: Pressuremeter Stress-Strain Curve for Site 2 at 11.7 feet

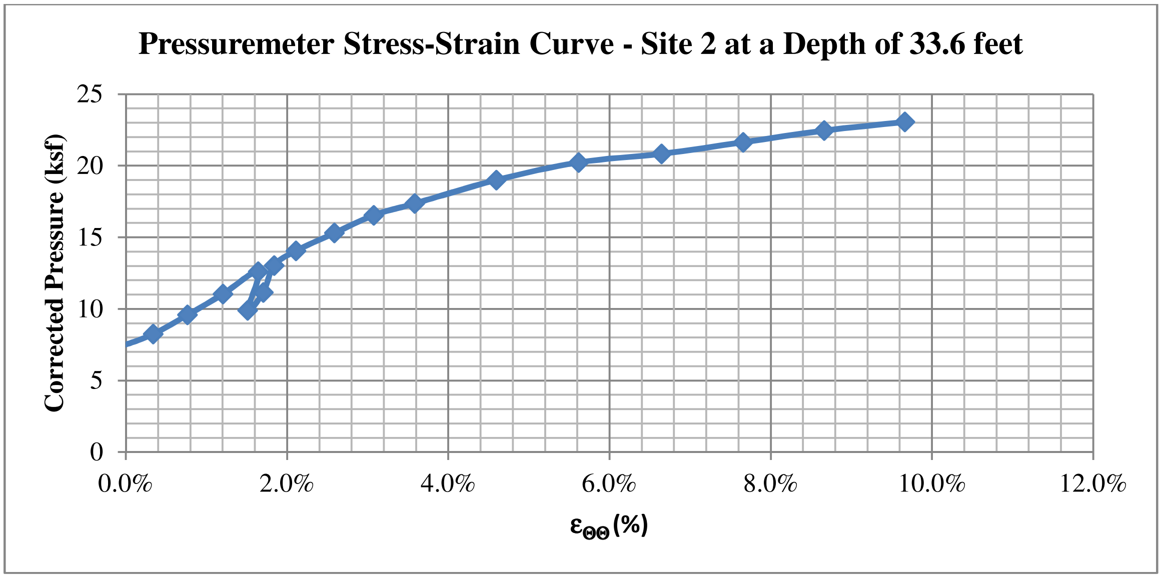

Figure 3-7: Pressuremeter Stress-Strain Curve for Site 2 at 33.6 feet

3.4 - Geophysical Testing for Shear Wave Velocity

The shear wave velocity profile was developed at each site by using a seismic surface wave method. The seismic surface wave geophysical testing utilized conventional geophysical equipment which included 24, 4.5 Hz geophones spaced at 15 feet apart and connected with cable to a DAQLink II seismograph which was connected to a laptop computer. Ambient seismic noise was used as the seismic source, the primary source of which was the highway traffic nearby. About 25 records, each thirty seconds long, were recorded at each site.

The Refraction Microtremor method was used to reduce the data to a shear wave velocity profile that extended over 100 feet into the subsurface. The field data was reduced and a shear wave velocity profile developed using Seis-Opt ReMi software.

The shear wave velocity profiles were obtained from the seismic surface wave testing for both Site 1 and Site 2 and are presented on Figures 3-8 and 3-9.

