Roger A. Failmezger, P.E., F. ASCE, In-Situ Soil Testing, L.C., Lancaster, Virginia, email: insitusoil@prodigy.net, and
Paul J. Bullock, Ph.D., P.E., M.ASCE, GRL Engineers, Inc., Gainesville, Florida, email: pjbullock@bellsouth.net

Which in-situ test should I use?—A designer's guide

Abstract: In-situ tests can greatly increase the volume of geomaterial investigated at a foundation site, with savings in both cost and speed when compared to sampling and lab testing. Historically, they have been developed to evaluate specific parameters for geotechnical design. Some tests directly measure the response to a particular type of load, such as a plate load test or a pile load test. These tests verify design assumptions, and possibly determine soil or rock properties by inversion. The two most common in-situ tests, the Standard Penetration Test and the Cone Penetrometer Test, primarily identify soil type and stratigraphy, along with a relative measure of strength. Interpretation of these two tests may also utilize indirect correlations with specific soil properties, but typically with high statistical variability (partly due to inherent testing variability, partly due to ignoring the soil's stress history test, and partly due to crude empiricism). Other tests, such as the Iowa Borehole Shear Test, the Dilatometer Test, and the Pressuremeter Test, attempt to directly measure in-situ the soil properties that might be otherwise determined from laboratory tests of "undisturbed" (more accurately termed "intact") samples. Stress-path variations, disturbance effects due to insertion of the test device, and alternative test procedures may affect the results of these tests. The research literature contains numerous correlations between in-situ test results and various geotechnical parameters. To use these correlations with reliability, the engineer must understand their basis and potential for error, and then choose the in-situ test(s) that provide the most reliable correlation(s) for the desired soil properties and design parameters. In general, this requires a test that closely models the intended design use or directly measures the soil properties required for design. This paper examines the commonly available in-situ tests and provides testing recommendations for specific geotechnical design applications.

Introduction

In-situ tests generally investigate a much greater volume of soil more quickly than possible for sampling and laboratory tests, and therefore they have the potential to realize both cost savings and increased statistical reliability for foundation design. Many well-written technical papers and manuals have previously discussed and compared the various in-situ tests, e.g. Schmertmann (1975) and Kulhawy and Mayne (1990). This paper presents an overview of the available in-situ tests and points out some important details often overlooked by practicing engineers. After discussing different geotechnical foundation design needs, it provides recommendations to help the engineer choose the most appropriate in-situ test to satisfy the design requirements of specific types of foundations.

Available In-Situ Tests

The following sections briefly discuss the basic details of the available in-situ tests, and some important, yet sometimes unrecognized, details. Additional information can be found in technical papers shown in the references and the Standard Test Methods available from ASTM International.

Standard Penetration Test (SPT), ASTM D 1586, D 4633, and D 6066: While the standard penetration test is probably the most common in-situ test performed in North and South America, the term "standard" is misleading. Although the test is relatively simple to perform, only skilled drillers routinely achieve meaningful results. In 1902, C.R. Gow designed a 1-inch diameter heavy-wall sampler to be driven with a 110 pound weight. In 1927, L. Hart and G.A Fletcher developed the standard 2-inch-diameter "split-spoon" sampler (Figure 1). Later, Fletcher and H. A. Mohr standardized the test using a 140-pound hammer with a 30-inch drop to measure the blow count for three consecutive 6-inch increments of penetration, reporting the total blow count for final 12 inches as the N_{SPT} value. Terzaghi and Peck (1948) published early geotechnical design correlations, which popularized the SPT and encouraged its acceptance as a "standard".

Figure 1: Split spoon SPT sampler

The three styles of SPT hammer in common use (see Figure 2) deliver energy to the drill rods that varies from about 35 % to 95% of the theoretically available driving energy of 4200 in-lbs. This variation, plus the use of non-standardized drilling techniques, led Schmertmann (1978) to investigate their effect on the value of N_{SPT}, which he found to exceed a factor of two. In addition, Schmertmann (1979) also found that N_{SPT} varied approximately inversely in proportion to the hammer energy delivered to the drill rods. With the advent of modern computers, energy measurement devices allow technicians to easily measure the actual driving energy entering the rods as described in ASTM D4633. The engineer can then correct the

Figure 2: a) Automatic Hammer ~95% eff.,
b) Safety Hammer ~60% eff.,
c) Donut Hammer ~35% eff.
(photo from GeoServices Corp.)

