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# FIELD AND LABORATORY MEASUREMENTS OF DYNAMIC SHEAR MODULUS OF PIEDMONT RESIDUAL SOILS

James A. Schneider[1], Laureano Hoyos Jr.[2], Paul W. Mayne[3], Emir J. Macari[4], and Glenn J. Rix[3]

## ABSTRACT

Laboratory and field measurements of the small-strain shear modulus of a Piedmont residual soil were compared to gain a better understanding of mechanisms affecting stiffness. Resonant column tests were performed to study influences of void ratio, overconsolidation ratio, plasticity, confining stress, and degree of weathering. In-situ test results showed effects of increased confining stress with depth, and OCR, mass density, and void ratio were estimated using correlations to assess current in-situ state. In-situ measurements of low-amplitude shear moduli from seismic piezocones, seismic flat dilatometers, SASW, and crosshole tests were found to be in good agreement with laboratory values using the resonant column.

## INTRODUCTION

The use of in-situ tests to compliment conventional drilling, sampling, and laboratory testing has become an expedient and cost-effective way to determine strength and stiffness parameters over an entire site. Although correlations can be used to estimate the in-situ shear modulus of the soil (e.g., Mayne & Rix, 1993), a direct measurement of the soil property is desirable. Field measurements of shear wave velocity include crosshole tests (CHT), downhole tests (DHT), suspension logging, seismic reflection, seismic refraction, and spectral analysis of surface waves (SASW). A representation of these test procedures is shown in Figure 1. The seismic piezocone penetration test (SCPTu; Campanella et al., 1986) and seismic flat dilatometer test (SDMT; Martin & Mayne, 1997) are two hybrid in-situ tests that provide a downhole measurement

[1] Graduate Student, Georgia Institute of Technology, Atlanta, GA 30332-0355
[2] Post Doctoral Fellow, Georgia Institute of Technology, Atlanta, GA 30332-0355
[3] Associate Professor, Georgia Institute of Technology, Atlanta, GA 30332-0355
[4] Chair, School of Civil & Env. Engineering, Louisiana State University, Baton Rouge, LA 70803