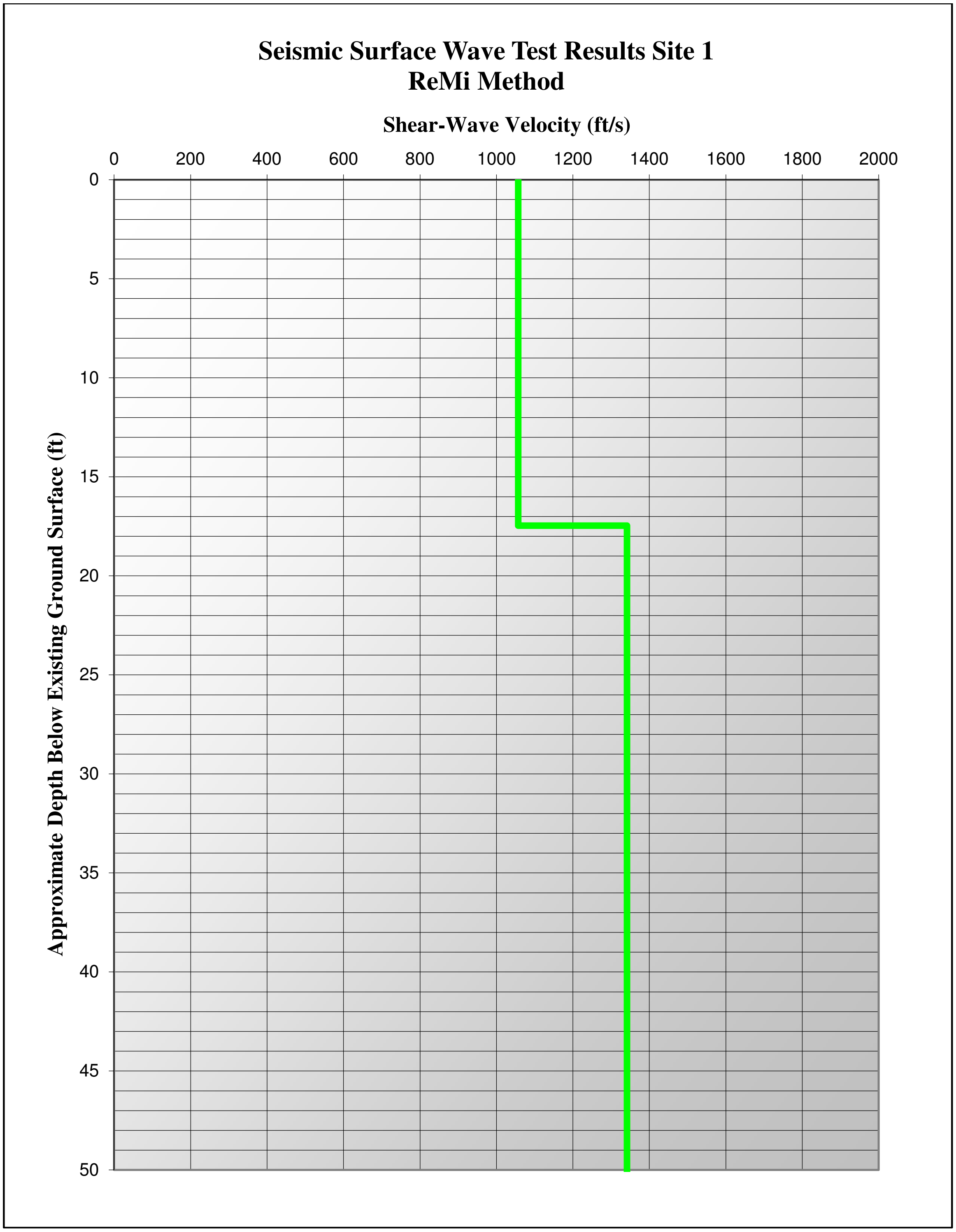

Figure 3-8: Seismic Surface Wave Test Results for Site 1

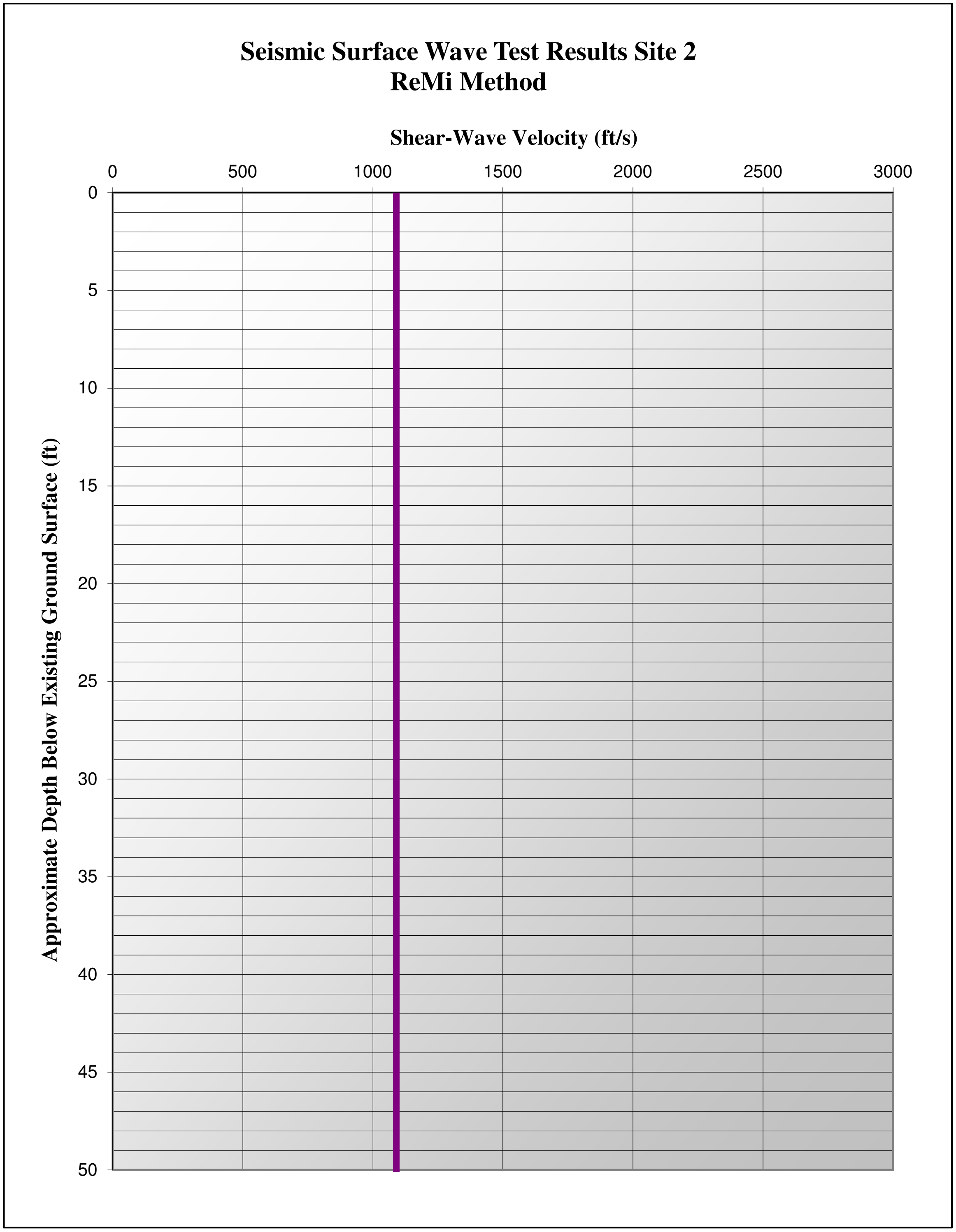

Figure 3-9: Seismic Surface Wave Test Results for Site 2

In order to calculate the small strain modulus, the average total unit weight of the soil at each shear wave velocity interval was calculated and a Poisson's ratio of 1.15 was assumed. The Poisson's ratio of 0.15 was assumed because published testing indicates that the appropriate Poisson's ratio at small strain ranges from 0.1 to 0.2 (Stokoe, Joh, & Woods, 2004). The shear wave velocities, total unit weight, small strain shear modulus, assumed Poisson's ratio, and small strain elastic modulus are presented in Table 3-7.

Table 3-7: Summary of Geophysical Test Results

Site Number	Top Depth (feet)	Bottom Depth (feet)	Shear Wave Velocity (ft/s)	Total Unit Weight (pcf)	G_{max} (ksf)	ν	E_{max} (ksf)
1	0	17	1057	135	4673	0.15	10747
	17	50	1342	135	7532	0.15	17324
2	0	50	1091	131	4842	0.15	15960

CHAPTER 4 ANALYSIS AND DISCUSSION OF TEST RESULTS

4.1 - Ratio of Small Strain Modulus to Initial Loading and Reload Pressuremeter Modulus

The goal of this study was to estimate an intermediate strain modulus when the small strain modulus is known. In order to accomplish this goal, a ratio of the small strain modulus to the intermediate strain modulus is calculated for the use of an engineer who has obtained the small strain modulus. Once a ratio has been determined, the engineer could divide the known small strain modulus by the ratio to obtain the desired intermediate strain modulus as shown in the following equations:

$$E_{int} = \frac{E_{ss}}{R_{initial}} \qquad\qquad E_{int} = \frac{E_{ss}}{R_{reload}}$$

Where E_{int} is the intermediate strain modulus, E_{ss} is the small strain modulus, and $R_{initial}$ is the ratio of small strain modulus to initial loading pressuremeter modulus and R_{reload} is the ratio of small strain modulus to reload pressuremeter modulus.

