Invited Lecture, Fourth International Geotechnical Conference, Cairo University, January 2000, pp. 95-120.

Enhanced Geotechnical Site Characterization By Seismic Piezocone Penetration Tests

Paul W. Mayne, PhD, P.E.

Professor, School of Civil & Environmental Engineering
Georgia Institute of Technology, Atlanta, Georgia 30332-0355 U.S.A.
Phone: 404-894-6226 and Fax-2281; Email: pmayne@ce.gatech.edu

ABSTRACT

Hybrid in-situ geotechnical tests provide an optimization of data collection by combining two or more techniques into a single sounding. The seismic piezocone penetration test (SCPTu) offers a versatile approach since it is economical and quickly provides vertical profiles of four independent measurements with depth: cone tip resistance (q_T), sleeve friction (f_s), penetration porewater pressure (u_b), and downhole shear wave velocity (V_s). The shear wave is a fundamental nondestructive property of all solids in civil engineering that corresponds to the small-strain stiffness, designated by the shear modulus (G_0). Recent research shows that G_0 applies to both static and dynamic properties, as well as to both undrained and drained loading conditions in geotechnical situations. Since the traditional penetration readings from the cone correspond to failure states, an entire stress-strain-strength curve can be derived at each depth from the SCPTu results. Applications are presented to illustrate how nonlinear stiffnesses are obtained and used in foundation case histories involving shallow footings and pile foundations.

INTRODUCTION

For soil materials, a complete set of intrinsic properties and engineering parameters is never well known with a high degree of accuracy or reliability. This is because of the natural variability and global uniqueness of geomaterials, as well as the realistic budget constraints that restrict the numbers of tests and undisturbed samples that can be obtained on a project using standard rotary drilling techniques. Moreover, extensive series of laboratory tests are required for the discrete determination of selected parameters, albeit at great expense for high-quality block sampling, specimen preparation, and long test durations.

In the analyses of foundation bearing capacity and slope stability investigations, a geotechnical site characterization requires the assessment of the effective cohesion intercept (c'), effective stress friction angle (ϕ'), and/or the undrained shear strength (s_u) of the various soil layers, as appropriate. Analyses concerned with the magnitude of vertical settlements of shallow foundations require the effective preconsolidation stress (σ_{vmax}') and associated values of compression indices (C_r, C_c, C_c), or alternatively, the constrained modulus ($D = 1/m_v$). Time effects can be considered by evaluating flow parameters, namely the coefficient of consolidation (c_v) and the soil permeability (k), or long-term creep effects which are often expressed in terms of the coefficient of secondary compression (C_α). For specific concerns involving pile foundations under axial loading, the analyses may require the assessment of soil adhesion factors (α or β parameters), point bearing stresses (q_{ult}), elastic or Young's modulus (E), Poisson's ratio (ν), as well as subgrade reaction coefficients (k_s). Additional parameters of interest may include the horizontal stress state (K_0), unit weight (γ_T), and damping ratio (D_c), or the inplace relative density (D_R) and potential for soil liquefaction during seismic events. With such a large selection of geotechnical parameters, it is necessary to take a suite of measurements if one hopes to characterize many aspects of soil behavior, as well as provide input to FEM or FLAK analyses.

As a consequence to the large number of soil parameters and properties, there has been increased interest in the use of in-situ tests to provide the geostratigraphy and quick assessment of soil properties during the site exploration. This is not to convey the notion that laboratory testing competes with field testing, but in fact, the two are actually complementary to each other. Lab testing offers full control over boundary conditions, strain rates, and drainage conditions on selected quality specimens. In-situ testing offers immediate and continuous profiling of the subsurface materials.

In some instances, some design engineers unfortunately rely solely on soil test borings to provide almost all of the necessary geotechnical data for their evaluations and analyses, perhaps supplemented by a few laboratory tests. It is quite unrealistic to believe that the one-number from the standard penetration test (SPT) can provide all of the adequate and reliable information for analysis. Figure 1 depicts the incredulous wish-list thinking of this state-of-practice. A single N-value is obtained using ASTM D-1586 guidelines and hopefully corrected to 60 percent energy efficiency (ASTM D-4633). The single value is often subsequently utilized via empirical correlations to generate a wide

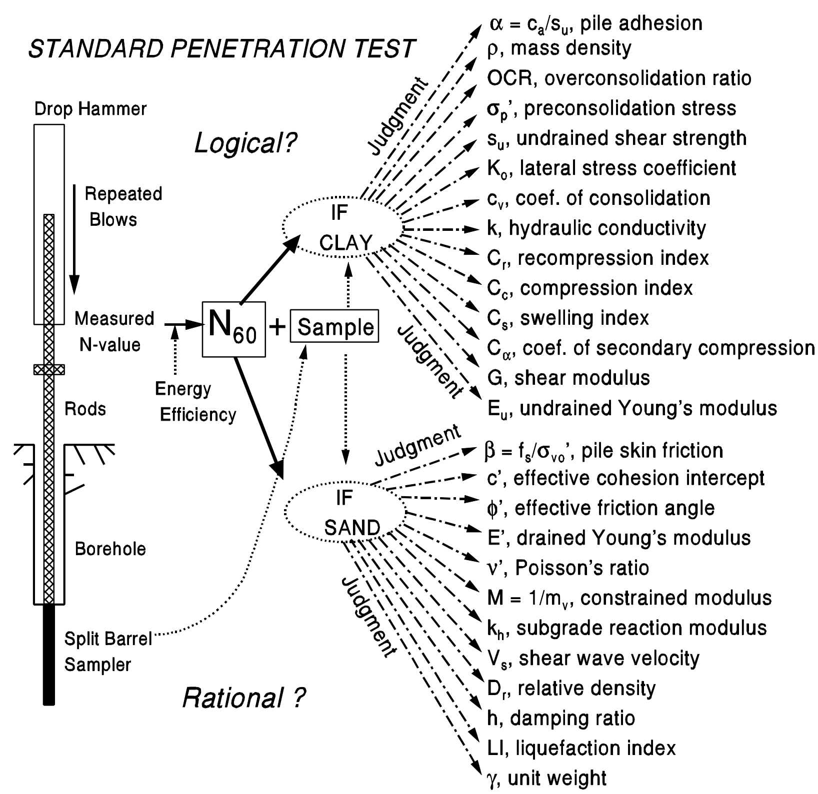

Figure 1. Over-Reliance on SPT-N Values for Interpretation of Geotechnical Parameters.

number of geotechnical parameters that are input into routine computer geotechnical analyses. This over-reliance on the SPT-N value is improper and often results in uneconomical and non-optimal designs of foundations, walls, slopes, and earthworks involving ground modification.

With the advent of more sophisticated computer software (finite difference, discrete element, and finite elements) comes a demand for higher quality input parameters. This is best achieved by complementing the soil boring program with additional in-situ and laboratory tests. In this paper, available in-situ testing methods are reviewed, with a primary emphasis on the seismic piezocone penetration test. It is believed that the implementation of enhanced in-situ testing can provide more accurate numbers for analysis, remove the reliance on "judgment", and thus provide better economy in design and performance.

IN-SITU TESTING DEVICES

Primarily in the last two decades, many in-situ testing devices have been introduced for evaluating the geostratigraphy and soil engineering properties of the ground. The traditional approach to site investigation is to drill soil borings by rotary methods and obtain samples for laboratory testing. Difficulties in pushing thin-walled tubes coupled with soil disturbance due to transportation, extrusion, trimming, moisture losses, and stress-release effects result in laboratory specimens of poor quality, perhaps not representative of field conditions. Laboratory tests are somewhat expensive, require long test durations, and provide only discrete values at select locations. In comparison, in-situ methods test the soil in its natural environment under current geostatic and anisotropic stress states. In-situ testing is fast, continuous, and provides immediate results for use in analysis. Thus, the optimal site characterization program involves a series of soil borings, complemented by a series of in-situ tests and laboratory reference testing.

Over 50 different in-situ field testing devices have been developed for either general or specific measurements in soil (Robertson 1986; Lunne et al. 1992). Some of the common and readily available in-situ tests are listed in Table 1, including the standard penetration test (SPT), cone penetration test (CPT), piezocone (PCPT), flat dilatometer (DMT), pressuremeter (PMT), vane shear test (VST), and several geophysical methods: crosshole test (CHT), downhole test (DHT), and spectral analysis of surface waves (SASW). The utilization and interpretation of these basic tests is well-established (Wroth 1984; Kulhawy & Mayne, 1990). Table 1 also lists variations of the tests whereby additional data are obtained during conduct of the test. The philosophy here is that more measurements are better in helping to assess a full suite of soil parameters needed for analysis and design.

While analytical, theoretical, and numerical modeling of penetration testing and probe installation effects have aided in a better understanding of field measurements, the interpretation of data is made difficult by the inherent complexities associated with natural materials, particularly initial stress state, anisotropy, soil fabric, particle shape, sensitivity, aging, mineralogy, geochemistry, and other factors. Thus, radically new geotechnical devices are hampered by the need to conduct extensive series of verification and calibration tests, often resorting to empiricism in the final interpretation schemes. The routine in-situ tests (SPT, CPT, DMT, PMT) have at least two decades of experience in practical applications. Therefore, the development of hybrid devices, particularly penetration tests with geophysical measurements, is particularly attractive because of the interpretative procedures for P-, S-, and R-waves are fairly well-established.

Of particular interest, the standard cone penetration test provides a tip or point resistance (q_T) and sleeve or side friction resistance (f_s), and in the framework of scaled systems, may be considered analogous to a miniature driven pile foundation. For site characterization, a variety of specialized

Table 1. Listing of Common In-Situ Tests and Variants

Field Test Method	Test Designation	Test & Variational Procedures	Number of Measurements
Standard Penetration	SPT	ASTM D-1586 (N_{60})	1 (+ drive sample)
	SPTT	N-value + torque	2 (+ drive sample)
Vane Shear Test	VST	ASTM D-2573 (s_u, S_t)	2 (peak + remolded)
	VST/r	+ rotation angle	3 (strain level)
Cone Penetration	CPT	ASTM D-3441/D-5778	2 (q_c, f_s)
	SCPT	+ downhole seismic data	4 (+V_p, +V_s)
	PCPT	+ porewater pressure	3 (+u)
	PCPT/d	+ dissipation tests	3 (time effects)
	SPCPT	[q_c, f_s, u, V_p, V_s]	5 (combined)
	RCPT	+ resistivity	3 (+conductivity)
Pressuremeter Test	PMT	ASTM D-4679	4 (p_L, E, σ_{ho}, τ_{max})
	SBPMT	self-boring device	4 (complex to run)
	PIPMT	push-in type	3 (p_L, E, τ_{max})
	FDPMT	full-displacement	3 (p_L, E, τ_{max})
	CPMT	FDPMT type with cone	5 (+ CPT data)
	PCPMT	FDPMT with piezocone	6 (+ PCPT data)
Flat Dilatometer Test	DMT	pneumatic readings	2 (p_o and p_1)
	DMT/t	+ blade thrust	3 (p_o, p_1, q_D)
	DMT/c	+ deflation reading	3 (p_o, p_1, p_2)
	DMT/d	dissipation readings	3 + time decay
Geophysical Tests:			
Seismic Refraction	SR	P-waves from surface	1 (Compression wave)
Surface Waves	SASW	R-waves from surface	1 (Rayleigh wave)
	SASW/d	R-waves + damping	2 (+ attenuation)
Crosshole Test	CHT	P- and S-waves in boreholes	2 (body waves)
	CHT/d	CHT + damping ratio (D)	3 (+ attenuation)
Downhole Test	DHT	P- and S-waves with depth	2 (body waves)
	DHT/d	+ damping with depth	3 (+ attenuation)
Hybrid Tests:			
Cone Pressuremeter	CPMT	Cone & pressuremeter	5 (CPT + full-PMT)
	PCPMT	Piezocone & pressuremeter	6 (PCPT + FDPMT)
Seismic Piezocone	SPCPT	Dowhhole & piezocone	6 (DHT + PCPT)
Seismic Dilatometer	SDMT	Downhole & dilatometer	5 (DHT + DMT)
Seismic Pressiocone	SPCPMT	S-wave, cone, & FDPMT	9 (DHT+PCPT+PMT)