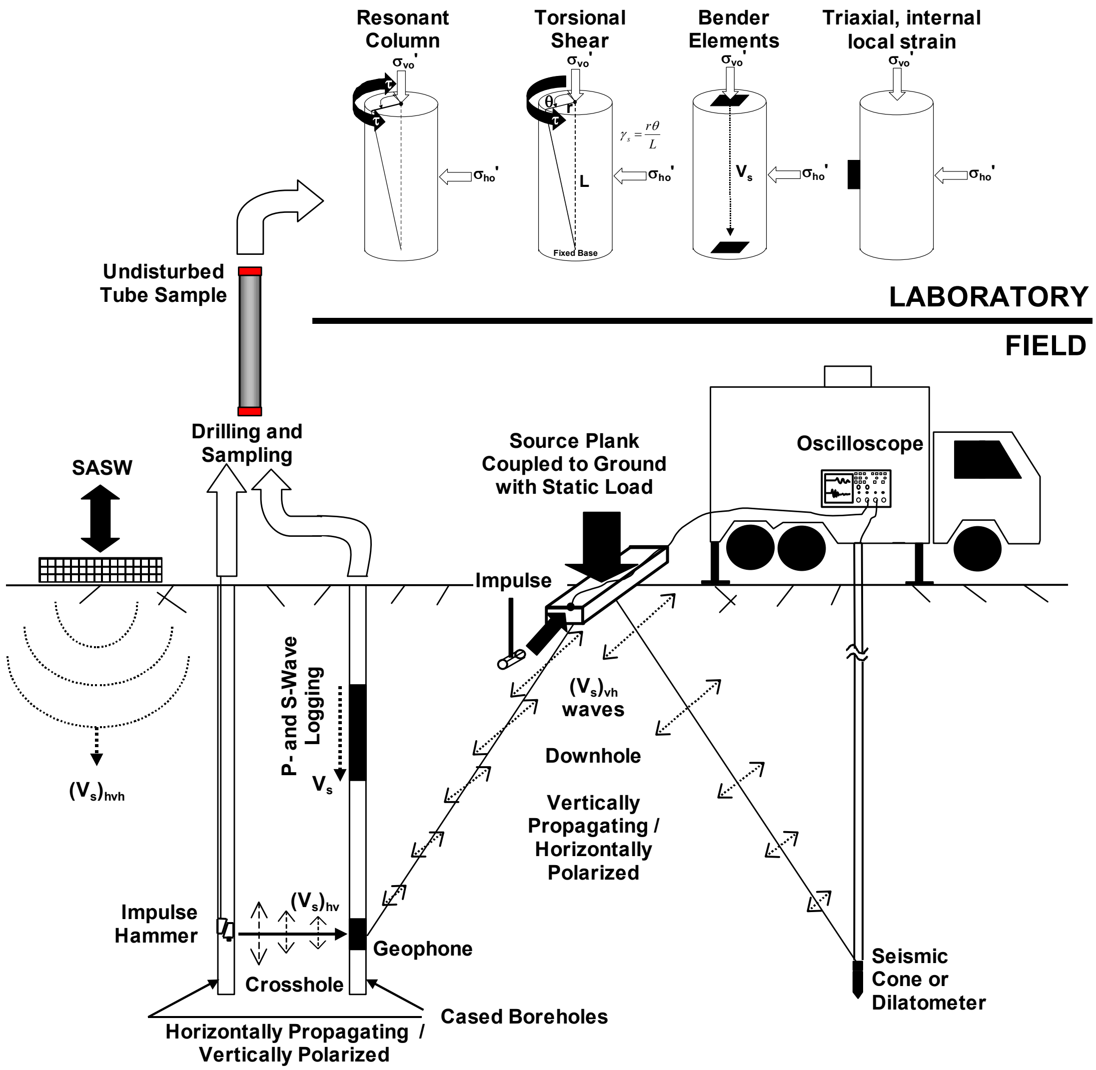


**Figure 1. Field and Laboratory Methods for Determining Shear Modulus**

of shear wave velocity in addition to penetration test parameters, thus optimizing data collection.

Laboratory measurements have long been the reference standard for determining the properties of geomaterials. To develop a greater confidence of the results of in-situ tests, it is helpful to compare field results to conventional laboratory tests. In the laboratory, parameters such as shear strain, confining pressure, frequency, number of loading cycles, void ratio, and overconsolidation ratio (OCR) can be varied to analyze soil response. In addition to these parameters, tests on undisturbed specimens provide insight into the effect of the degree of weathering for residual soils (Macari & Hoyos, 1996; Hoyos & Macari, 1999). Analysis of in-situ test data can estimate soil parameters (such as void ratio and OCR) that will affect the maximum small-strain shear modulus, $G_{max}$, but the confidence in these correlated values for residual silty sands is not high without laboratory confirmation. A comparison of laboratory and in-situ measurements in Piedmont residual soils was undertaken to better define mechanisms affecting $G_{max}$.

## SHEAR STIFFNESS

The shear wave velocity, $V_s$, is a soil property used to determine the shear modulus, G, of the soil:

$$G = \rho \cdot V_s^{\ 2} \tag{1}$$

where $\rho = \gamma_t/g_a$ = mass density, $\gamma_t$ = total unit weight, and $g_a$ = gravitational acceleration = 9.8 m/s$^2$. For plane waves, the shear strain, $\gamma_s$, is defined as the ratio of peak particle velocity, $\dot{u}$, to shear wave velocity:

$$\gamma_s = \frac{\dot{u}}{V_S} \tag{2}$$

At small strains, particle motion resulting from propagation of shear waves is nondestructive. As $\gamma_s$ increases past the elastic threshold shear strain (Dobry et al., 1982), $\gamma^e_{th}$, the shear modulus will decrease from the maximum small strain value, $G_{max}$. In-situ tests have commonly been assumed to be small strain ($\gamma_s < \gamma^e_{th}$), and the measurement of shear wave velocity will be directly related to the maximum shear modulus. However, at shallow depths it has been noticed that shear strains above the threshold strain may be reached during DHT, and thus a strain-based correction factor may be necessary to obtain $G_{max}$ (Larsson & Mulabdic, 1991).

Dynamic loading from wind, waves, equipment vibrations, or earthquakes will lead to an accumulation of shear strains, and the shear modulus will be reduced once the threshold shear strain has been exceeded. Empirical modulus reduction schemes have been developed for unaged clean quartz sands and insensitive clays for dynamic loading (e.g., Vucetic & Dobry, 1991), however, their validity in residual silty soils has not been verified. The appropriate $\gamma^e_{th}$ must be known, as well as the appropriate modulus reduction with increasing shear strain. The small-strain shear modulus is also relevant to static monotonic loading of foundations and walls. For these problems, the characteristic shear strains of most soil elements are generally in the range of $10^{-1}$ to $10^{-2}$ percent (Burland, 1989). Thus the modulus values used for settlement calculations and deformation analyses should correspond to the small- to intermediate-strain regime, requiring reduction schemes to be applied after the threshold strain has been reached. Moreover, it should be noted that the shear modulus for monotonic cases has been shown to reduce at a much faster rate than for dynamic cases (e.g., LoPresti et al., 1993), so reduction schemes determined from dynamic testing should not be used for monotonic problems.

## SPRING VILLA TEST SITE, OPELIKA, AL

In-situ testing and sampling for laboratory specimens have been conducted at the Spring Villa national geotechnical experimentation site (NGES), located in the southwestern part of the Piedmont geologic province near Opelika, Alabama. Numerous soil borings at the site indicate over 30 meters of silty-sands to sandy-silts (SM-ML) grading into partially weathered gneiss and decomposed schist (Vinson & Brown, 1997). A range of index properties determined at this site are provided in Table 1, and index properties of specimens used in the laboratory phase of this study are contained in Table 2. The groundwater table existed at about 2.4 meters depth during initial drilling and sampling.