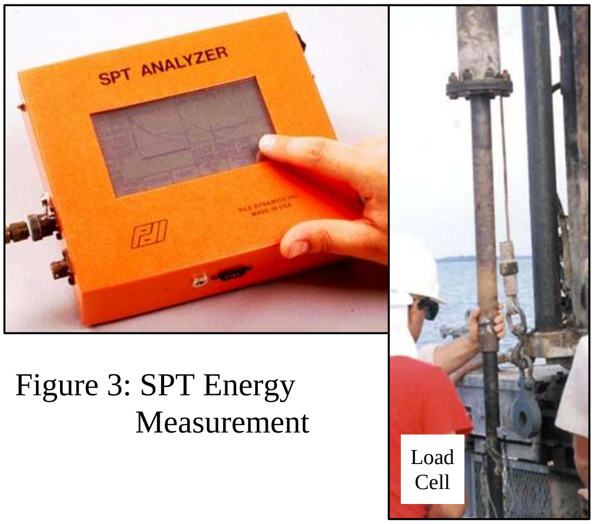

Figure 3: SPT Energy Measurement

measured value of N_{SPT} to N_{60}, the equivalent blow count at 60% of the theoretical hammer energy (thought to represent the average energy in the correlation database). Skempton (1986) presented a method to compute N_{60} values from raw N_{SPT} data, which is incorporated in ASTM D 6066.

Unfortunately, N_{60} values rarely appear on boring logs. The barrel on the old samplers had the same inner diameter as the shoe. Today, an alternative sampler barrel in common use has a larger inside diameter to accommodate liners with an inner diameter the same as the shoe. However, liners are rarely used - Skempton suggests multiplying the N-value by 1.2 for this correction. Automatic trip hammers, now in widespread use, may deliver almost 95% of the theoretical energy if well-maintained. For these hammers, a correction of 1.58 may be needed to get N_{60}. Without making the N_{60} correction, designs tend to be overly conservative and costly. Even with the best techniques, predicting how the soil responds to static structural loading based on the results of a dynamic test can be highly inaccurate.

Dilatometer Test (DMT), ASTM D 6635: In 1975, Dr. Silvano Marchetti invented the Flat Dilatometer, consisting of sharpened blade with a circular membrane located on one side, to investigate H-pile behavior for lateral loads. He performed tests at ten well-documented research sites and developed empirical correlations with classical soil properties. In 1980, he published a classic paper presenting those correlations; most of which are routinely used today. In 1981, Marchetti traveled to the United States on sabbatical and worked with Drs. John Schmertmann and David Crapps. While they were initially skeptical of Dr. Marchetti's invention, they were convinced by the impressive speed and accuracy of the results.

Figure 4: DMT Blade

Figure 4 shows a photograph of the stainless steel Dilatometer blade under a direct push rig. The blade, 15 mm thick and 96 mm wide in cross-section, is pushed into the soil at a constant rate of 2 cm/sec, preferably using a load cell to measure the penetration thrust as shown in Figure 5. Generally the operator stops penetration at 20 cm depth intervals, records the thrust at the test depth using a load cell, and then inflates the membrane.

The surrounding soil usually collapses the 60-mm-diameter stainless steel membrane flush against the blade during the penetration. (In very weak soils, a vacuum must be applied prior to pushing.) Electrical conductivity between the center of the membrane and the underlying body of the blade completes a circuit that activates a buzzer and a light on the dilatometer control unit. To run the test, the operator slowly inflates the membrane with nitrogen gas supplied from the control unit. When the membrane center moves away from the blade, the electrical continuity is lost and the light and buzzer go off. At that instant the operator reads the gas pressure at the control unit and records the membrane lift-off pressure as the "A-pressure" on the data sheet. The operator then continues to inflate the membrane. When the membrane has inflated an additional 1.1 mm at its center, an electrical switch inside the blade reestablishes the electrical circuit and reactivates the buzzer and light, prompting the operator to record the corresponding gas pressure as the "B-pressure". When below the water table, the operator can slowly deflate the membrane, and record the water pressure that pushes the membrane back in contact with the blade as the "C-pressure". Nearly all of the correlations are based on the thrust, "A-pressure" and "B-pressure". The "C-pressure" can be used to determine the groundwater table in clean sands and to determine the undrained shear strengths of soft clay (Lutenegger, 2006).