The calculation of the ratio of measured small strain modulus to intermediate strain modulus was performed for both the pressuremeter modulus and the unload-reload pressuremeter modulus. The results of these calculations are presented in Table 4.1.

Table 4-1: Summary of Small Strain to Intermediate Strain Modulus

Site Number	Depth (feet)	E_{PMT} (ksf)	$E_{R,PMT}$ (ksf)	E_{ss} (ksf)	$R_{initial}$	R_{reload}
1	7.3	1060	1455	10747	**10.1**	**7.4**
	17.3	135	460	17324	**128.3**	**37.7**
	28	530	805	17324	**32.7**	**21.5**
2	7.9	765	2090	15960	**20.9**	**7.6**
	11.7	730	2145	15960	**21.9**	**7.4**
	33.6	505	1450	15960	**31.6**	**11.0**

4.1.1 - Initial Loading Pressuremeter Modulus vs. Reload Pressuremeter Modulus

The first item to discuss is the obvious difference between the ratios for the initial loading pressuremeter modulus and the unload-reload pressuremeter modulus. This is to be expected for two reasons. First, the initial loading of the pressuremeter is performed after the pressuremeter is inserted into a pre-bored hole. The pre-bored hole has likely experienced significant relaxation as the previously existing confinement of the soil has been removed as the hole is excavated. Also attributed to the drilling of the hole is the probable disturbance of the soil in the area of the pressuremeter test which likely causes a reorientation of the soil grains upon initial loading. Second, the initial loading modulus is calculated at a greater strain level than that of the unload-reload modulus (Briaud, The Pressuremeter, 1992). The initial loading pressuremeter modulus is known to be a relatively low modulus, and although this low modulus is still higher than the modulus given by most correlations, many have found that the unload-reload modulus is more effective in accurately predicting settlements (Briaud, The Pressuremeter, 1992). The initial loading pressuremeter modulus may be desirable if very large strains are expected.

4.1.2 - Range of Initial and Reload Pressuremeter Modulus Ratios

The ratios calculated from the initial loading pressuremeter modulus ranged from 10.1 to 32.7, with one test resulting in a ratio of 128.3 and the ratios calculated from the reload pressuremeter modulus ranged from 7.4 to 37.7. The large range of ratios is most likely explained by the seismic surface wave method's inability to detect relatively thin layers. The velocities presented are best understood as an average over that depth interval. This will result in much variability if softer or stiffer soils than average for that layer interval are tested by the pressuremeter.

Although this range is larger than desirable for use in practice to reduce a small strain modulus, the information is still useful. The engineer could use an average value for elastic modulus provided that they are confident that they are not over-predicting the modulus in the near surface, where the greatest strains are expected.

4.1.3 - Comparison of Initial and Reload Pressuremeter Modulus Ratios with Respect to Depth

Another trend that is clearly shown within the data is the relative consistency of the ratios in the upper 17 feet. For the ratio to the unload-reload modulus, these values range from 7.4 to 7.6; a remarkably small range. Although a greater range than for the unload-reload modulus ratio, the ratio for the initial loading pressuremeter modulus is still relatively consistent, ranging from 10.1 to 21.9. This was not an expected outcome of the testing. Although this was not an expected outcome of the data, the relationship requires a discussion of some possible explanations.

A possible explanation of the consistency of the ratios in the upper 17 feet is the possible reduction in the ability of the surface wave method for calculating shear wave velocity with increasing depth. This reduction in ability due to the smaller depth of penetration of surface waves has been demonstrated in the literature (Rutledge, Mauldon, & Smith, 2005), although much of the literature indicates that although thinner layers are more difficult to represent with depth, the average shear wave velocity is represented relatively well (Louie, 2001).

A more likely explanation has to do with the differences in modulus degradation for different soil types. The ReMi test performed at Site 1 indicates a probable layer boundary at a depth of 17 feet. This is important, because it has been shown that the rate at which a modulus degrades is different for different soil types. If the soils at this site in the upper 17 feet are relatively consistent, and the soils below 17 feet are quite different than those above them, then it would make sense for the ratio of small strain modulus to intermediate strain modulus to be different. The soil boring performed at Site 1 does not indicate a significant difference in the soil types above and below 17 feet, although the soils below 17 feet may be considered slightly finer grained, consisting of more clay. Despite the slight increase in fines, most of the soils still classify as sands both above and below 17 feet.