**Table 1. Variation in Typical Index Properties at Spring Villa NGES (adapted from Brown & Vinson, 1998)**

| Property | Number of Tests | Average Value | Standard Deviation |
|---|---|---|---|
| Water Content, $w_n$ (%) | 64 | 34 | 7.5 % |
| Sand (%) | 48 | 47 | 17 % |
| Silt (%) | 22 | 33 | 8 % |
| Clay (%) | 22 | 10 | 6 % |
| Liquid Limit, LL[1] | 22 | 46 | 10 |
| Plasticity Index, PI[1] | 22 | 8 | 6 |
| Unit Weight, $\gamma_t$ (kN/m$^3$) | 35 | 18.2 | 0.5 |

[1] LL and PI data do not include 20 tests, which were reported as "nonplastic"

**Table 2. Index Properties of Laboratory Specimens from Spring Villa NGES**

| Depth (m) | Soil Type | $w_n$ (%) | LL | PI | Sand (%) | Silt (%) | Clay (%) | $e_o$ | AOCR[2] |
|---|---|---|---|---|---|---|---|---|---|
| 3 | ML | 30.2 | 37 | 10 | 20.3 | 78.1 | 1.6 | 0.953 | 4.0 |
| 4 | ML | 27.2 | 39 | 12 | 25.3 | 69.5 | 5.2 | 1.036 | 3.6 |
| 5 | SM | 32.8 | 28 | 4 | 60.2 | 35.1 | 4.7 | 1.221 | 3.4 |
| 7 | SM | 25.4 | - | NP[1] | 58.2 | 37.0 | 4.8 | 0.953 | 3.2 |
| 8 | ML | 23.2 | 34 | 5 | 31.8 | 63.0 | 5.2 | 1.111 | 3.0 |
| 9 | SM | 45.3 | - | NP[1] | 68.4 | 26.8 | 4.8 | 1.716 | 1.1 |

[1] NP = "nonplastic"
[2] AOCR = "apparent overconsolidation ratio"

## LABORATORY TESTING PROGRAM

In the current study, a total of 12 resonant column specimens (142 mm height, 72 mm diameter solid cylinders) were recovered and trimmed from 10 Shelby tubes retrieved at the Spring Villa NGES. The specimens correspond to residual soil formations (SM and ML) at depths ranging from 3 m to 10 m. Weathering of the soils is most evident at the ground surface and decreases with increasing depth. The apparent OCR profile (determined from oedometer tests and presented in Table 2) also decreases with depth.

The purpose of the laboratory portion of this study was to gain a better understanding of how the dynamic shear modulus, *G*, is influenced by the degree of weathering (soil depth), void ratio, *e*, and apparent overconsolidation ratio, AOCR. The depth, index properties, and sand, silt, and clay contents in each of the specimens tested are presented in Table 2. Two samples per depth were tested in the resonant column device for validation of test results. Each specimen was tested at five different levels of confinement corresponding to 0.5, 1, 2, 4, and 8 times the in-situ effective overburden stress. This range of confining pressures was selected to reflect states of stress below, near, and above the in-situ stress state.