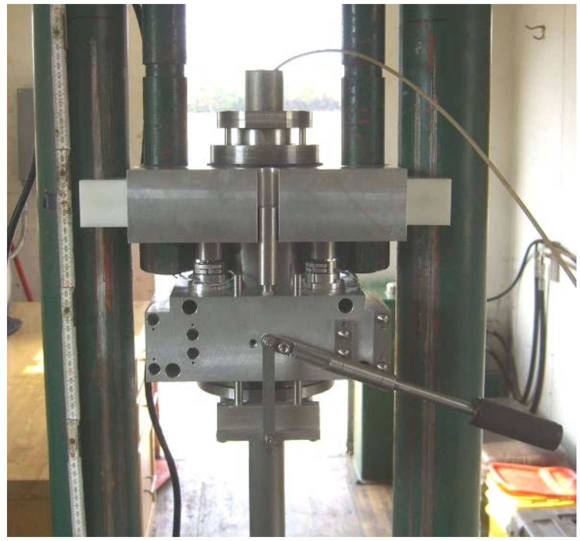
Figure 5: Push Clamp using Four Load Cells to Measure Thrust

The dilatometer blade has a cross-sectional area of about 14 cm^2 and can be pushed with a direct push rig into soil with an N_{60}-value of about 45 blows per foot or with a heavy drill rig into soil with an N_{60}-value of about 35 blows per foot. Tests can be successfully performed in all penetrable soils, including clay, silt, and sand. If the soil contains a significant amount of gravel, there may be point contacts against the membrane instead of a continuous medium, causing inaccurate results. Furthermore, the gravel will often tear a hole in the membrane.

DMT results have been correlated with the parameters that geotechnical engineers need the most -- soil shear strength and deformation properties. The computer program for the dilatometer data reduction evaluates and outputs the following soil properties and parameters:

- Tangent vertical constrained modulus [M],
- Undrained shear strength for clays [c_u],
- Drained friction angle for sands [ϕ'],
- Total unit weight of soil [γ_t],
- Coefficient of lateral earth pressure at rest [K_o],

- Preconsolidation pressure [p_c], and
- Overconsolidation ratio [OCR].

Cone Penetrometer Test (CPT), ASTM D 3441 and D 5778: The mechanical cone penetrometer probe, invented in The Netherlands in 1932 by P. Barentsen, measures the quasi-static thrust required to push a solid, conical tip having a 60 degree apex angle and a cross-sectional area of 10 cm^2 into the foundation soil. The operator advances the cone using a nested, dual-rod system, the outer rods providing strength to penetrate the cone in a collapsed configuration, and the inner rods allowing him or her to advance only the cone tip at each test depth (generally at 20-cm intervals) while measuring the hydraulic thrust pressure at the top of the rods. In 1953, Begemann modified the probe to include a friction sleeve just behind the tip. For the friction cone test, the inner rods initially advance only the tip for a short distance, and then engage both the tip and a friction sleeve together. The center of the friction sleeve is located 20 cm above the tip, and the value of unit soil adhesion acting on it is computed by subtracting the tip-only thrust force (from the previous test depth) and dividing by the sleeve area of 150 cm^2. The engineer then divides the unit tip bearing from the previous test depth by the unit adhesion to determine the friction ratio (both readings then apply to the same depth), and uses an empirical chart to identify the type of soil. Depth plots of unit bearing and friction ratio also provide a relative profile of the site stratigraphy.

Figure 6: CPT and Data Acquisition Computer

The improvement of electronics and computers in the 1980s led to the development of stainless steel electrical cone penetrometer probes that obtain and record more reliable test measurements and eliminate the dual-rod system (Figure 6). Strain gauges are used to measure the tip and friction values and a pressure transducer measures the pore water pressures generated during penetration. With the electric cone, data are collected at penetration increments of 0.5 cm to 5 cm depending on the computer acquisition system, such as the one shown in Figure 6. Engineers prefer the electronic cone's accuracy and productivity, relegating the mechanical cone to profiles containing strong materials that might damage the more expensive electrical cone. The cone penetrometer can be pushed with a direct push rig into soil with an N_{60}-value of about 50 blows per foot or with a heavy drill rig into soil with an N_{60}-value of about 40 blows per foot.