Finally, the data presented in Tables 3-1 and 3-2 show an increase in moisture content below a depth of approximately 17 feet. This, in conjunction with the increase in the amount of fine grained soils, may explain why the ratios are different below 17 feet. Although it is known that the shear wave velocity is unaffected by the presence of water, it is possible that the moisture content has an effect on the soil stiffness measured by the pressuremeter.

It has been established above that the ratios for this data in the upper 17 feet are relatively consistent. The ratios below 17 feet are very inconsistent with each other, but are always greater than those in the upper 17 feet ranging from 31.6 to 128.3 for the initial loading pressuremeter modulus ratio and 11.0 to 37.7 for the unload-reload pressuremeter modulus ratio.

The inconsistency of these results could be explained by the diminished reliability of seismic surface wave methods in determining the shear wave velocity with depth. Even if the average shear wave velocity over a large layer is relatively accurately determined by seismic surface wave methods, it is likely that the resolution of the data is poor; that it cannot recognize relatively thin layers. If the relatively thin layers are not captured by the method, then thin layers of either harder or softer zones will yield much different values.

The values of the ratios below 17 feet were also shown to be much greater than the values above 17 feet. The most likely explanation for this phenomenon that the layers tested was simply within the zones that were softer than the average modulus as determined by the shear wave velocity. This does raise the question of the possibility that the zones tested in the upper 17 feet were simply stiffer than the zones tested in the lower 17 feet. This conclusion will be investigated deeper, when the results of the Standard Penetration test results correlations are compared with the results of the pressuremeter testing.

A final discussion on the apparent trend at 17 feet should note that when performing a deformation analysis, the accuracy at which the near surface soils are characterized is of much greater importance than the deep soil because the near surface soils bear the largest stress increase under loading and will therefore settle the most. The consistency, therefore, in the upper 17 feet may make the engineer more comfortable in making use of the above presented modulus reduction ratios. Although it is clear that the trend at 17 is not applicable at all sites at the depth of 17 feet, further investigation should be performed to determine if the pressuremeter results tend to be more consistent in the near surface.

4.1.4 - Discussion of Pressuremeter Test at Test Site 1 at a Depth of 17.3 feet

The ratios for the pressuremeter test performed at site 1 at a depth of 17.3 feet are noticeably larger than the ratios for the other 5 tests. This anomalous value may be explained by a result from the geophysical testing. The ReMi test performed at the site shows a break in layers at 17 feet. When the pressuremeter testing was performed, the depth to the center of the probe was measured to be 17.3 feet. If a layer boundary is present within the approximate 18 inches thick zone of testing by the pressuremeter, there may be an adverse effect on the test results (Briaud, The Pressuremeter, 1992). This result may be more pronounced if the layer boundary is very near the center of the inflating probe.

4.2 - Comparison of Initial Loading and Reload Pressuremeter Modulus Ratios to Ratios Calculated Using SPT Blowcounts

As a means of better understanding the ratios developed in section 4.1, a comparison of the ratio of small strain modulus to an intermediate strain modulus where the intermediate strain modulus is estimated by correlations with SPT blowcount data is warranted. These correlations were performed and the results are presented in section 3.4. Similar to the equations presented in Section 4.1, a ratio can be used to estimate the intermediate strain modulus as shown below:

$$E_{int} = \frac{E_{ss}}{R_{SPT}}$$

Where E_{int} is the intermediate strain modulus, E_{ss} is the small strain modulus, and R_{SPT} is the ratio of small strain modulus to the correlated intermediate strain modulus from SPT blowcount data.

The ratios of the small strain modulus to the intermediate strain modulus as calculated using the two published blowcount correlations were calculated and are presented in Table 4-2. The correlations were not performed for the tests at Site 2 at depths of 7.9 and 33.6 feet because soil classification testing indicated that these are clay soils. Standard Penetration test blowcount correlations are not appropriate for clay soils.