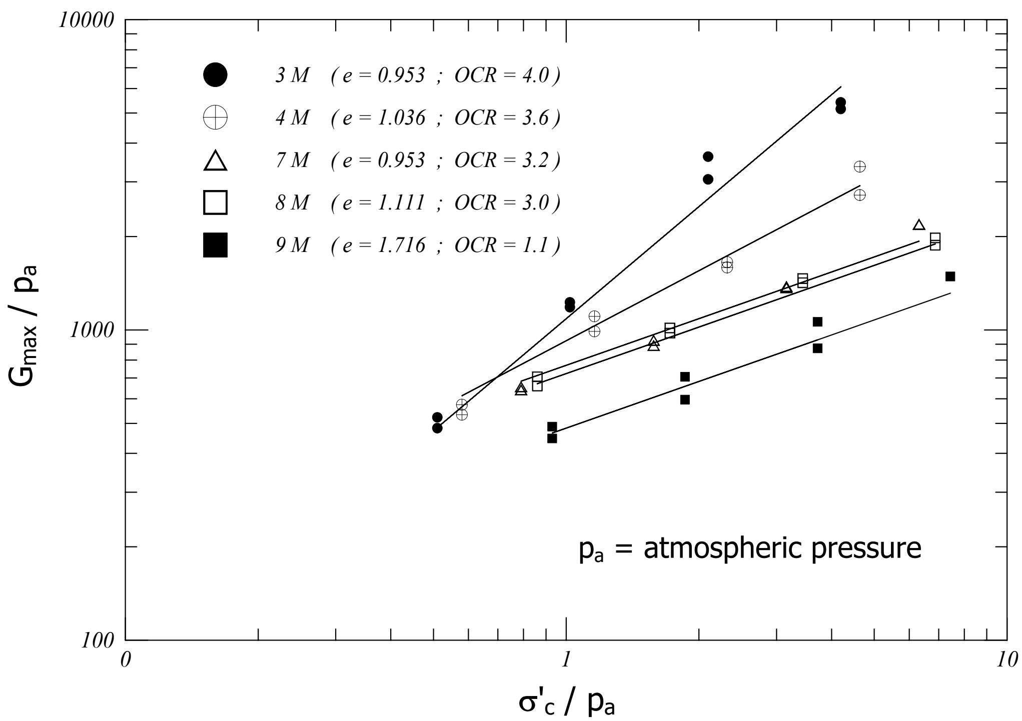


**Figure 2. $G_{max}$ as a Function of $\sigma'_c$ from Resonant Column Tests**

Several expressions have been proposed for estimating the maximum shear modulus, $G_{max}$, as a function of void ratio, $e$, OCR, and confining pressure, $\sigma'_c$. The following normalized empirical equation to evaluate $G_{max}$ at low–amplitude shear strains can be determined from the work of Hardin and Drnevich (1972):

$$\left(\frac{G_{max}}{p_a}\right) = 321\frac{(2.97-e)^2}{1+e}OCR^M\left(\frac{\sigma'_c}{p_a}\right)^N \tag{3}$$

where, $e$ = void ratio ($\leq 2.0$); $M$ = exponent ranging between 0 and 0.5 depending on the PI; $N \approx 0.5$ for most sandy and clayey soils; $\sigma'_c$ = mean effective stress; and $p_a$ = reference atmospheric pressure.

Figure 2 shows all resonant column test data in terms of $G_{max}$ as a function of confinement, $\sigma'_c$, both normalized with respect to the atmospheric pressure, $p_a$, on the basis of soil depth. Regression factors $M$ and $N$ were varied to estimate $G_{max}$ as a function of $\sigma_c'$, $e$, and $OCR$ from Equation 3, and the best-fit lines are shown in Figure 2 (Hoyos & Macari, 1999).

## THRESHOLD STRAIN AND MODULUS REDUCTION

A laboratory study by Borden et al. (1996) focused on the dynamic properties of Piedmont residual soils by performing a combination of resonant column and torsional shear tests using a Stokoe device. These properties were evaluated with respect to confining pressure, shear strain amplitude, cyclic frequency, and number of cycles. Confinement levels ranged from 25 to 100 kPa, and both cyclic frequency between 0.2 and 10 Hz and number of cycles up to $1\text{x}10^6$ were found to have no significant influence on the shear modulus and damping ratio of these residual soils.

Figure 3 shows modulus reduction from the Borden et al. (1996) study as well as this study. These data points are plotted with respect to typical modulus reduction curves as a function of plasticity presented in Vucetic & Dobry (1991). The samples have plasticity indices ranging from NP to 31, and the Borden et al. (1996) test results generally follow the plasticity based trends, with a majority of the data between the PI = 0 and the PI = 30 curves. The Opelika data tend to lie above the expected Vucetic & Dobry (1991) plasticity curves, on the PI = 50 curve.

Major differences were noticed in threshold shear strains, which varied from about 6 $\text{x}10^{-6}$ to 2 $\text{x}10^{-5}$, for the data from both studies. It should be noted that the modulus reduction curves from the Hoyos & Macari (1999) study were performed at 8 times the effective overburden stress. The threshold shear strains were generally related to confining stress, with lower confining stress related to a lower $\gamma^e_{th}$. Some scatter in this trend is likely due to plasticity and interparticle bonding of undisturbed specimens. A micromechanical approach to threshold strain was undertaken by

Dobry et al. (1982) using Mindlin's theory of particle contacts, and a similar approach using Hertz theory of particle contacts has been reviewed (Santamarina & Fam, 1999; Santamarina & Aloufi, 1999). These studies came to similar conclusions that the threshold shear strain is a function of the inverse of the shear modulus of the particles and the confining stress raised to the $^2/_3$ power. The difference in threshold strains from confining stresses of 25 to 100 kPa (Borden et al., 1996 study) would be up to 2.5 times, which is fairly insignificant considering the other soil properties affecting $\gamma^e_{th}$. When the high confinement used in the Hoyos & Macari (1999) study is considered, differences in $\gamma^e_{th}$ due to confinement may be up to ten times those from the Borden at al. (1996) study. Therefore, this stress increase would likely dominate the normalized cyclic shear modulus reduction, leading to a higher apparent plasticity when compared to the Vucetic & Dobry (1991) curves.