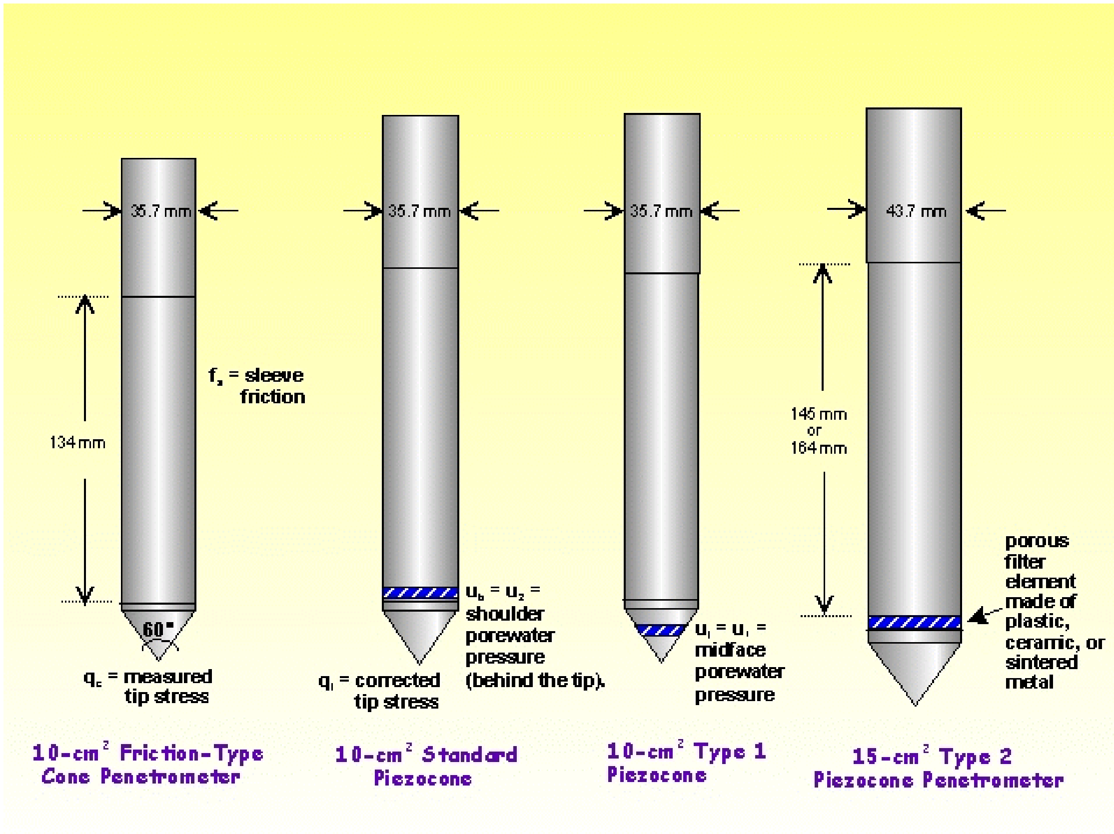

Figure 2. Standard Types of Cone Penetrometers in Use in North & South America.

cone penetrometers is available to facilitate additional separate measurements directly during the sounding (Mayne, et al. 1995; Lunne, et al. 1997). A selection of penetrometers is shown in Figure 2, including single-, dual-, and triple-element types for taking porewater pressure readings. These have midface (u_1) and shoulder (u_2) elements, as well as behind the sleeve (u_3), per the nomenclature established by Campanella & Robertson (1988).

Most of the in-situ tests can be performed using conventional drill rigs to hydraulically push the penetrometers, blades, or probes into the ground, without the need for a boring. The success depends upon the size of the rig, expertise of the driller, and the hardness of the ground. In soft ground, it is quite feasible to push cone soundings and flat dilatometers to depths of 30 meters or more using a standard drill rig. Earth anchoring systems can be used to increase the push capacity of rigs.

Specialized vehicles are available which facilitate pushing and optimize productivity. A large cone truck is shown in Figure 3, illustrating the enclosed cabin that allows testing during inclement weather. Some rigs are equipped with track-mounting to allow access in difficult terrain, as shown in Figure 4. Thus, rain or shine, snow and sleet, or hot and humid climate, the field testing continues and the planned testing schedules can be met. These cone trucks are now wide-spread throughout many parts of the world.

In some instances, the upper ground surface is covered by asphalt, concrete, or a layer of gravelly fill. With large cone trucks, it may be possible to push a "dummy cone" to break through a hard or cemented zone. Afterwards, a calibrated electric cone can be used to continue the sounding and take

Figure 3. Closed-Cab Cone Truck in Georgia.

Figure 4. Track-Mounted Cone Rig in Alabama.

continuous measurements with depth. Conventional drill rigs are versatile for both augering and hydraulic pushing capabilities. However, these drill rigs typically weigh only about 10 tonnes and push from the back end, thus only 5 tonnes reaction is available. Cone trucks weigh 25+ tonnes and push through the center of gravity, and therefore capable of greater penetrability.

SMALL-STRAIN STIFFNESS

A fundamental property of geomaterials is the shear wave velocity (V_s) which is representative of the nondestructive response at very small strains ($\gamma_s < 10^{-6}$ decimal). Measurements of V_s can be obtained for all solid materials in civil engineering practice, including steel, concrete, wood, fibreglass composites, and graphite alloys, as well as soil, rock, and intermediate geomaterials (i.e., saprolite). Because of this universality in application, V_s is therefore attractive as a basic measurement quantity for systematically characterizing differences between natural geomaterials and the elastic stress-strain behavior in terms of engineering mechanics principles. Since water cannot sustain shear, S-wave measurements of soils are unaffected by the presence of groundwater (unlike the P-wave).

The measurement of V_s in soils can be accomplished using laboratory and/or field tests and these are illustrated by Figure 5. Tests on small laboratory specimens include the resonant column (RC), torsional shear (TS), bender elements (BE), and special triaxial systems with internal strain measurements. Field test methods include the crosshole test (CHT), downhole test (DHT), spectral analysis of surface waves (SASW), seismic cone penetration test (SCPT), seismic refraction (SF), and suspension logging technique. Detailed reviews of the laboratory and field methods for measurement of G_{max} are given by Woods (1994) and Campanella (1994), respectively.

The stiffness of materials at small strains is finite and denoted by the low-strain shear modulus $G_{max} = G_0 = \rho V_s^2$, where ρ = total mass density of the material. Extensive research has shown that the value of G_0 in soils is the same for both static (monotonic) and dynamic loading conditions (Jamiolkowski, et al. 1994a; Tatsouka et al. 1997). The magnitude of G_0 is also independent of drainage because the strains are too small to cause excess porewater pressures, and thus applies to both drained and undrained conditions. The parameter G_0 is relatively insensitive to the overconsolidation ratio (OCR) of sands (Alarcon-Guzman, et al. 1989; LoPresti et al. 1993), and despite earlier studies

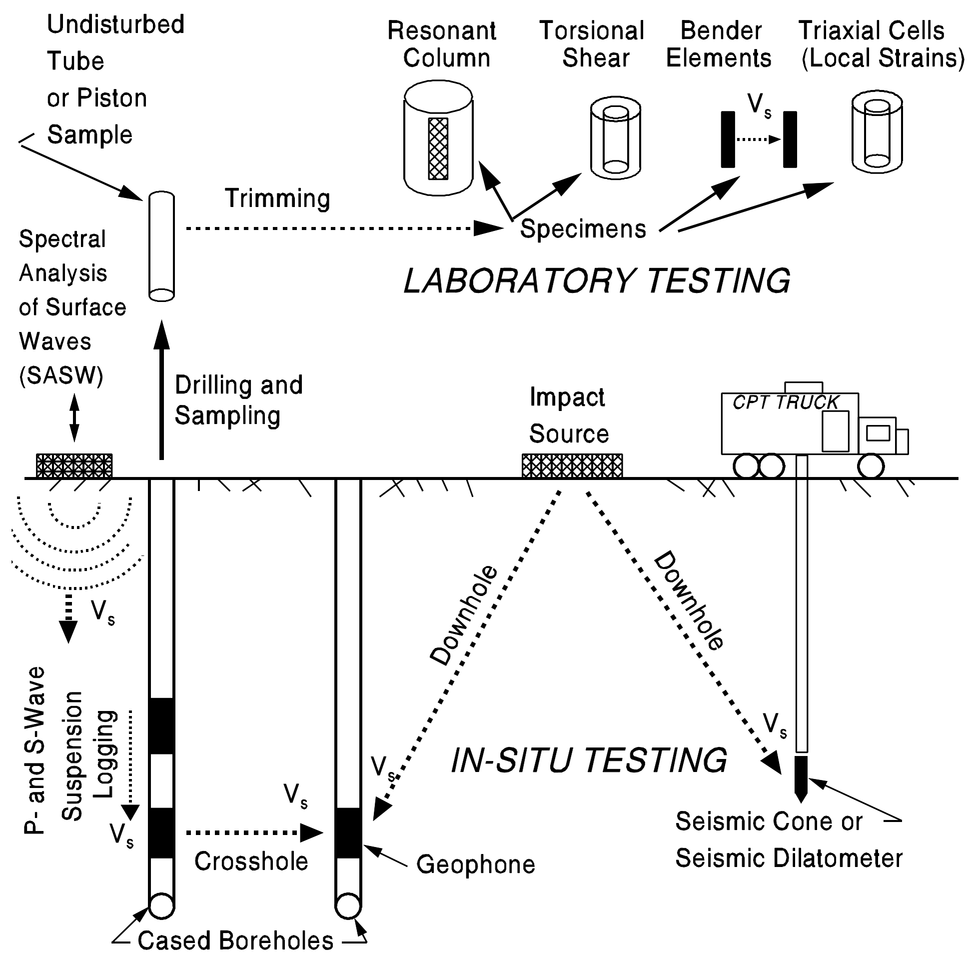

Figure 5. Laboratory and Field Methods for Determination of Shear Wave Velocity (V_s) and Small-Strain Stiffness ($G_0 = \rho V_s^2$) of Soils.

(Hardin and Black, 1968), recent research has show G_0 to be somewhat insensitive to OCR in natural clays (Jamiolkowski, et al. 1994b). This independency facet of G_0 therefore establishes its importance in defining a universal reference or benchmark value of stiffness for deformation problems (Burland, 1989). A quantification of the initial stiffness in terms of G_0 is appropriate to analyses involving foundation systems, retaining walls, tunnels, and pavement subgrades, as well as problems involving cyclic and seismic loading conditions.

Field test results from Rayleigh wave surveys (or, SASW for spectral analysis of surface waves) for characterizing the G_0 profile in residuum of the Piedmont geologic province is presented in Figure 6. The site served as a test area for instrumented drilled shaft foundations that were loaded in axial compression (Harris & Mayne, 1994). The natural soils consist of very silty fine sands derived from the weathering of the underlying gneiss and schist bedrock. The results of an electric cone sounding at the test site are also presented. Figure 6 indicates the G_0 profile at this site is adequately represented by a simple Gibson-type soil with modulus increasing linearly with depth. While the profiles shown were obtained from two separate tests (SASW and CPT), it is now quite easy and economical to obtain the same information via a hybrid test, the seismic cone (SCPT).