Engineers have obtained reasonable accuracy in correlations between the CPT unit bearing and soil strength parameters, such as friction angle and undrained cohesion (see

Lunne, et al., 1997). More indirect correlations with at rest coefficient of lateral earth pressure, modulus, and overconsolidation ratio are much less reliable due to the significant effects of stress history and the in-situ state of stress. The addition of pore pressure measurements, generally made just behind the tip, to the electrical cone (CPTU) improves stratigraphy profiling and various indirect correlations. By collecting data at close depth intervals, thin layers are detected. Two correlation charts are used to identify the soil type: Friction Ratio (R_f) vs. Corrected Cone Bearing (q_T) and Pore Pressure Ratio (B_q) vs. Corrected Cone Bearing (q_T). Generally, the pore pressure ratio correlation chart is more sensitive to thinner layers, while the friction ratio chart is better for cohesionless soils. When there is a discrepancy in soil type between the two charts, either pore pressure dissipation tests or sampling can be used to identify the correct soil type.

Pressuremeter Test (PMT), ASTM D 4719: Louis Menard began his work with the pressuremeter test in 1954 while still a college student, studying first under Professor Kerisel in France, and later under Professor Ralph Peck at the University of Illinois. Menard improved and advanced a foundation test concept begun by Kogler in 1933, and then returned to France in 1957 where he started a company to build and use the PMT. He compiled a large data base of load tests and companion pressuremeter tests to refine his empirical design formulas and persuade other engineers to use the PMT. To show his confidence and encourage acceptance of the test, Menard guaranteed foundation designs based on the PMT with $10,000,000 of professional liability insurance from Lloyds of London (Hartmann, 2008).

Figure 7: Texam PMT

The PMT is typically performed by inserting a cylindrical probe into an open borehole, supporting it at the test depth, and then inflating a flexible membrane in the lateral direction to a radial strain of as much as 40% depending on the probe design. The PMT operator may expand the pressuremeter probe in equal pressure increments (stress controlled test) or in equal volume increments (strain controlled test), typically stopping the test when initial volume of the probe has doubled or when reaching the maximum allowable pressure. About 40 data points are obtained from a strain controlled test versus and about 10 data points from a stress controlled test, thus a better defined curve can be obtained from strain controlled tests. Creep tests can be performed near the yield point of the test to evaluate time effects of the modulus. Ideally the PMT provides an axisymmetric, plane strain test (the horizontal plane), typically drained in sands and silts, and undrained in cohesive soils. Early PMT probes employed guard cells at their top and bottom to force the measurement cell located between them to expand only in the lateral direction. Briaud (1992) showed that the error in test results did not exceed 5% for single-cell probes (Texam in Figure 7) with a length at least six times its diameter.

Researchers have also used self-boring and push-in probes with some success in specific types of soils. Probes may also be designed with very stiff membranes for testing at high pressures and lower strain in soft rock.

The PMT results include the at-rest horizontal earth pressure, the pressuremeter elastic modulus, the reload modulus, and the pressuremeter limit pressure (plastic failure), but generally require an empirical approach for foundation design or for correlation with classic geotechnical parameters such as the shear strength or Young's modulus. While the PMT stress path can be modeled theoretically, the effects of stress history and anisotropy, testing in the direction of the minor principal stress (usually) in a material with behavior controlled by confining stress, and the disturbance of stress release and softening at the borehole wall (or stress increase for push-in probes), usually lead to an empirical approach. Good test results begin with a high quality borehole having minimal disturbance to its side walls, typically requiring mud wash rotary techniques. Maintaining the drilling mud level at or near the top of the borehole minimizes the horizontal stress release from drilling. During drilling, the operator should carefully monitor the rotation rate, advance rate, and mud flow rate to obtain a high quality borehole.

Modern data acquisition systems speed field testing and computer programs relieve the drudgery of data analysis, but the PMT remains one of the most labor-intensive and time-consuming in-situ tests. Pressuremeter tests are particularly valuable in dense sands, hard clays and weathered rock, if the DMT and CPT cannot penetrate those formations. Pressuremeter tests can also be used in remote sites that only skid rigs can access.

Iowa Borehole Shear Test (BST): While the shear strength of soils can be critical for the design of earth slopes, the calculation of earth pressure against retaining walls, and the determination of foundation bearing capacity, it can be a time consuming and expensive to measure with laboratory shear tests. The BST (Figure 8), developed by Dr. R.L. Handy at Iowa State University, provides a convenient method to accurately measure the drained shear strength of soils in-situ. Tests typically require between 30 and 60 minutes to perform, and the results are immediately available. It is similar to a laboratory direct shear test with the sides of the borehole being sheared.