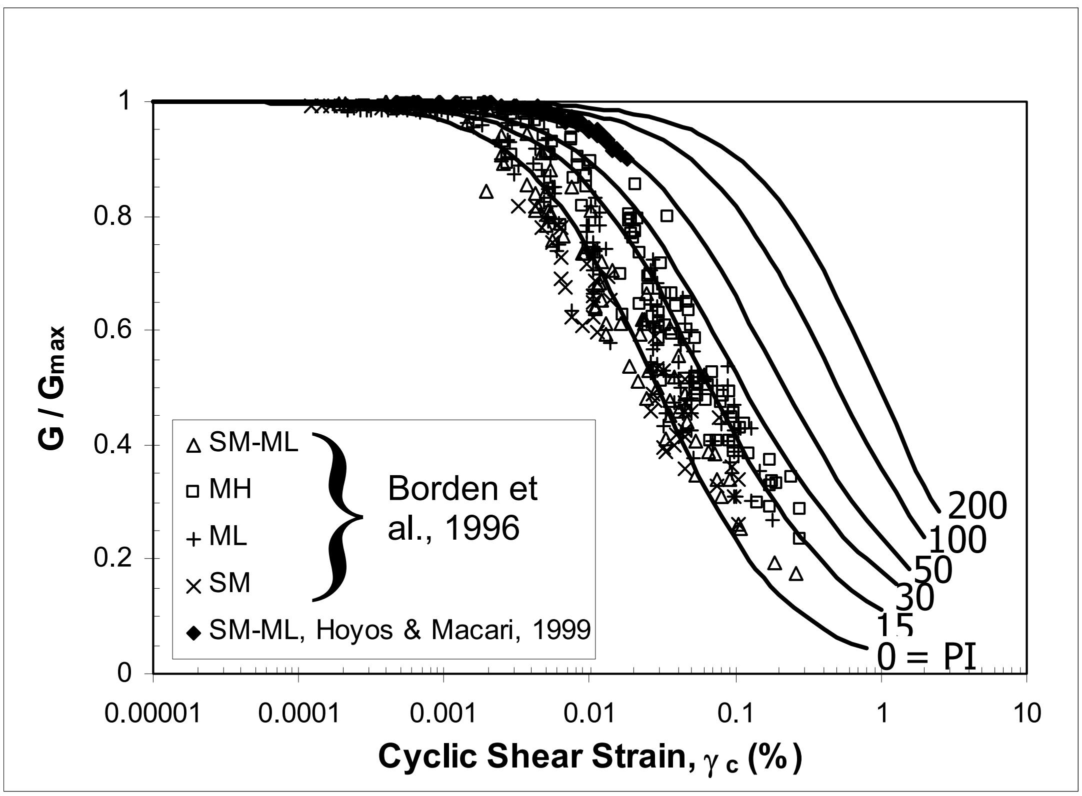


**Figure 3. Laboratory Data for Piedmont Soils Compared to Normalized *$G / G_{max}$* Curves Based on Plasticity as Presented in Vucetic & Dobry (1991)**

## IN-SITU TESTING PROGRAM

In-situ determination of shear wave velocities at the Spring Villa test site included crosshole testing, seismic piezocone testing, seismic flat dilatometer testing, and surface waves testing. Two crosshole arrays were set up with three boreholes each, and measurements were taken at 1-meter depth intervals. To ensure completeness, arrival times were recorded from the source to the first receiver, the source to the second receiver, and between the first and second receiver. Three seismic piezocones and two seismic flat dilatometers were performed with downhole shear wave velocity determined using the pseudo-interval method of analysis (Campanella, et al., 1986). Downhole velocities were taken at approximately 1-meter intervals, while piezocone penetration data were taken at 5-cm intervals, and DMT pressure readings were taken at 30-cm intervals. SASW testing was performed at the Opelika site, with shear wave velocity determined by inversion processing. Figure 4 shows a typical seismic piezocone sounding at Spring Villa with a comparison of shear wave velocities obtained by CHT, SDMT, and SASW.