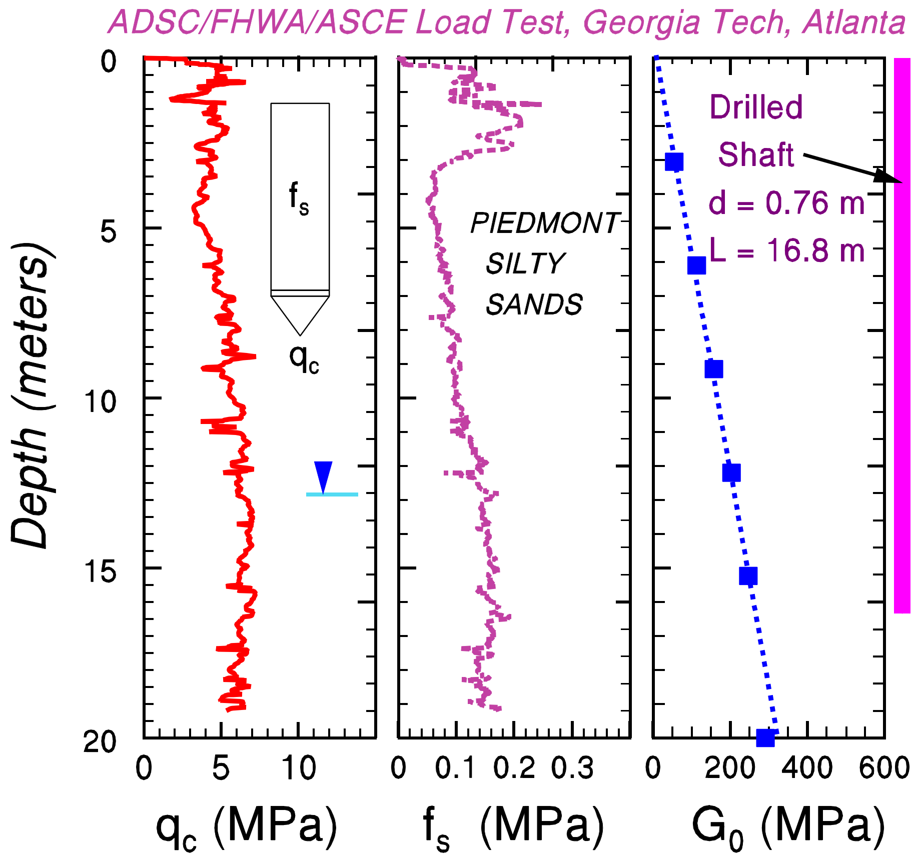

Figure 6. Small- and Large-Strain Measurements in Residual Silty Sand, Atlanta, Georgia.

With the seismic piezocone penetration test (SCPTu), a single sounding produces four separate readings with depth: tip resistance (q_T), sleeve friction (f_s), pore pressure (u_b), and shear wave arrival time (t_s) from a downhole procedure. Figure 7 shows the results of a SCPTu from Memphis, Tennessee that penetrates a 4-m silt layer overlying a 7-m sand layer. Below a depth of 11 meters, a stiff overconsolidated clay is encountered, as evidenced by the high penetration porewater pressures (well above hydrostatic values). From 20 to 22 m, an apparent crustal layer of fissured clay occurs.

For the downhole portion, a velocity geophone located within the penetrometer is used to measure the time arrivals of shear waves. A horizontal plank positioned parallel with the geophone axis and at the ground surface is struck to generate a source rich in shear wave energy. The S-waves are polarized horizontally and emanate vertically as a downhole test (DHT). The DHT portion is conducted at each successive change of of cone rods, generally one meter apart. Since each rod takes approximately 30 seconds to install and because arrival times are typically between 20 and 100 milliseconds, there is little loss in production time testing from normal CPT operations. A pseudo-interval type shear wave velocity (V_s) is determined by interval readings (Campanella 1994). An advantage of the SPCPT is its ability to provide measurements at two opposite regions of the stress-strain-strength curve (i.e., G_{max} and τ_{max}) and therefore elastic stiffnesses and failure parameters of the soil are easily mapped with depth from a single sounding (Burns & Mayne, 1996).

Results of a SCPTu in residual soils of the Piedmont geology are presented in Figure 8. The silty to sandy soils are derived from the weathering of schist and gneiss. Shear wave velocities from several different tests are comparable, including CHT, SASW, and DHT from both SDMT and SCPTu.

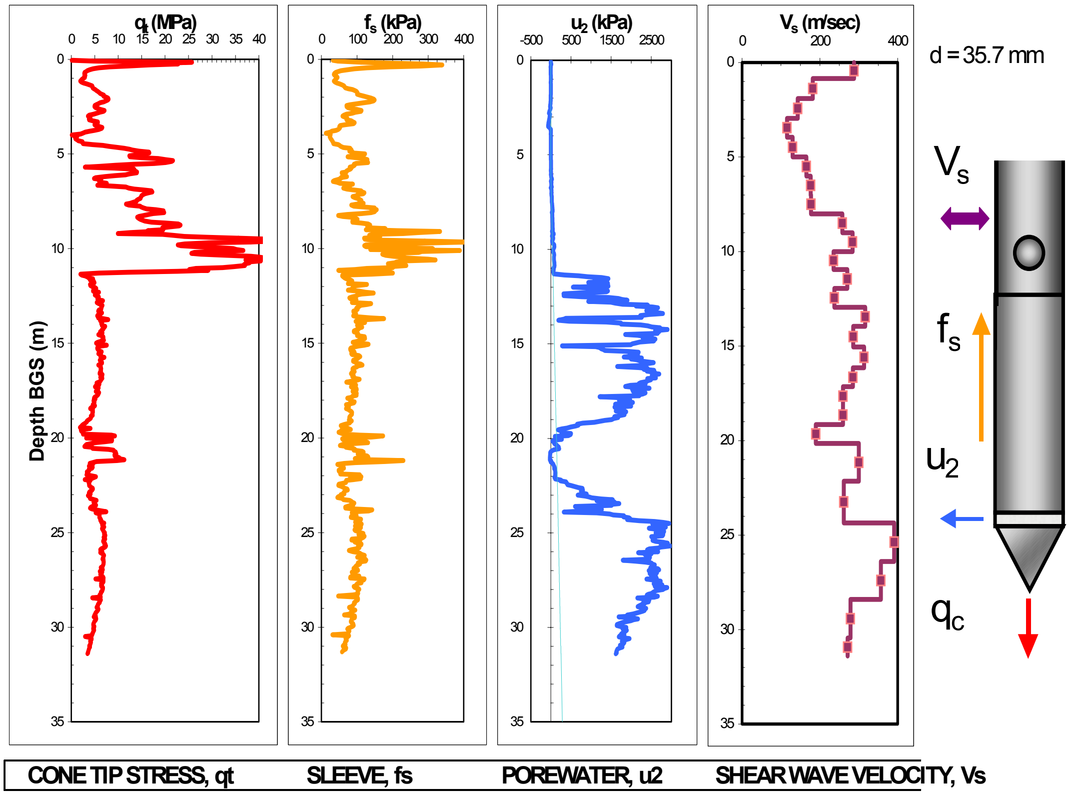

Figure 7. Seismic Piezocone Results in Sedimentary Soils from Memphis, Tennessee.

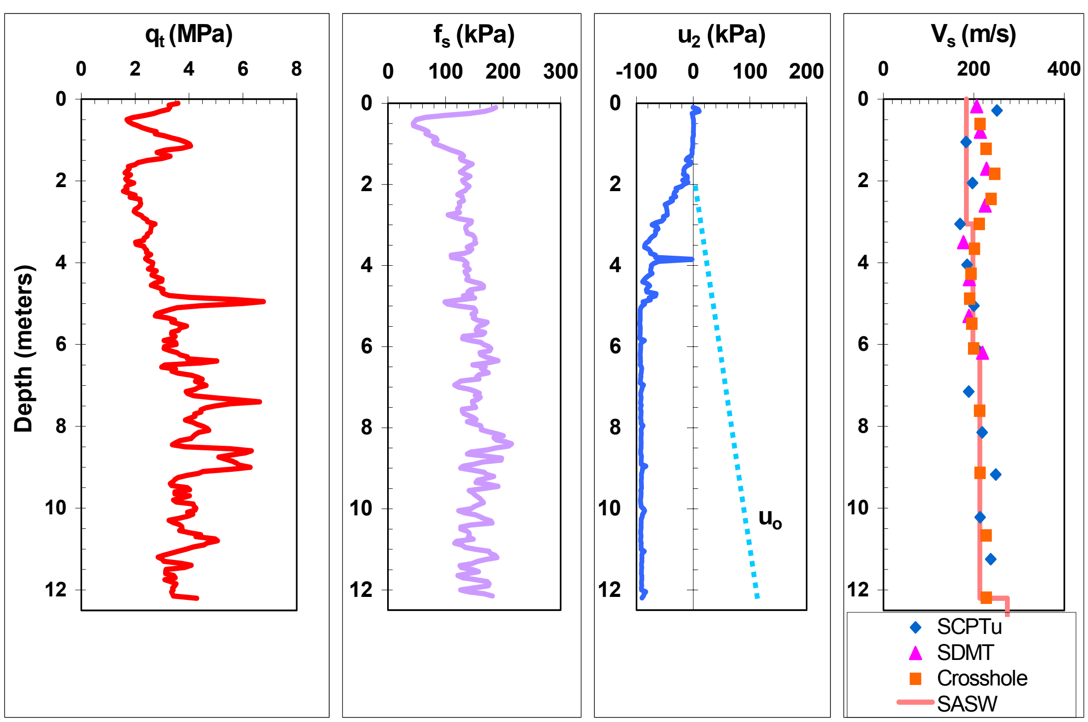

Figure 8. Seismic Piezocone Results in Residual Soils from Opelika, Georgia.

MODULUS DEGRADATION

The stress-strain-strength behavior of soil materials is highly nonlinear at all phases of loading with a true linear elastic behavior observed only at very small strains in the vicinity of the operational G_{max}. As a consequence, the quantification of modulus degradation with level of shear strain (G/G_{max} versus $\log\gamma_s$) is currently one of the most active research areas in geotechnical engineering (e.g., Vucetic & Dobry 1991; Shibuya et al. 1992). Alternatively, modulus degradation can be expressed as a function of mobilized stress level, or G/G_{max} versus τ/τ_{max} (Tatsuoka & Shibuya, 1992; LoPresti et al. 1993; Tatsuoka et al. 1997). Note that the ratio $\tau/\tau_{max} = 1/FS$, where τ = shear stress, τ_{max} = shear strength, and FS = factor of safety. In this manner, the results can be obtained from full-scale performance data where an equivalent modulus can be backcalculated and compared with in-situ measurements of small-strain properties.

A number of different expressions have been proposed to represent modulus degradation. The simple hyperbola (Kondner, 1963) offers the convenience in that only two parameters are required: (1) shear modulus, G_{max}, and (2) maximum shear stress, or the shear strength, τ_{max}. Notably, the simple hyperbola fails to adequately model the complete and complex behavior of soils in most instances over the full range of strains (Tatsuoka and Shibuya, 1992). Consequently, a number of modified hyperbolic expressions have been proposed (Duncan and Chang, 1970; Hardin and Drnevich, 1972; Prevost and Keane, 1990; Fahey and Carter, 1993; Mayne, 1994; Fahey et al. 1994), but which increase the number of required parameter to either 3 or 4. The well-known Ramberg-Osgood expression requires 4 parameters but results in shear stresses increasing indefinitely without bound (Burghignoli et al. 1991). A periodic logarithmic function has also been proposed (Jardine et al. 1986, 1991), yet relies on 5 curve-fitting parameters for success. Most recently, Puzrin & Burland (1996, 1998) present a logarithmic stress-strain function for soils and rocks that utilizes one, three, or four parameters, depending upon available information.

Laboratory torsional shear tests show that the monotonic decay of stiffness is at a faster rate of degradation than cyclic tests. Data from Toyoura sand (Teachavorasinskun, et al. 1991) are presented in Figure 9 in terms of mobilized stress level. While the cyclic torsional shear tests show modulus degradation reasonably represented by a standard hyperbola, the monotonic tests lose stiffness much quicker. For monotonic torsional shearing of normally-consolidated sands, the modified hyperbola proposed by Fahey and Carter (1993) takes the form:

$$G/G_{max} = 1 - f(\tau/\tau_{max})^g$$

where f and g are fitting parameters and G = shear modulus = $E/[2(1+\nu)]$, The value of τ/τ_{max} may be considered as the reciprocal of the factor of safety, or 1/FS, and thus is equivalent to mobilized load level, Q/Q_{ult}.