Figure 8: BST shear head (left) and shear dynamometer (right)

To perform the BST, the operator inserts the shear head into borehole into a 3-inch diameter borehole to the chosen test depth. A normal stress is then applied to push apart two serrated stainless steel plates (total area 10 in^2), pressing them laterally against the sidewalls of a borehole. After allowing the soil to consolidate at the applied normal stress, usually between 5 minutes for cohesionless soil and about 10 to 20 minutes for cohesive soil, the operator pulls the shear head upward to measure the shear strength of the soil in contact with the plates. This shear test is typically repeated four to five times at progressively higher normal stresses to prepare a plot of normal stress versus shear strength. In sands, silts, and stiff clays, the BST provides a drained test, while results for softer cohesive soils may be partially drained. An available pore pressure sensor located in the shear head can provide an indication of drainage. Because the same soil is tested, the data can usually be fitted linearly with a coefficient of correlation of 0.99 or better.

For soils with an N_{60} value of 15 or more blows per foot, the smaller set of plates (total area 1.6 in^2) should be used to ensure that the plates are fully embedded into the soil. Because the pressure gauges are calibrated to measure the stress of the larger (standard size) plates, for the smaller plates the recorded pressures must be multiplied by 6.25 to account for the differences in the plate areas.

An oversize borehole can adversely affect the accuracy of the test results, as can loosening or softening of the borehole sidewalls. A borehole prepared with a76-mm (3 inch) diameter Shelby tube usually tends to minimize disturbance. Hand augers are also a good choice for more remote locations. Boreholes prepared using mud-rotary drilling methods will reduce the shear strength until the normal stress causes the shear heads to penetrate through any mud-caking.

Research is being performed to evaluate the residual shear strength in over-consolidated clays. After measuring the peak shear strength value, the BST plates are collapsed and lowered back to the starting depth for the data point. A normal stress equal to about 90% of the peak normal stress is then reapplied to the clay and the plates are pulled upward to the ending depth of the peak value. The resulting shear stress is recorded. This procedure is repeated until the shear stress becomes a constant value. An example set of residual borehole shear test data is shown as Figure 9.

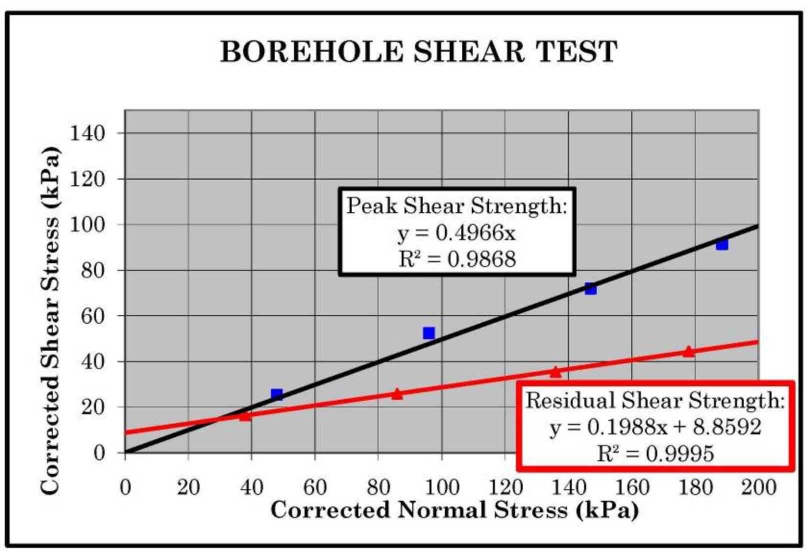

Figure 9: Peak and Residual BST

Dr. Handy also has developed a rock borehole shear test (RBST) device to measure the shear strength properties of rock. The device is quite robust and can apply a normal stress of 80 MPa and a shear stress of 50 MPa. The device is placed inside of a cored borehole, and the test is conducted using hydraulic pressure to apply the normal stress and to pull the plates for the shear stress. A shale or siltstone is likely to be smeared during the test and, after each data point, the plates are rotated axially by 45^0 for the next normal stress, obtaining a maximum of four data sets. With granite, the rock is likely to chip during each shearing. The rock shear device will probably need to be removed from the borehole and the rock chips cleaned from the device. The device should then be lowered to about 5 mm above the previous shear depth for the next test data set.