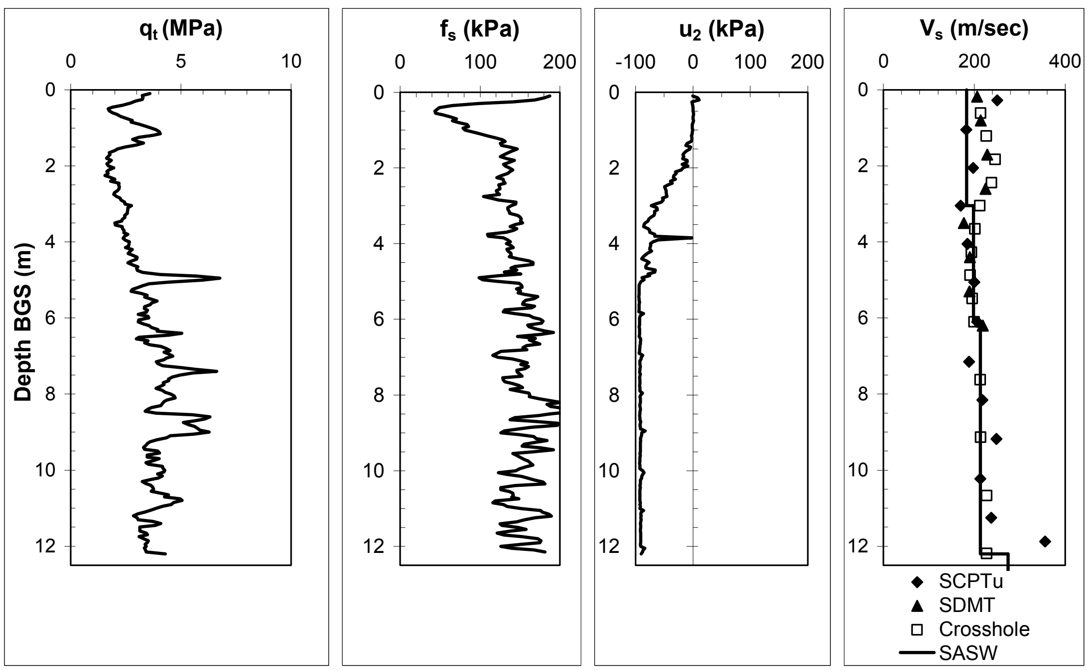


**Figure 4. Seismic Piezocone Sounding at Spring Villa Site**

As determined from laboratory experiments, shear wave velocity is a function of test conditions such as stress state, OCR, void ratio, and strain level of the measurement (Hardin & Black, 1968). It will be desirable to estimate these parameters from in-situ test data to have a better understanding of laboratory and in-situ test results. To determine the maximum shear modulus of the soil, the mass density must be known (Eq. 1). This property must be estimated from in-situ tests or determined from sampling and laboratory testing.

The analysis of preconsolidation pressure by cone penetration tests varies for sands and clays. For sands, penetration is drained for pore pressure measurements taken behind the tip. The tip resistance is primarily controlled by the effective stress friction angle, relative density, and horizontal stress state, which can then be related to OCR through $K_o$, $\phi'$, OCR relationships (Mayne, 1995). For intact clays, penetration is undrained and significant pore pressures develop around the tip, which can be related to OCR through cavity expansion solutions (Chen & Mayne, 1995). The tip resistance is primarily controlled by the undrained shear strength of the soil, which can be related to OCR through normalized undrained shear strength relationships. Similar concerns come into play with analysis of dilatometer data, since $p_o$ and $p_1$ readings are typically drained in sands and undrained in clays.

When evaluating Piedmont residual silty sands and sandy silts, penetration is partially undrained and typically leads to negative porewater pressures behind the tip which dissipate rapidly within about 2 minutes of paused penetration. Since the penetration pore water pressures are negative, this parameter will not provide an accurate estimation of preconsolidation pressure. The sand content will lead to higher tip resistance than a typical overconsolidated clay, and the silt content will lead to lower tip resistance than a typical overconsolidated sand. Therefore, a lower bound of OCR could be estimated from correlations for sand and an upper bound estimate of OCR could be estimated from correlations for clay using CPT tip resistance, but this range will be too wide for any practical application.

The shear wave velocity of the soil will be controlled moreso by the interaction at particle contacts than by grain size or soil type, so $V_s$ from the seismic cone or seismic dilatometer will be more appropriate to assess AOCR than penetration resistance in residual silty materials. Mayne et al. (1998) compared shear wave velocity to preconsolidation stress at clay sites to determine the following relationship ($n = 262$; $r^2 = 0.823$):

$$\sigma'_p = \left(V_s / 4.59\right)^{1.47} \tag{4}$$

where $V_s$ is in m/s and $\sigma'_p$ is in kPa. Whether the AOCR at the Spring Villa site is caused by interlocking of particles, or by desiccation and fluctuation of the groundwater table, the shear wave velocity should provide a reasonable approximation of apparent overconsolidation ratio caused by the residual soil structure.

Void ratio effects shear wave velocity through the coordination number of the fabric (Santamarina & Fam, 1999), and mass density is directly included in shear modulus calculations through Equation 1. In saturated soils, mass density will be directly related to the void ratio through the specific gravity of the particles as, $\rho_{sat} = [(G_s+e)/(1+e)]$. There are a number of relationships to estimate mass density or unit weight from in-situ test parameters. Lunne et al. (1997) present a preliminary approximation of unit weight for various zones in the Robertson et al. (1986) CPT classification chart. The $E_D$ and $I_D$ parameters determined from dilatometer data can be related to total mass density through the approximate diagram presented in Schmertmann (1986). A statistical relationship has been presented by Mayne et al.

(1999) relating total mass density data from gravels, sands, silts, and clays, to shear wave velocity, $V_s$, in m/s and depth, $z$, in meters (n = 727; $r^2$ = 0.730):

$$\rho_t \approx 1 + \frac{1}{0.614 + 58.7(\log z + 1.095)/V_s} \qquad (5)$$

For this study, the shear wave velocity correlation presented in Equation 5 was used for all $G_{max}$ calculations from field data.