For a conventional hyperbola, f = g = 1. For monotonic loading of uncemented and unaged quartzitic sands and insensitive and unstructured clays, results from laboratory torsional, triaxial, and simple shear tests, adopted values of f = 1 and g = 0.3 appear to give reasonable approximations for first-order evaluations (Mayne 1995; Burns & Mayne, 1996). Additional efforts looking at backcalculated moduli from full-scale load tests indicate similar values for footings and pile foundations (Mayne & Dumas, 1997). For example, the modulus degradation for Pisa clay (LoPresti et al. 1995) is shown in Figure 10 with the modified hyperbola form. Note here that the shear strain is obtained from the relation: $\gamma_s = \tau/G$ that relates to an equivalent axial strain, $\epsilon_a = \gamma_s/(1+\nu)$.

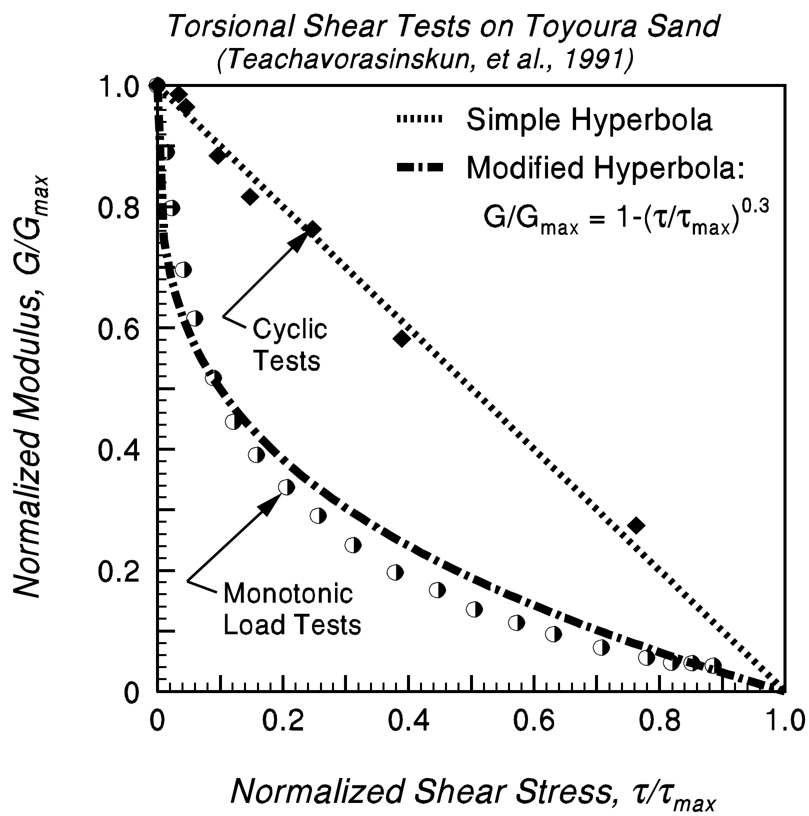

Figure 9. Modulus degradation results for Toyoura sand (Teachavorasinskun et al. 1991).

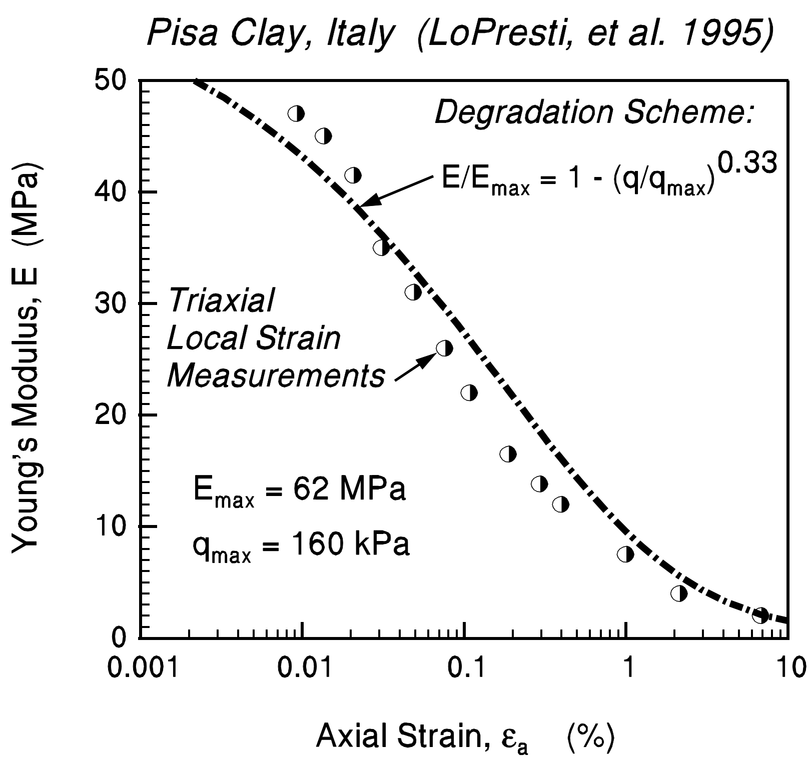

Figure 10. Modulus degradation for Pisa clay (LoPresti et al. 1995).

YIELD SURFACES AND STRENGTHS OF SOILS

The strength and deformational characteristics of soils are controlled by their three-dimensional yield surfaces (Diaz-Rodriquez et al. 1992). This facet is most commonly determined by laboratory tests on high-quality samples, particularly natural clays, as evidenced by the preconsolidation stress (σ_p') obtained in one-dimensional consolidation tests. The yield stress or preconsolidation separates elastic from plastic behavior and distinguishes deformational problems from those involved with failure or bearing capacity. With cone penetration, the strength of coarse-grained materials (sands) can be obtained from limit plasticity solutions, whilst for fine-grained materials (silts and clays), the strength relates to the degree of preconsolidation (Wroth 1984). A full suite of soil parameters interpreted from CPT results is summarized by Mayne et al. (1995).

For sands, the effective stress friction angle can be evaluated from the normalized cone tip resistance, $q_{c1} = (q_c/p_{atm})/(\sigma_{vo}'/p_{atm})^{0.5}$, where p_{atm} = a reference stress equal to one atmosphere (1 atm. = 1 bar = 100 kPa). The expression based on statistical analyses of calibration chamber test data on uncemented quartzitic sands is (Kulhawy & Mayne, 1990):

$$\phi' = 17.6^\circ + 11.0^\circ \log (q_{c1})$$

Figure 11 shows the application to the silty sand encountered at the Georgia Tech campus site in Atlanta, Georgia. Results of thirteen consolidated triaxial shear tests conducted on undisturbed tube samples by two commercial laboratories determined $34.5^\circ < \phi' < 36^\circ$ and $c' = 0$ for this site. These agree with the CPT interpretations over the depth range from 4 to 16 meters in the natural residual soils, despite the high fines content of the soils (approx. 70% fine sand, 25% silt, and 5% clay).

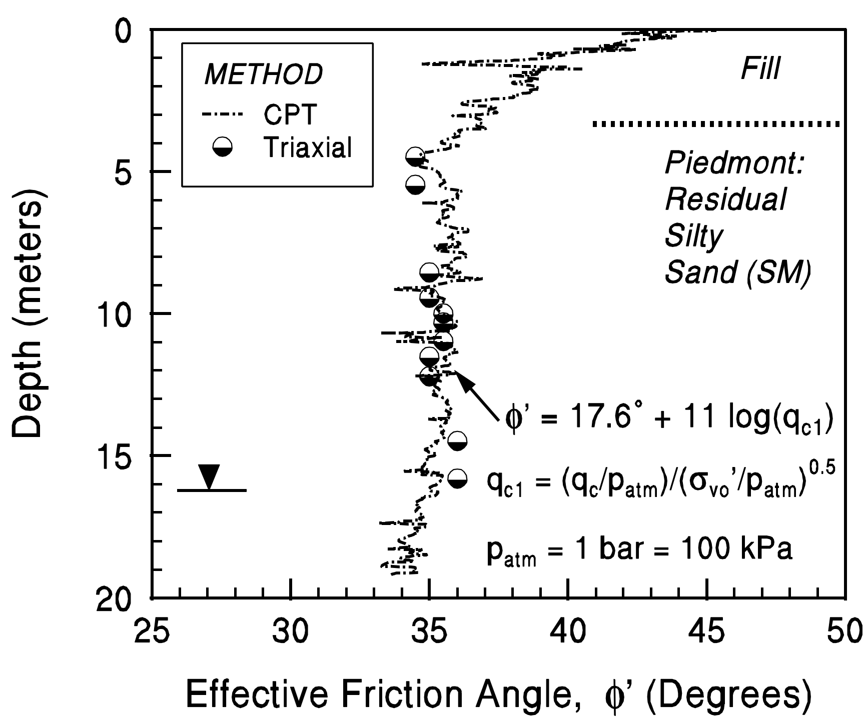

Figure 11. Effective Stress Friction Angle of Piedmont Silty Fine Sand at Atlanta Test Site.

For clays, closed-form piezocone expressions for profiling the overconsolidation ratio (OCR = σ_p'/σ_{vo}') can be applied to data from soft to stiff intact clays, as well as hard and fissured clay deposits. The approach uses the effective cone tip resistance (q_T - u_m), where u_m is the measured penetration porewater pressure. Commercial piezocones obtain measurements of penetration pore water pressure at one of two commonly preferred positions: (1) on the cone tip/face (u_t or u_1), or (2) near the shoulder behind the cone tip (u_b or u_2). Maximum profiling capabilities in clay deposits occur with Type 1 piezocones. On the other hand, Type 2 piezocones are required if the measured cone tip resistances (q_c) are to be corrected (q_T) because of pore pressure effects acting on unequal projected areas of the cone (Jamiolkowski, et al. 1985; Lunne et al. 1986a; Campanella and Robertson, 1988). The important difference is that Type 1 cones always give positive pore pressures, whereas Type 2 cones give positive pore pressures in soft to firm intact clays, but zero to negative pore pressure response in very stiff to hard fissured materials (Mayne et al. 1990). Figure 12 shows results from both piezocone types advanced in a firm lacustrine clay in Port Huron, Michigan and indicates the hierarchy of measurements ($q_T > u_1 > u_2$).

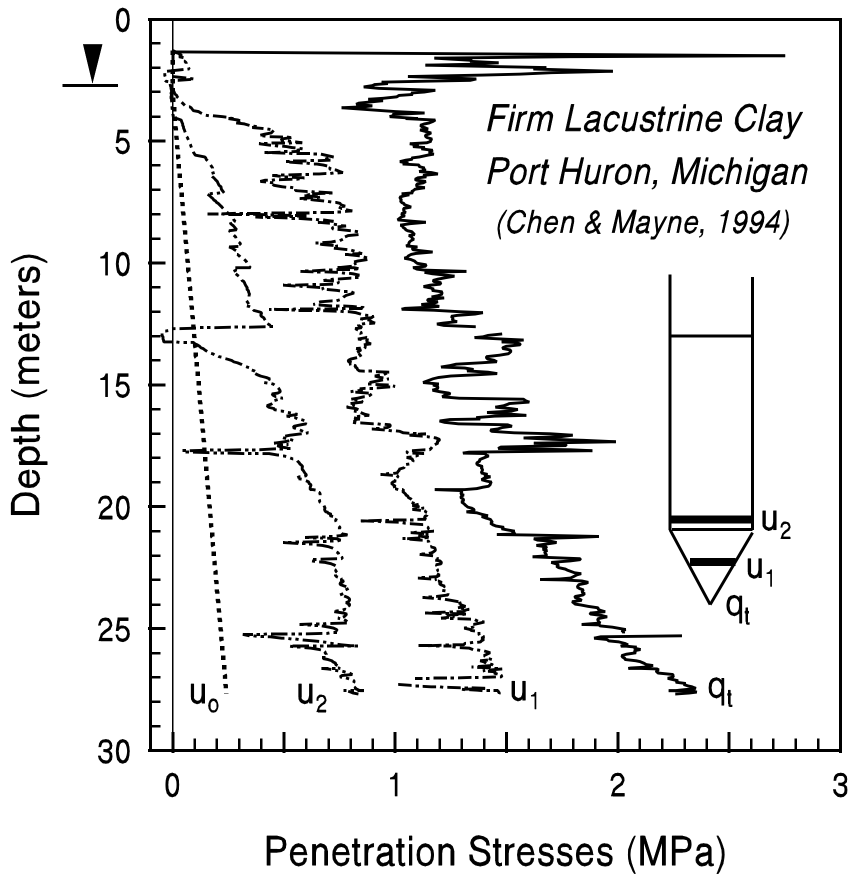

Figure 11. Midface- and Shoulder-Type Piezocone Soundings in Michigan Clay Deposit.