Table 4-2: Summary of Small Strain to Intermediate Strain Modulus by Blowcount Correlations

Site Number	Depth (feet)	^{+}N	$N_{1,60}$	$^{*}E_{spt}$ (ksf)	N_{60}	$^{**}E_{spt}$ (ksf)	E_{ss} (ksf)	$^{*}R_{spt}$	$^{**}R_{spt}$
1	7.3	25	19	379	30	470	10747	28.3	22.9
	17.3	36	20	483	43	658	17324	35.8	26.3
	28.0	49	24	481	59	771	17324	36.0	22.5
2	7.9	46	34	-	55	-	15960	-	-
	11.7	19	12	246	23	395	15960	64.8	40.4
	33.6	31	15	-	37	-	15960	-	-

+ N values were obtained from the SPT test nearest the depth of the pressuremeter test. See Table 4-1 for more information on SPT blowcounts.
*Correlations from (AASHTO, 2011)
**Correlations from (McGregor & Duncan, 1998)

An initial discussion is warranted in the discrepancy in the results between the two methods. These values compare relatively well with each other, but it is obvious that the second correlation method proposed by Duncan results in the prediction of a stiffer soil (lower R_{spt} ratio). This may be cause by a number of reasons, but the more important item to note is that they are relatively consistent with one another, and that they are significantly greater than those measured by the pressuremeter test.

A comparison of the ratios calculated for the pressuremeter tests and SPT blowcounts are presented in Table 4-3, Comparison of Small Strain to Intermediate Strain Modulus Values obtained from Pressuremeter and Standard Penetration Tests.

Table 4-3: Comparison of Small Strain to Intermediate Strain Modulus Values obtained from Pressuremeter and Standard Penetration Tests

Site Number	Depth (feet)	E_{ss}/E_{PMT}	$E_{ss}/E_{R,PMT}$	$*R_{spt}$	$**R_{spt}$
1	7.3	10.1	7.4	28.3	22.9
	17.3	128.3	37.7	35.8	26.3
	28.0	32.7	21.5	36.0	22.5
2	7.9	20.9	7.6	-	-
	11.7	21.9	7.4	64.8	40.4
	33.6	31.6	11.0	-	-

*Correlations from (AASHTO, 2011)
**Correlations from (McGregor & Duncan, 1998)

As described in the introduction, the modulus values calculated by SPT blowcounts are typically smaller than those calculated in the laboratory or with in-situ methods such as the pressuremeter or dilatometer. The calculated values are smaller because of the conservatism employed due to the relatively large amounts of uncertainty in the relationship (Sabatini, Bachus, Mayne, Schneider, & Zettler, 2002). As expected, the modulus values calculated from SPT blowcounts are generally smaller (larger E_{ss}/E_{spt} ratio) than those calculated by both the initial loading pressuremeter and reload pressuremeter tests. Exceptions are the initial loading pressuremeter modulus for the test at Site 1 at a depth of 17.3 feet and one of the values of the blowcount correlated ratio for Site 1 at a depth of 28.0 feet. As was discussed in Section 5.1, the pressuremeter test at Site 1 at a depth of 17.3 feet is considered anomalous and the very low value may have been caused by the possible layer boundary around 17 feet.

As in Section 4.1, the data above 17 feet and below 17 feet should be discussed. Due to the limitations of the blowcount correlations, only two values exist below 17 feet that we can discuss. The first of which (Site 1 at a depth of 17.3 feet) has already been discussed to be potentially anomalous due to the probable layer boundary located near the centerline of the pressuremeter probe, and therefore, it is difficult to draw a conclusion on this test. It is noteworthy, however, that the intermediate strain modulus calculated by the blowount correlations appear to be relatively consistent with that calculated with the pressuremeter test for the test at Site 1 at a depth of 28 feet. This may lead us to believe that the correlations possibly provide a better estimation of stiffness as the tests become deeper below the surface. More research would have to be performed to further this discussion topic.

As discussed in Section 4.1, the ratios developed from pressuremeter data increase significantly below a depth of 17 feet. The question should be raised as to whether or not this is simply due to stiffer soils existing above 17 feet than below 17 feet. One point that seems to indicate that this result was not simply due to stiffer soils being present above 17 feet is that the ratios developed by the Standard Penetration test blowcounts do not show the same trend. In fact, the largest ratio is present at a depth of 11.7 feet. It is therefore unlikely, although still possible because of the limited amount of data, that the soils tested above 17 feet were simply stiffer than the soils tested below 17 feet.