## COMPARISON OF FIELD AND LABORATORY MEASUREMENTS

In general, the present study shows excellent agreement between in-situ and laboratory test results. The in-situ test data in Figure 5 present shear wave velocities, maximum shear modulus calculated from Equation 1, and shear strain levels calculated from Equation 2. The resonant column data in Figure 5 are from the tests associated with a confining stress equal to the overburden stress at the associated depth ($\sigma_c' = \sigma_{vo}'$), and were recorded at strain levels below $\gamma^e_{th}$. The strain levels associated with resonant column tests in Figure 5 are the threshold shear strains determined in this study. The values of $G_{max}$ are slightly increasing with depth between 2.5 m and 11 m. The increase in stiffness is due to overburden stress effects, but a slight decrease should also be noted from a reduction in apparent OCR (Eq. 3). Therefore the effects almost counteract each other, resulting in a slightly increasing shear modulus. It is important to recognize the influence of the saturation condition of the soil on its dynamic response, as it is noted from the in-situ tests. Values of $G_{max}$ and $V_s$ are greater above the ground water table (2.4 m), highlighting the importance of matric suction effects on the stiffness of the soil.

Shallow in-situ test strain levels in Figure 5 appear to be above the threshold shear strain, which would indicate that a strain-based correction factor would be necessary to obtain $G_{max}$. Larsson & Mulabdic (1991) state that laboratory values of shear modulus are typically lower than those measured in the field due to disturbance in the sampling process, and that a strain-based correction factor is necessary for soft clays at strains above $10^{-6}$. Correction factors of about 10% at a shear strain levels of $10^{-5}$ are presented in Larsson & Mulabdic (1991), and their application would increase the field determined $G_{max}$. The strain-level corrected field shear modulus may accommodate for disturbance inevitable in laboratory specimens, but this subject remains a concern for future research. Since the laboratory and field values are quite similar in this study and properties of Piedmont residual silty sands are not typical of a soft clay, no strain-based correction factor was applied to lab or field data.

The soil stiffness profile at the Spring Villa site between 2.5 and 11 meters is homogeneous, but different weathering profiles across the Piedmont province will lead to variation in profiles at different locations, and potentially within an individual site. Deviation in values of $G_{max}$ may be attributed to void ratio and OCR (Hardin &