For clays, piezocone expressions have been derived from considerations of cavity expansion theory and modified Cam Clay to interrelate the in-situ overconsolidation ratio (OCR) with cone tip resistance (q_T), penetration pore water pressure (u_1 or u_2), and effective overburden stress (σ_{vo}'). The approach attempts to balance theory with experimental field test results and details are provided elsewhere (Mayne, 1991, 1992). In the simplified version of the model given here, no attempt has been made to account for the effects of initial stress state (K_o), strength anisotropy, stress rotation, or strain rate.

Two input parameters to the analytical model include: (1) the effective stress frictional parameter $M = 6\sin\phi'/(3-\sin\phi')$, in which ϕ'= effective stress friction angle, and (2) the Λ = plastic volumetric strain ratio = $1 - C_s/C_c$, in which C_s = swelling index and C_c = virgin compression index. For natural intact and uncemented clays, the parameter Λ averages about 0.75. In certain structured and cemented materials, however, the value of Λ may be as high as 1.0. For the two primary piezocone types, the predictive forms are (Mayne & Chen, 1994):

TYPE 1 with midface pore pressures:
$$OCR = 2 \cdot \left[\frac{1}{1.95 \cdot M} \cdot \left(\frac{(q_T - u_1)}{\sigma_{vo}'} + 1\right)\right]^{1/\Lambda}$$

TYPE 2 with shoulder pore pressures:
$$OCR = 2 \cdot \left[\frac{1}{1.95 \cdot M + 1} \cdot \left(\frac{(q_T - u_2)}{\sigma_{vo}'}\right)\right]^{1/\Lambda}$$

At low OCRs, the model is not extremely sensitive to either M or Λ. Thus, typical representative values of $M = 1.2$ (corresponding to $\phi' = 30°$) and $\Lambda = 0.75$ for natural unstructured clays can be used for routine practice.

Many clay sites exist worldwide with reference oedometric profiles of OCR and piezocone test data for model verification. For illustration here, predictions of OCR using data from both types of piezometric elements are presented for four sites shown in Figure 13. These include: (13a) lightly to normally consolidated soft clay at the Bothkennar research site in Scotland (Powell, et al. 1988; Jardine, et al. 1995); (13b) medium stiff moderately overconsolidated deposit of lean sensitive clay at Haga, Norway (Lunne, et al. 1986a); (13c) hard microfissured and cemented clay at Taranto, Italy (Jamiolkowski, et al. 1985; Battaglio, et al. 1986); and (13d) heavily overconsolidated and fissured London clay at Brent Cross (Lunne, et al. 1986b). In these cases, relatively good predictions are evident for both Type 1 and 2 piezocones in a variety of clays that cover a wide range of OCRs from about 1 to over 60. Additional case studies are reported by Lunne et al. (1997) using this method.

Once the OCR profile has been established, the "average" value of mobilized undrained shear strength can be evaluated from the normalized undrained strength ratio corresponding to simple shear type loading conditions (Jamiolkowski, et al. 1985). The empirical expression derived from experimental data (e.g., Ladd, 1991) agrees well with constitutive soil models, such as critical-state soil mechanics (Kulhawy & Mayne, 1990) and is obtained from:

Normalized Undrained Strength:
$$\frac{s_u}{\sigma_{vo}'} = \tfrac{1}{2} \sin\phi' \cdot OCR^{0.75}$$

For foundation bearing capacity and embankment stability problems, the undrained strength of clays and drained strength of sands can be evaluated from piezocone data and used in conventional limit equilibrium methods or plasticity theorems for bearing capacity (e.g., Vesic, 1975) to determine the relevant factor of safety (FS) with respect to failure.

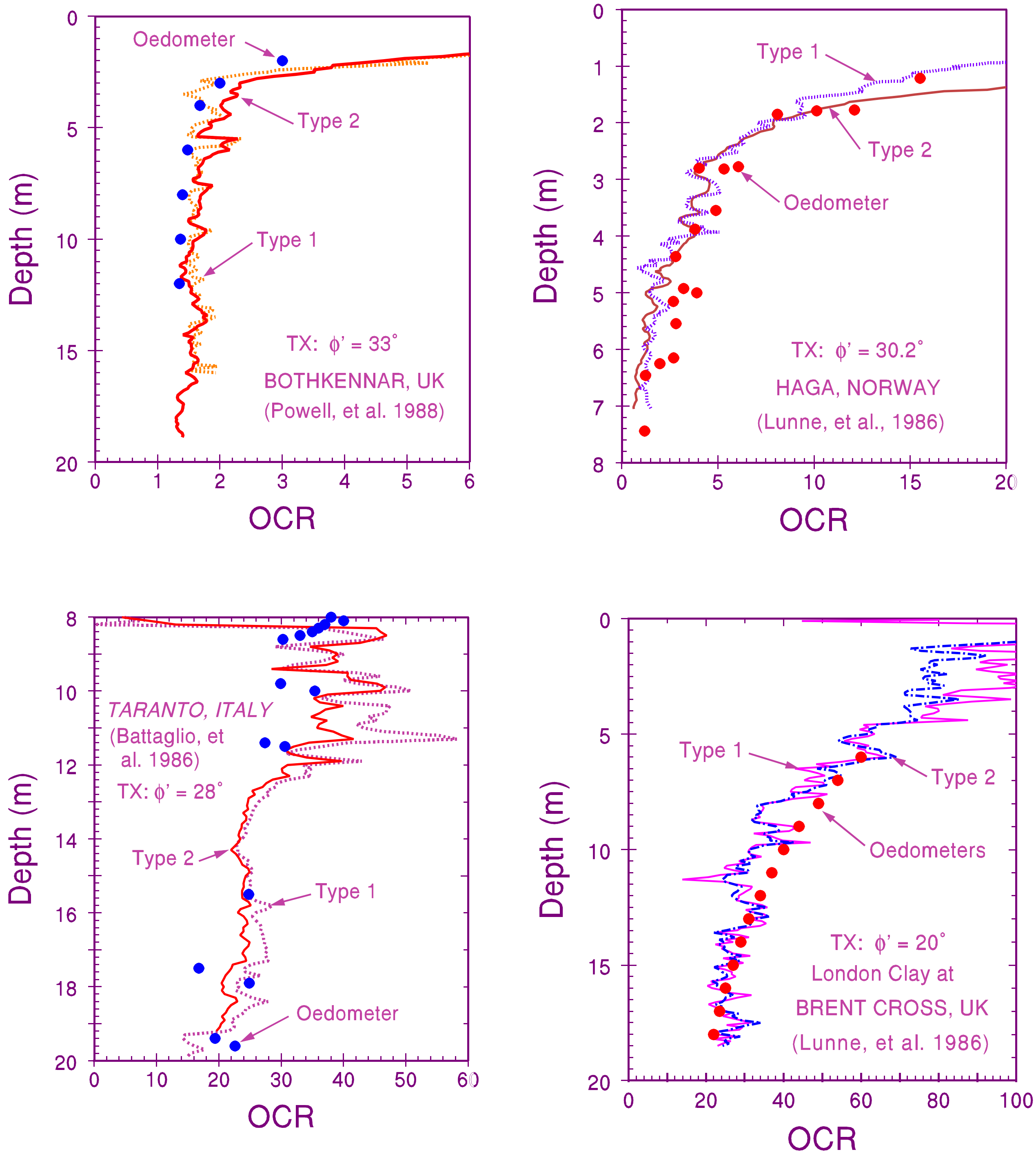

Figure 13. Measured and Piezocone-Estimated OCRs for Four Clay Sites.

AXIAL RESPONSE OF SHALLOW FOUNDATIONS

Using elastic continuum theory, the axial load-deformation response of shallow foundation is conveniently expressed by:

$$\textit{Footing Settlement:} \qquad \delta = \frac{q\ B\ I_{hr\nu}}{E_s}$$

where δ = vertical deflection (settlement), q = applied surface stress = Q/B^2, Q = axial force, B = equivalent square footing dimension, I = displacement influence factor from elastic theory and includes the consideration of finite depth to rock, foundation rigidity, and Poisson's ratio of the soil, and $E_s = 2G(1+\nu)$ = equivalent elastic soil modulus. Values of the displacement influence factor I are tabulated for a variety of conditions, such as given in Poulos & Davis (1974). For example, for the simple case of a flexible circular footing of diameter B resting on an elastic half-space (infinite depth to rock) with homogenous modulus E_s and $\nu = 0$, the factor I = 1.0. Approximate influence factors can be easily obtained from a spreadsheet solution (Mayne & Poulos, 1999).

To account for nonlinearity using an initial stiffness derived from shear wave velocity measurements, the modified hyperbola can be introduced, whereby:

$$\textit{Nonlinear Footing Settlement:} \quad \delta = \frac{Q \cdot I_{hr\nu}}{B \cdot E_0[1-(Q/Q_u)^{0.3}]}$$

Where $E_0 = 2G_0(1+\nu)$ = elastic modulus at small strains, $G_0 = G_{max} = \rho V_s^2$, and Q_u = ultimate bearing capacity of the footing or mat foundation. A reasonable value of $\nu = 0.2$ applies to all soils within the ranges of foundation settlement problems.

With this approach, a prediction is made for a shallow spread footing situated on sand at the Texas A&M national geotechnical experimentation site near College Station, Texas (Briaud & Gibbens, 1994), as shown in Figure 14. The sand is about 11 meters thick over clay shale and the groundwater lies about 5 meters deep at this site. Five different sizes of spread footings ranging from 1 to 3 meters square were constructed at the site and load tested using a reaction frame. The FHWA funded the research program, which included extensive series of in-situ testing (SPT, PCPT, PMT, DMT, CHT) and laboratory testing (triaxial, resonant column, index). For the prediction of the 3-meter square footing given in Figure 14, the results of cone penetration testing interpreted an effective $\phi' = 39^\circ$ (Mayne 1994) and crosshole tests determined the shear wave velocity profile to be constant with depth at $V_s = 210$ m/s. The ultimate axial bearing capacity of the footing was calculated using Vesic (1975) theory. The measured and nonlinear hyperbolic representation are seen to be in reasonable agreement.