CHAPTER 5 CONCLUSIONS

The primary objective of this study was to compare the small strain modulus as calculated from the shear wave velocity determined from seismic surface wave methods to the intermediate strain modulus directly measured by the pressuremeter test. The comparison was made by calculating a ratio of small strain modulus to pressuremeter modulus. An additional comparison was made by calculating the ratio of small strain modulus to an intermediate strain modulus calculated by using SPT blowcounts. These comparisons led to multiple conclusions.

It was shown that the ratio of small stain modulus to pressuremeter modulus was relatively consistent in the upper 17 feet and both larger and more variable below 17 feet. This could be due to a number of factors, each of which could be examined in more detail with additional testing. First, the ReMi test performed at Site 1 indicated a probable layer change at a depth of 17 feet. It is probable that the soil type changes below a depth of 17 feet. It is known that the degradation of soil modulus is dependent on soil type, and therefore, if the soil type has changed significantly at 17 feet, this should change the ratio of small strain modulus to intermediate strain modulus. Second, it is possible that the shear wave velocities measured by ReMi seismic surface wave method are less accurate with increasing depth. This has been shown in the literature. Finally, due to the small amount of data available, it is possible that this data trend is merely coincidental.

It was also shown that, when looking at all of the small strain modulus to pressuremeter modulus ratios irrespective of their test depth, there was a relatively large range of ratios (10.1 to 128.3 for the initial loading pressuremeter modulus and 7.4 to 37.7 for the unload-reload pressuremeter modulus). Although these ranges are not as small as desirable, this information can be used by the engineer to approximate the upper and lower bound of stiffness degradation to an intermediate strain level. The average ratio may also be used by the engineer if they are confident that they are not underestimating stiffness in the near surface soils, which are subjected to the greatest strains.

The ratios of small strain modulus to the unload-reload pressuremeter modulus were generally smaller (stiffer intermediate strain modulus) than those calculated using the published SPT blowcount correlations. The unload-reload pressuremeter modulus is a direct measurement of soil stiffness at intermediate strains and is therefore believed to be less conservative than the modulus values calculated using Standard Penetration test blowcounts. The engineer may, at a minimum, prefer to use the upper bound of the ratio of small strain modulus to unload-reload pressuremeter modulus as opposed to traditional blowcount correlation method.

CHAPTER 6 RECOMMENDATIONS

As discussed above, the findings of this study are mostly inconclusive due primarily to the small size of the data set. Although the findings are inconclusive, trends were identified that should be researched further. These trends include that of the range of ratios of small strain modulus to intermediate strain modulus determined from pressuremeter tests being very large and the relatively consistent ratios in the near surface soils.

It is thought that the large range of ratios is caused by the inability of the surface wave methods to accurately represent relatively thin layers; layers that were directly measured by the pressuremeter. A feasible method for testing this hypothesis would be to perform a downhole type seismic test to directly measure shear wave velocity at the depth that the pressuremeter test was performed. Increasing the size of the data set by performing more pressuremeter tests would also be valuable as it would better identify trends in the data and reveal outliers.

The relative consistency and higher stiffness of the pressuremeter data in the upper 17 feet is thought to have been caused by the decreasing ability of the seismic surface wave methods to accurately represent shear wave velocity with increasing depth. This may have also been caused by a change in soil type, which was shown in the results of the seismic surface wave testing and laboratory testing. It was also shown that the SPT blowcounts do not indicate that the data in the upper 17 feet is simply a more consistent and stiffer material with respect to the difficulty of penetration. As the data seems to indicate that this trend was caused by soil type and moisture content it would make sense to further investigate the effect of soil type and moisture content on modulus degradation.

As the data set for this study is relatively small, additional soil borings with additional moisture content tests and soil classification tests should be performed along with the additional pressuremeter tests.

With the amount of existing data at the project site, it would be beneficial for researchers if settlement monitoring was performed during and after construction so that effective stiffness parameters could be back calculated. This would provide a true value that the values derived from the different methods could be compared to.

Finally, a laboratory testing program could be performed on relatively undisturbed samples. Testing should include Resonant Column Torsional Shear testing to estimate the initial shear modulus which could then be compared to the value determined in the field, and triaxial testing in axial compression could be used to estimate the intermediate strain modulus.

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