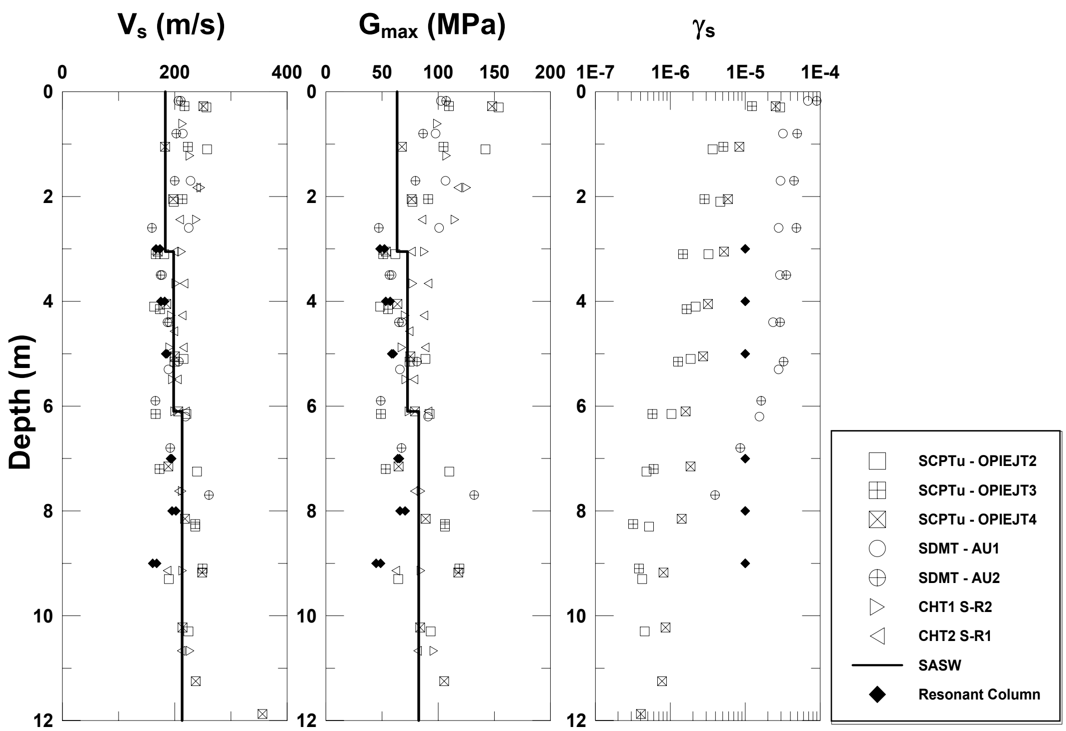


**Figure 5. Comparison of Field and Laboratory Values of Stiffness**

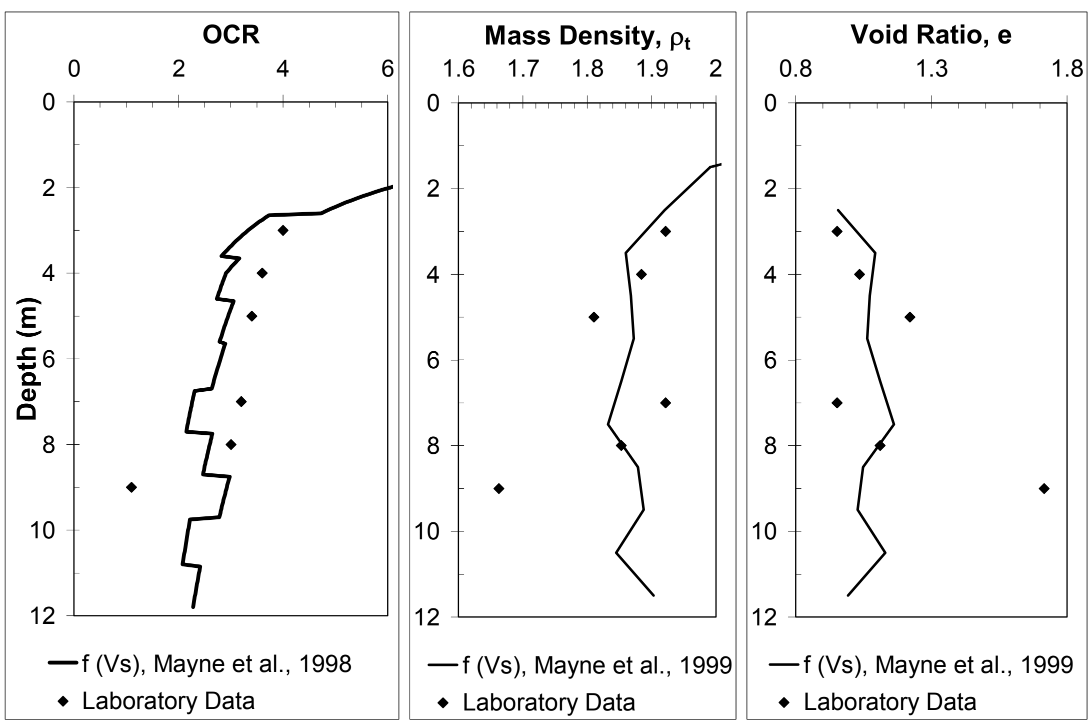


**Figure 6. Estimated Properties from Lab Data and In-Situ Shear Wave Velocity**

Drnevich, 1972), which can be estimated from in-situ test correlations as described in the previous section. Figure 6 presents a comparison of OCR, mass density, and void ratio determined from shear wave velocity test correlations, as well as laboratory data. This figure shows that shear wave velocity can provide reasonable estimations of in-situ properties for residual silty soil conditions that may cause difficulties in the processing of piezocone or flat dilatometer data. The shear wave velocity profile underpredicts the laboratory determined overconsolidation ratio, but the estimation of the preconsolidation stress in Piedmont residual soils requires considerable judgement, which may lead to variability in laboratory results.

## SUMMARY

The mechanisms affecting the maximum shear modulus and reduction schemes for Piedmont residual soils were studied through a comparison of laboratory and field results. The stiffness profile below the water table at the Spring Villa NGES slightly increased due to overburden effects. Laboratory and field values of shear modulus matched closely, so there is no evidence that a strain based correction factor is necessary in Piedmont residual silty sands. Apparent overconsolidation ratios determined from laboratory oedometer tests matched well with estimates from shear wave velocity. Void ratio and mass density were fairly constant over the range of depths studied, and results of shear wave velocity correlations matched well with laboratory values.

As expected, $G_{max}$ increased with increasing confining pressure. There was also a general shift to the right in $G/G_{max}$ curves with increasing confining stress, but this is primarily attributed to an increase in threshold shear strain. The AOCR of the soil contributed to increases in shear stiffness. As depth increased, a drop in AOCR seemed to counteract some of the expected increase in $G_{max}$ due to overburden effects. Apparent overconsolidation ratio did not have any noticeable effect of $G/G_{max}$ reduction curves, but this may have been partially masked by the high confinement used in the laboratory phase of this study. Plasticity index did not significantly affect the maximum shear modulus, but the modulus reduction curves generally shifted to the right with increasing PI as previously shown in Vucetic & Dobry (1991). The slopes of the modulus reduction curves were parallel, but increases in elastic threshold shear strain with plasticity shifted the curves.

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