A similar representation is presented in Figure 15 where the nonlinear modulus degradation approach was applied to large footing load tests (stacked Kentledge blocks) situated on clay at the Bothkennar test site in Scotland where a seismic piezocone test provided all the necessary parameters from one sounding. Two load tests were conducted with square side dimension, B = 2.2 and 2.4 meters (Jardine et al. 1995). Here, the cone parameters q_T and u_b can be used to determine the OCR, and thus the undrained shear strength (s_u) for input into the well-known bearing capacity equation: $q_{ult} = N_c\, s_u + \sigma_{vo}$, where the bearing factor $N_c = 6.14$ for a square footing. The downhole V_s measurements

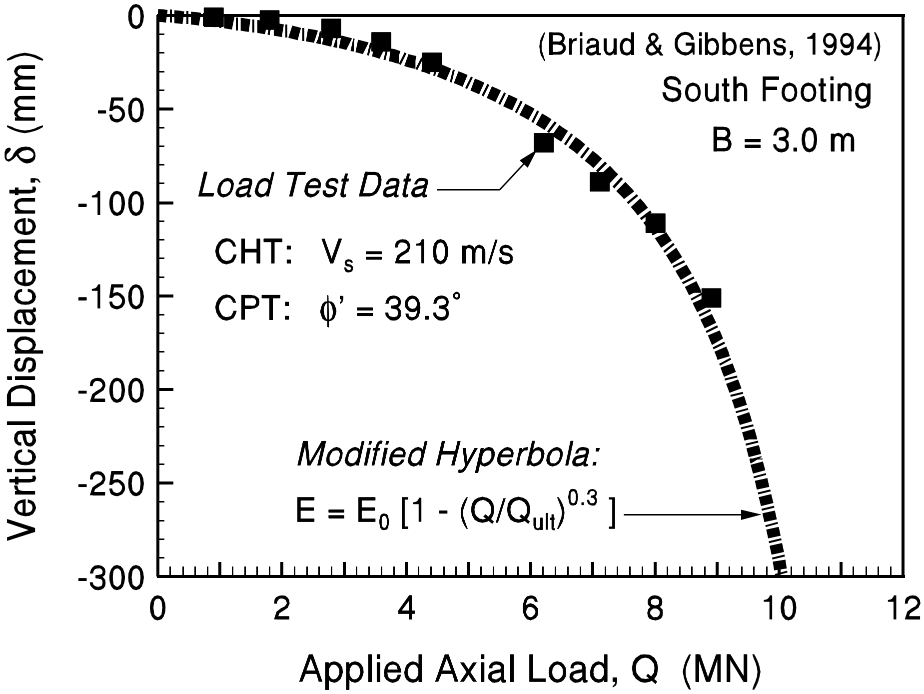

Figure 14. Measured and predicted response for large spread footing on sand at College Station, Texas (data from Briaud & Gibbens, 1994).

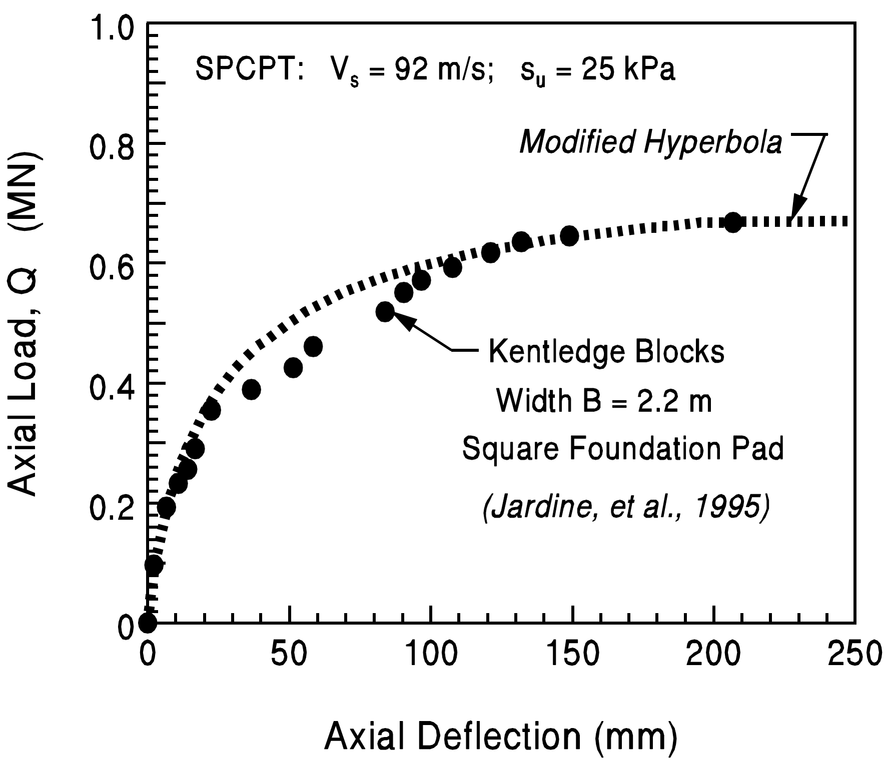

Figure 15. Measured and predicted footing load test on soft clay at Bothkennar, U.K. (data from Jardine, et al. 1995).

from the SCPT provided the initial stiffness (Nash, et al. 1992). Even though the concept of yield stress is neglected during the empirical nonlinear scheme, the modulus degradation procedure is in general agreement with the measured load-deflection response.

AXIAL RESPONSE OF DEEP FOUNDATIONS

The axial load-displacement response of driven piles and drilled shafts may be expressed in terms of elastic continuum theory. Solutions have been developed using boundary element formulations (Poulos and Davis, 1980; Poulos, 1989), finite elements (Jardine et al. 1986), and approximate closed-form solutions by Randolph and Wroth (1978, 1979). The generalized method characterizes the soil by two elastic parameters: soil modulus (E_s) and Poisson's ratio (ν_s). Soil modulus may be either uniform with depth (constant E_s) or a Gibson-profile (linearly increasing E_s with depth). The pile may either be a floating-type or end-bearing type where the tip is underlain by a stratum of stiffer material.

The elastic theory solution for the vertical displacement (δ) of a pile foundation subjected to axial compression loading is expressed by:

$$\textit{Axial Pile Settlement:} \qquad \delta = \frac{Q \cdot I_\rho}{E_{sL} \cdot d}$$

where Q = applied axial load at the top of the shaft, E_{sL} = soil modulus at the pile tip or foundation base, d = foundation diameter, and I_ρ = influence factor. Solutions for I_ρ depend on the pile slenderness ratio (L/d), pile modulus, and soil modulus (Randolph & Wroth, 1979, 1979; Poulos & Davis, 1980; Poulos, 1989). The modified form of the expression to account for nonlinear modulus degradation is:

$$\textit{Nonlinear Pile Settlement:} \qquad \delta = \frac{Q \cdot I_\rho}{E_{max} \cdot d \cdot [1-(Q/Q_u)^{0.3}]}$$

At the Georgia Tech site discussed earlier (see Fig. 6), results of an axial load test on a drilled shaft foundation having a diameter d = 0.76 m and embedded length L = 16.8 m constructed completely within the weathered Piedmont sands are shown in Figure 16. The reinforcing cage was instrumented with vibrating wire strain gages to permit load transfer measurements.

Analysis of the load-displacement curves by nine different criteria gave an interpreted average "failure" load of Q_{ult} = 3.11 MN (Harris & Mayne, 1994). This is close to the calculated capacity Q_{ult} = 2.97 MN using the CPT method presented by Fioravante et al. (1995) and capacity of Q_{ult} = 2.90 MN obtained using the CPT approach of Busamante and Gianeselli (1982). Adopting the soil-pile continuum model above and ν = 0.2, equivalent secant values of elastic moduli can be backcalculated from the load test data. Figure 16 also shows the predicted response using the modified hyperbola for E/E_{max} degradation curves versus mobilization factor (Q/Q_{ult}). The modified hyperbola (with assumed values: f = 1; g = 0.3) shows reasonable fit with the full scale data. From a more rigorous standpoint, the components of base and shaft resistance could be analyzed separately using the hyperbolic approach suggested by Fleming (1992).

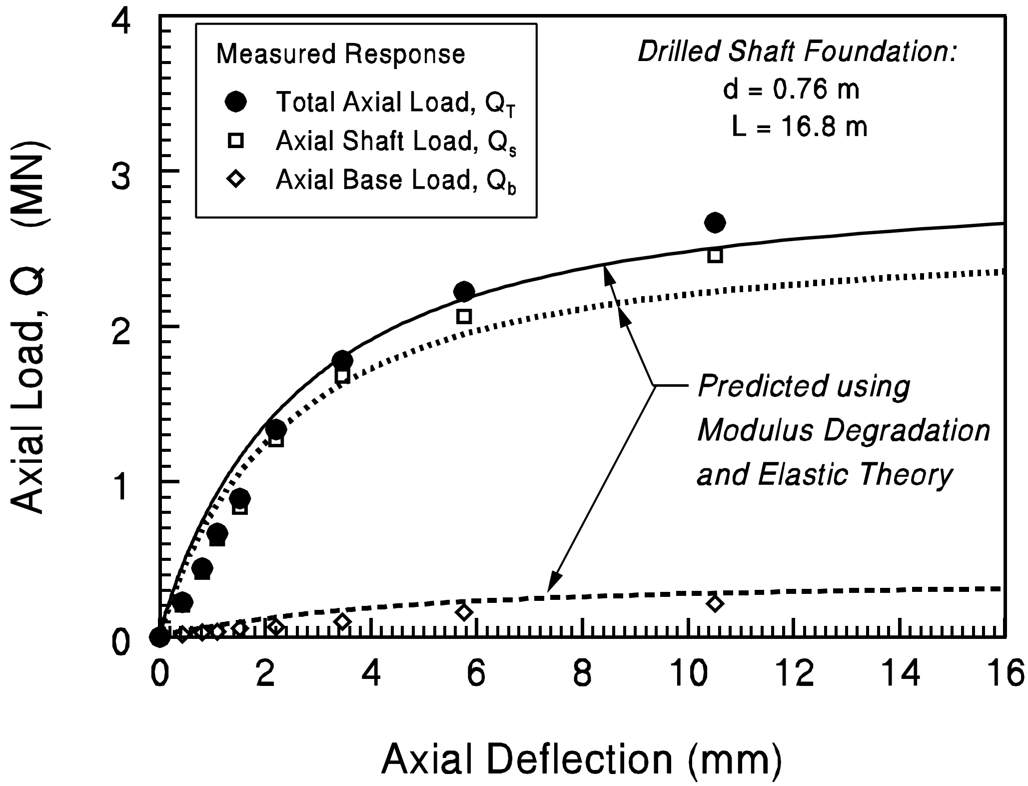

Figure 16. Measured and Predicted Axial Drilled Shaft Response in Piedmont Residual Silty Sand in Atlanta, Georgia (data from Harris & Mayne, 1994)

The total axial pile capacity (Q_t) can be separated into the side component (Q_s) and base component (Q_b) and evaluated from piezocone penetration test data using the procedures given by Eslami & Fellenius (1997) or by Takesue et al. (1998). The initial stiffness is obtained from elastic continuum theory based on the small-strain elastic modulus. The modulus degradation by modified hyperbola is shown in Figure 17 for a driven pipe pile in sensitive clay at Saint Alban, Quebec (Konrad & Roy, 1987). Here, a combination of the results of piezocone tests (Roy, et al. 1982) and SASW surveys (Lefebvre, et al. 1994) provided the necessary input for capacity and stiffness needed in the elastic continuum approach.

In each of the aforementioned cases, the degradation was accomplished using a modified hyperbola with assumed parameters (f = 1 and g = 0.3) to obtain an equivalent modulus for use in elastic continuum analyses. Initial stiffnesses were obtained from in-situ shear wave velocities and foundation bearing capacity was evaluated from the CPT resistance measurements that correspond to large-strain behavior. Herein, applications were presented for several case studies involving spread footings on both sand and clay, and for deep foundations situated in both silty sand and in clay. Considering the simplicity of the approach, consistent results were found in these particular cases. However, in structured soils, cemented geomaterials, and soft rocks, the empirical f and g parameters may be different than the above select values and thus should be evaluated accordingly (Fahey & Carter, 1993).

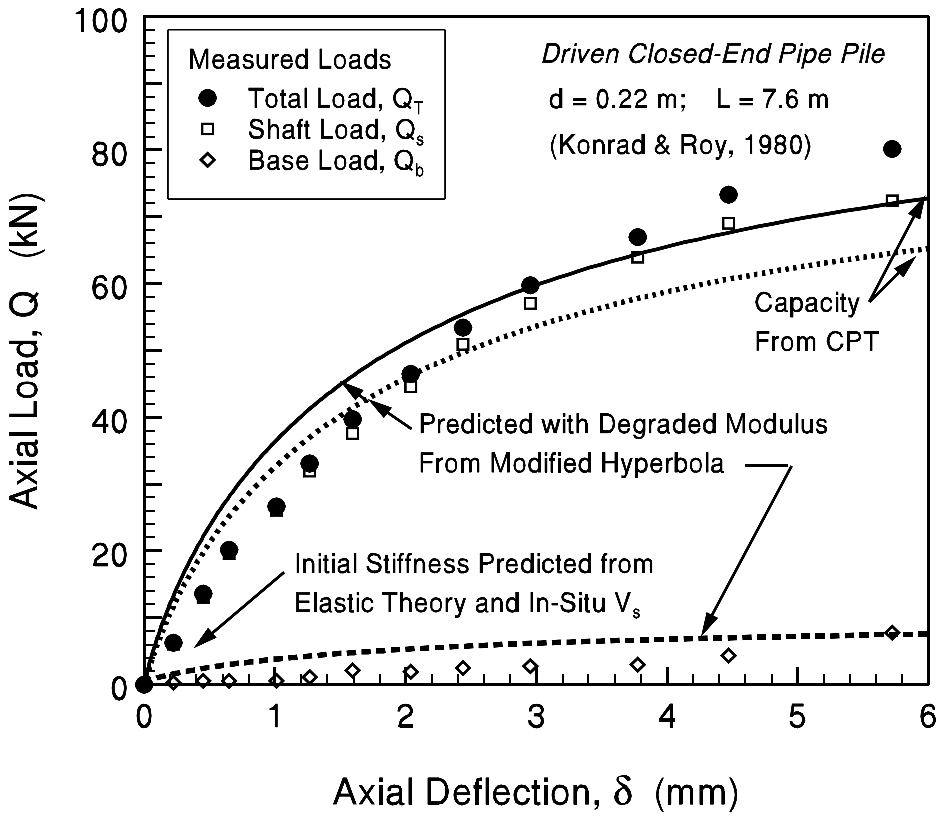

Figure 17. Measured and Predicted Driven Pile Response in Sensitive Marine Clay at Saint Alban, Quebec (data from Konrad & Roy, 1987).

RESEARCH NEEDS

In future efforts, fundamental research should be undertaken to better define the degradation curves for G/G_{max} behavior for a variety of different soils and rocks under monotonic and cyclic loading using resonant column, torsional shear, and triaxial testing. These results should be complemented by the development of in-situ testing capabilities for evaluating modulus degradation over a wide range of strains and in the natural ground. It appears feasible that the development of the seismic piezocone pressuremeter test (SPCPMT) could be easily implemented as a complete tool for evaluating the complete stress-strain-strength-time properties of soils. Although seemingly complex at first glance, in fact, the SPCPMT is simply a hybrid device that would combine the experiences and benefits of piezocone, full-displacement type pressuremeter, and downhole geophysics into a single sounding (Figure 18), thus capable of 10 direct and independent measurements (plus time-rate decays) at each test depth. With such a device, the modulus degradation response of site-specific geomaterials could be investigated with such a device, thus surplanting the need for an empirical scheme as described earlier. Notably, the modified hyperbola used herein applies only to uncemented and non-structured soils. The SPCPMT might use a true-interval velocity determination by two geophones at a set vertical distance apart, thus providing more accuracy and quicker assessments of V_s than the current pseudo-interval approach that relies on a single horizontal geophone.

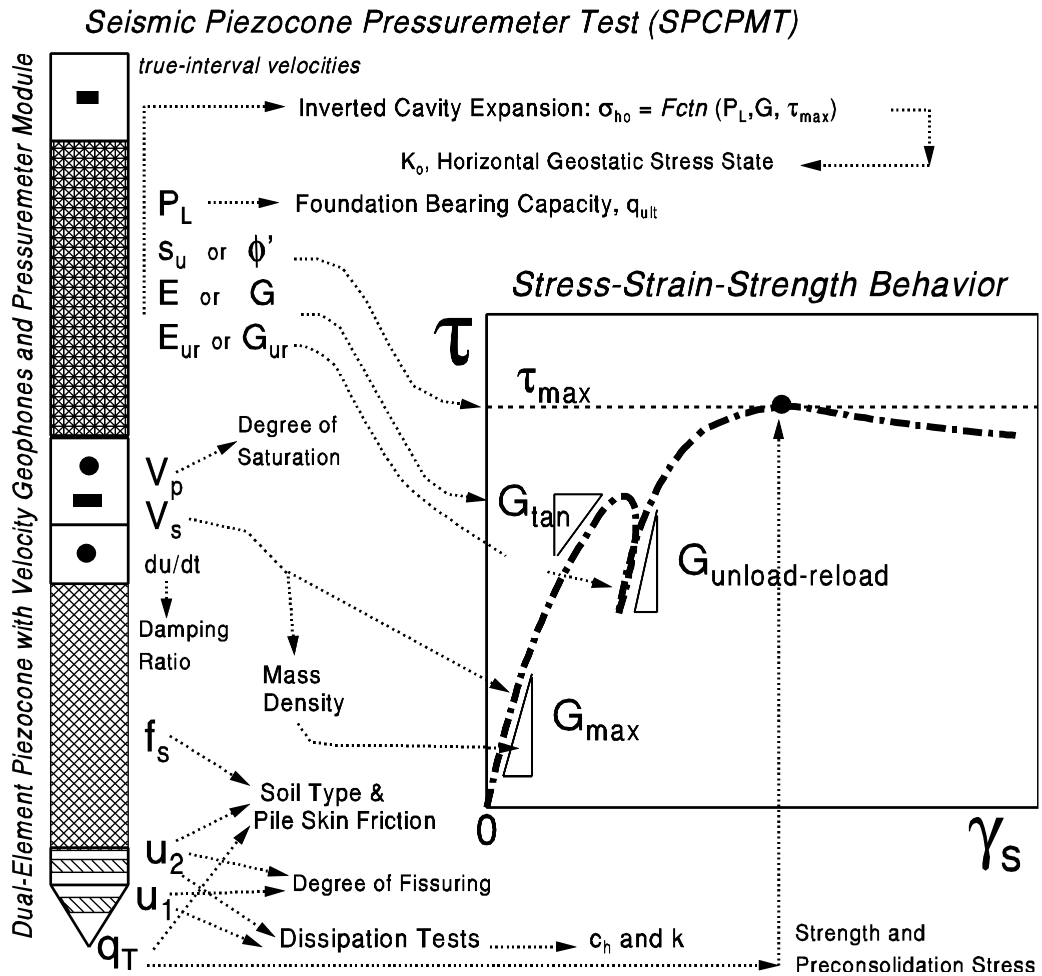

Figure 18. Future Hybrid In-Situ Test: The Seismic Piezocone Pressuremeter.

Another simple approach for deriving the full suite of stress-strain-strength response of soils is the incorporation of a velocity geophone within a flat dilatometer setup so that small-strain stiffness ($V_s \Rightarrow G_0 \Rightarrow E_0$), intermediate stiffness (DMT modulus, E_D), and large-strain failure states (i.e., p_o and p_1) are obtained within a single sounding. A simple version of the seismic dilatometer test (SDMT) has been presented by Martin & Mayne (1997). An extensive set of correlative relationships for interpreting the basic DMT pressure readings is given by Mayne & Martin (1998).

In future efforts, the mathematical representation of modulus degradation should be investigated (Tatsuoka and Shibuya, 1992) within a scientific framework. In this regard, a recent bounding surface formulation (Whittle and Kavvadas, 1994) provided a constitutive modeling of the G/G_{max} relationships for Boston Blue clay.

Finally, in the aforementioned backcalculation of E/E_{max} degradation, a constant value of $\nu = 0.2$ was adopted in the analyses. However, instrumented triaxial data by Jamiolkowski et al. (1994a) show that, for strains exceeding about 0.1%, Poisson's ratio increases to failure. Additional studies are needed for quantification of this parameter, as well as the development of field test methods for its measurement in-situ.

CONCLUSIONS

Field investigations for geotechnical site characterization can be enhanced by conducting a complementary set of borings, laboratory tests, and in-situ tests in order to better delineate the geotechnical input parameters required in analytical, numerical, and advanced computer analyses. Hybrid in-situ tests, such as the seismic piezocone, cone pressuremeter, and seismic flat dilatometer show great promise since they provide profiling capabilities for small- to high-strain properties within a single sounding. Applications to foundation analysis and design will benefit in terms of better performance predictions, higher reliability, and improved economies in construction. Additional research is needed in quantifying the nonlinear stress-strain-strength of soils, particularly those affecting the modulus degradation response, rate & time effects, and behavior of Poisson's ratio during loading.

ACKNOWLEDGMENTS: Funding for these research activities was provided by the National Science Foundation, U.S. Geological Survey, Mid-America Earthquake Center, and Federal Highway Administration.

REFERENCES

- Alarcon-Guzman, A., Chameau, J-L., Leonards, G.A., and Frost, J.D. (1989). Shear modulus and cyclic undrained behavior of sands. *Soils and Foundations* 29 (4), 105-119.
- Battaglio, M., Bruzzi, D., Jamiolkowski, M., and Lancellotta, R. (1986), "Interpretation of CPTs and CPTUs", *Proceedings*, 4th International Geotechnical Seminar, Field Instrumentation and In-Situ Measurements, Nanyang Technological University, Singapore, 129-143.
- Briaud, J-L. and Gibbens, R.M. (1994). *Predicted and Measured Behavior of Five Spread Footings on Sand* (Geotechnical Special Publication No. 41), ASCE, New York, 255 p.
- Burghignoli, A., Pane, V., and Cavalera, L. (1991). Modeling stress-strain-time behavior of natural soils: monotonic loading. *Proceedings*, 10th European Conf. on Soil Mechanics and Foundation Engineering, Vol. 3, Firenze, (Balkema, Rotterdam), 959-979.
- Burland, J.B. (1989). Small is beautiful: The stiffness of soils at small strains. *Canadian Geotechnical Journal*, Vol. 26 (4), 499-516.
- Burns, S.E. and Mayne, P.W. (1996). Small- and high-strain soil properties using the seismic piezocone. *Transportation Research Record* 1548, Washington, D.C., 81-88.
- Busamante, M. and Gianeselli, L. (1982). Pile bearing capacity by means of static CPT. *Proceedings*, Second European Symposium on Penetration Testing (2), Amsterdam, 493-506.
- Campanella, R.G. (1994). Field methods for dynamic geotechnical testing. *Dynamic Geotechnical Testing II* (STP 1213), ASTM, West Conshohocken, PA., 3-23.
- Campanella, R.G. and Robertson, P.K. (1988), "Current status of the piezocone test", *Penetration Testing 1988*, Vol. 1 (Proc. ISOPT-1, Orlando), Balkema, Rotterdam, 93-116.
- Diaz-Rodriguez, J.A., Leroueil, S., and Aleman, J. (1992). Yielding of Mexico City clay and other natural clays. *Journal of Geotechnical Engineering* 118 (7), 981-995.
- Duncan, J.M. and Chang, C.Y. (1970). Nonlinear analysis of stress and strain in soils. *Journal of the Soil Mechanics and Foundations Division*, ASCE, 96 (SM5), 1629-1653.
- Eslami, A. and Fellenius, B.H. (1997). Pile capacity by direct CPT and CPTu methods applied to 102 case histories. *Canadian Geotechnical Journal* 34 (6), 886-904.
- Fahey, M. and Carter, J.P. (1993). A finite element study of the pressuremeter test in sand using a nonlinear elastic plastic model. *Canadian Geotechnical Journal* 30 (2), 348-362.
- Fahey, M., Robertson, P.K, and Soliman, A.A. (1994). Towards a rational method of predicting settlements of spread footings on sand. *Vertical and Horizontal Deformations of Foundations and Embankments* (GSP 40), Vol. 1, ASCE, New York, 598-611.

- Fioravante, V. et al. (1995). Load carrying capacity of large diameter bored piles in sand and gravel. *Proceedings*, 10th Asian Regional Conf. on Soil Mechanics and Foundation Engineering.
- Fleming, W.G.K. (1992). A new method for single pile settlement prediction and analysis. *Geotechnique* 42 (3), 411-425.
- Hardin, B.O. and Black, W.L. (1968). Vibration modulus of normally consolidated clay. *Journal of the Soil Mechanics and Foundations Division* (ASCE) 94 (2), 353-369.
- Hardin, B.O. and Drnevich, V.P. (1972). Shear modulus and damping in soils. *Journal of the Soil Mechanics and Foundations Division*, ASCE, 98 (7), 667-692.
- Harris, D.E. and Mayne, P.W. (1994). Axial compression behavior of two drilled shafts in Piedmont residuum. *Proceedings*, International Conference on Design and Construction of Deep Foundations (Orlando), Vol. II, Federal Highway Administration, Washington, DC, 352-367.
- Jamiolkowski, M., Ladd, C.C., Germaine, J. and Lancellotta, R. (1985), "New developments in field and lab testing of soils", *Proceedings*, 11th International Conference on Soil Mechanics and Foundations Engineering, Vol. 1, San Francisco, 57-154.
- Jamiolkowski, M., Lancellotta, R., LoPresti, D.C.F., and Pallara, O. (1994a). Stiffness of Toyoura sand at small and intermediate strain. *Proceedings*, 13th International Conference on Soil Mechanics and Foundation Engineering (1), New Delhi, 169-172.
- Jamiolkowski, M., Lancellotta, R., and LoPresti, D.C.F. (1994b). Remarks on the stiffness at small strains of six Italian clays. *Proceedings*, Intl. Symposium on Pre-Failure Deformation Characteristics of Geomaterials, Hokkaido, Sapporo, Vol. 2, 817-836.
- Jardine, R.J., Potts, D.M., Fourie, A., and Burland, J.B. (1986). Studies on the influence of nonlinear stress-strain in soil-structure interaction. Geotechnique 36 (3), 377-396.
- Jardine, R.J., Potts, D.M., St.John, H.D., and Hight, D.W. (1991). Some practical applications of a nonlinear ground model. *Proceedings*, 10th European Conference on Soil Mechanics and Foundation Engrg (1), Firenze, 223-228.
- Jardine, R.J., Lehane, B.M., Smith, P.R., and Gildea, P.A. (1995). Vertical loading experiments on rigid pad foundations at Bothkennar. *Geotechnique* 45 (4), 573-597.
- Kondner, R.L. (1963). Hyperbolic stress-strain response: cohesive soils. *Journal of the Soil Mechanics and Foundations Division*, ASCE, 89 (1), 115-143.
- Konrad, J.M. and Roy, M. (1987). Bearing capacity of friction piles in marine clay. *Geotechnique* 37 (2), 163-176.
- Kulhawy, F.H. and Mayne, P.W. (1990). Manual on estimating soil properties for foundation design. *Report EL-6800.* Electric Power Research Institute, Palo Alto, 306 p.
- Ladd, C.C. (1991). Stability evaluation during staged construction. ASCE *Journal of Geotechnical Engineering* 117 (4), 540-615.
- Lefebvre, G., Leboeuf, D., et al. (1994). Laboratory & field determinations of small-strain shear modulus for a structured Champlain clay, *Canadian Geotechnical Journal*, Vol. 31 (1), 61-70.
- LoPresti, D.C.F., Pallara, O., Lancellotta, R., Armandi, M., and Maniscalco, R. (1993). Monotonic and cyclic loading behavior of two sands at small strains. ASTM *Geotechnical Testing Journal* 16 (4), 409-424.
- LoPresti, D.C.F., Pallara, O., and Puci, I. (1995). A modified commercial triaxial testing system for small-strain measurements. ASTM *Geotechnical Testing Journal*, Vol. 18 (1), 15-31.
- Lunne, T., Eidsmoen, T., Gillespie, D. and Howland, J.D. (1986a), "Laboratory & field evaluation of cone penetrometers", *Use of In Situ Tests in Geotechnical Engrg.* (GSP 6), ASCE, New York, 714-729.
- Lunne, T., Eidsmoen, T., Powell, J. and Quarterman, R. (1986b). "Piezocone testing in overconsolidated clays", *Proceedings*, 39th Canadian Geotechnical Conf., Ottawa, 209-218.
- Lunne, T., Lacasse, S., and Rad, N. (1992). General report: SPT, CPT, PMT, and recent developments in in-situ testing. *Proceedings*, 12th Intl. Conf. on Soil Mechanics & Foundation Engrg., Vol. 4, Rio de Janeiro, 2339-2403.
- Lunne, T., Robertson, P.K., and Powell, J.J.M. (1997). *Cone Penetration Testing in Geotechnical Practice*. Blackie Academic/Chapman-Hall Publishers, U.K., 312 pages.
- Martin, G.K. and Mayne, P.W. (1997). Seismic flat dilatometer tests in Connecticut Valley varved clay. ASTM *Geotechnical Testing Journal*, Vol. 20 (3), 357-361.
- Mayne, P.W. and Dumas, C. (1997). Enhanced in-situ geotechnical testing for bridge foundation analysis. *Transportation Research Record* 1569, National Academy Press, Wash, D.C., 26-35.
- Mayne, P.W. and Martin, G.K. (1998). Commentary on Marchetti flat dilatometer correlations in soils. ASTM *Geotechnical Testing Journal*, Vol. 21 (3), 222-239.
- Mayne, P.W., Kulhawy, F.H., and Kay, J.N. (1990), "Observations on the development of pore water stresses during piezocone testing in clays", *Canadian Geotechnical J.*, Vol. 27 (4), 418-428.
- Mayne, P.W. (1991), "Determination of OCR in clays by piezocone tests using cavity expansion and critical state concepts", *Soils and Foundations*, Vol. 31 (2), 65-76.

- Mayne, P.W. (1993), "In-situ determination of clay stress history by piezocone model", *Predictive Soil Mechanics*, Thomas Telford, London, 483-495.
- Mayne, P.W. and Chen, B.S-Y. (1994), "Preliminary calibration of PCPT-OCR model for clays", *Proceedings*, 13th International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, New Delhi, India, 283-286.
- Mayne, P.W. (1994). CPT-based prediction of footing response. *Predicted and Measured Behavior of Five Spread Footings on Sand* (GSP 41), ASCE, Reston, Virginia, 214-218.
- Mayne, P.W. (1995). Application of G/G_{max} modulus degradation to foundation settlement analyses. *Proceedings*, U.S.-Taiwan Workshop on Geotechnical Collaboration, National Science Foundation/Washington D.C. and National Science Council/Taipei, 136-148.
- Mayne, P.W., Mitchell, J.K., Auxt, J., and Yilmaz, R. (1995). U.S. national report on the CPT. *Proceedings, Symposium on the Cone Penetration Test*, Vol. 1, Swedish Geotechnical Society, Linköping, 263-276.
- Nash, D.F.T., Powell, J.J.M., and Lloyd, I.M. (1992). Initial investigations of the soft clay site at Bothkennar. *Geotechnique*, Vol. 42 (2), 163-182.
- Poulos, H.G. (1989). Pile behavior: theory and application", 29th Rankine Lecture, *Geotechnique,* Vol. 39, No. 3, 363-416.
- Poulos, H.G. and Davis, E.H. (1974). *Elastic Solutions for Soil and Rock Mechanics*, Wiley & Sons, New York, 411 p. (available from University of Sydney Press, 1994).
- Poulos, H.G. and Davis, E.H. (1980), *Pile Foundation Analysis and Design*, John Wiley & Sons, New York, 397 p. (published in 1991 by Krieger Publishing, Malabar, Florida).
- Powell, J.J.M, Quarterman, R.S.T., and Lunne, T. (1988). Interpretation and use of the piezocone in UK clays, *Penetration Testing in the UK*, Thomas Telford, London, 151-156.
- Prevost, J.H. and Keane, C.M. (1990). Shear stress-strain curve generation from simple material parameters. *Journal of Geotechnical Engineering* 116 (8), 1255-1263.
- Puzrin, A.M. and Burland, J.B. (1996). A logarithmic stress-strain function for rocks and soils. *Geotechnique* 46 (1), 157-164.
- Puzrin, A.M. and Burland, J.B. (1998). Nonlinear model of small-strain behavior of soils. *Geotechnique* 48 (2), 217-233.
- Randolph, M.F. and Wroth, C.P. (1978). Analysis of deformation of vertically loaded piles. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 104, No. GT12, 1465-1488.
- Randolph, M.F. and Wroth, C.P. (1979). A simple approach to pile design and the evaluation of pile tests. *Behavior of Deep Foundations* (STP 670), ASTM, West Conshohocken, PA, 484-499.
- Robertson, P.K. (1986). In-situ testing and its application to foundation engineering. *Canadian Geotechnical Journal*, Vol. 23 (4), 573-594.
- Roy, M., Tremblay, M., Tavenas, F. and LaRochelle, P. (1982). Development of pore pressures in quasi-static penetration tests in sensitive clay. *Canadian Geotechnical Journal*, Vol. 19 (1), 124-138.
- Shibuya, S., Tatsuoka, F., Teachavorasinskun, S., Kong, X., Abe, F. (1992). Elastic deformation properties of geomaterials. *Soils and Foundations* 32 (3), 26-46.
- Skomedal, E. and Bayne, J.M. (1988), "Interpretation of pore pressure measurements from advanced cone penetration testing", *Penetration Testing in the U.K.*, Thomas Telford, London, 279-283.
- Takesue, K., Sasao, H., & Matsumoto, T. (1998). Correlation between ultimate pile skin friction and CPT data. *Geotechnical Site Characterization*, Vol. 2, Balkema, Rotterdam, 1177-1182.
- Tatsuoka, F. and Shibuya, S. (1992). Deformation characteristics of soils and rocks from field and laboratory tests. *Rept. of the Inst. of Industrial Science*, Vol. 37, No. 1, Univ. of Tokyo, 136 p.
- Tatsuoka, F., Jardine, R.J., LoPresti, D.C.F., DiBenedetto, H., and Kodaka, T. (1997). Theme Lecture: Characterizing the pre-failure deformation properties of geomaterials. *Proceeedings*, 14th International Conf. on Soil Mechanics & Foundation Engineering, Vol. 4, Hamburg, 35 p
- Teachavorasinskun, S., Shibuya, S. and Tatsuoka, F. (1991). Stiffness of sands in monotonic & cyclic torsional simple shear. *Geotechnical Engineering Congress*, Vol. II (Boulder), GSP No. 27, ASCE, New York, 863-878.
- Vesić, A.S. (1975). Bearing capacity of shallow foundations. *Foundation Engineering Handbook*. Ed. Winterkorn & Fang, Van Nostrand Reinhold Co., New York, 121-147.
- Vucetic, M. and Dobry, R. (1991). Effect of soil plasticity on cyclic response. *Journal of Geotechnical Engineering* 117 (1), 89-107.
- Whittle, A.J. and Kavvadas, M.J. (1994). Formulation of MIT-E3 constitutive model for overconsolidated clay. *Journal of Geotechnical Engineering* 120 (1), 173-224.
- Woods, R.G. (1994). Laboratory measurement of dynamic soil properties. *Dynamic Geotechnical Testing II* (STP 1213), ASTM, West Conshohocken, PA., 165-190.
- Wroth, C.P. (1984). The interpretation of in-situ soil tests. (The 24th Rankine Lecture). *Geotechnique*, Vol. 34 (4), 449-489.