Ko Step Blade (KSB): While engineers can estimate the vertical stress of soil relatively well, they cannot estimate the horizontal stress. The coefficient of horizontal stress, K, ranges from 0.2 to 6 times the vertical stress (Schmertmann, 1985). When a vertical force is applied to the soil, it is resisted by the soil in three dimensions, two of which are horizontal, emphasizing the importance of the horizontal stress.

Unfortunately, horizontal stresses are difficult to measure. When we drill a hole, we remove them. When we push a device into the soil, we tend to increase them in looser soils and may decrease them in denser soils. Soil sampling causes too much disturbance for the engineer to measure horizontal stresses with laboratory tests.

The K_o step blade was invented to measure this difficult to obtain soil parameter. The blade contains four steps going from thin to thick from its bottom to top (Figure 10). At each step there is a circular membrane that is exerted outward, measuring the soil's horizontal stress. It is recognized that even the thinnest step causes disturbance to the horizontal stresses when it is pushed into the soil. At the desired test depth, the engineer measures the horizontal stress of the soil for each blade step. By plotting the blade thickness versus the log horizontal stress, engineer can extrapolate the horizontal stress at a zero blade thickness. The documented accuracy of this method is $\pm$10% (Handy, 2008). (Note that the maximum 7.5-mm-thickness of the K_o step blade is half that of the 15-mm-thick DMT blade.)

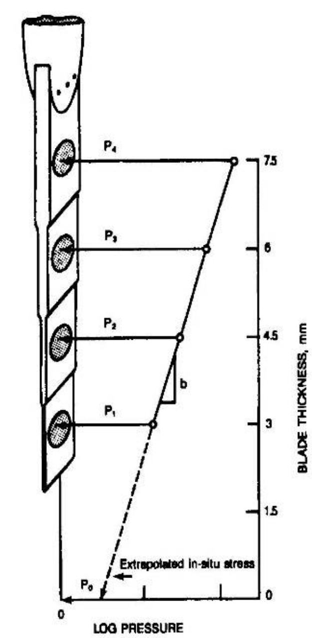

Figure 10: Ko step blade (Handy, 2004)

Vane Shear Test (VST), ASTM D 2573: This test accurately determines the undrained shear strength of purely cohesive soils by rotating a small vane having four blades (Figure 11) around its vertical axis to fail a cylinder of soil in torsional shear. The friction acting on the rods must be subtracted from the total torque applied at the top of the rod string, but nearly all test equipment is designed to make this subtraction. Vane size can be varied to allow testing a range of soil strength using the same torque head.

Figure 11: Various size VST shear heads (left) and manual penetration rig with torque head (right)

The undrained shear strength of clay, s_u, can be obtained directly from the maximum torque (T_{max}) by the simple equation:

$$s_u = 2T_{max} / (\pi D^2 H) \quad \text{(ignoring end effects)} \ldots\ldots (1)$$

By 1972 Bjerrum had realized that, when used in stability analyses, the vane s_u did not always give a factor of safety of 1.0 when failures had occurred. He recommended correcting the vane undrained shear strength using the following equation:

$$s_{u(field)} = s_{u\,(vane)} \times \mu, \quad \text{where } \mu = 1.7 - 0.54 \log PI\% \ldots\ldots (2)$$

By continuing to turn the vane blades five to ten revolutions, the residual undrained shear strength and the resulting sensitivity of the soil can also be readily determined. Note that sand, silt, or fibrous (roots or peat) inclusions disrupt the cylindrical failure surface around the vane, leading to erroneous results.

Falling Head Permeability Test (FHPT) or BAT outflow: In 1984, Torstensson invented a probe with a discrete filter (Figure 12) for performing outflow tests and inflow tests that also served to collect groundwater samples. Wilson and Campanella (1997) showed that the filter can clog with inflow tests, which can lead to inaccurate permeability measurements, particularly in more permeable soils. They also replaced Tortenson's hyperdermic needles with 0.375 inch diameter quick connects so that the more permeable soils could be tested. The test is similar to the laboratory falling head permeability test and uses Boyle's-Mariotte's law as the basis for the computations.

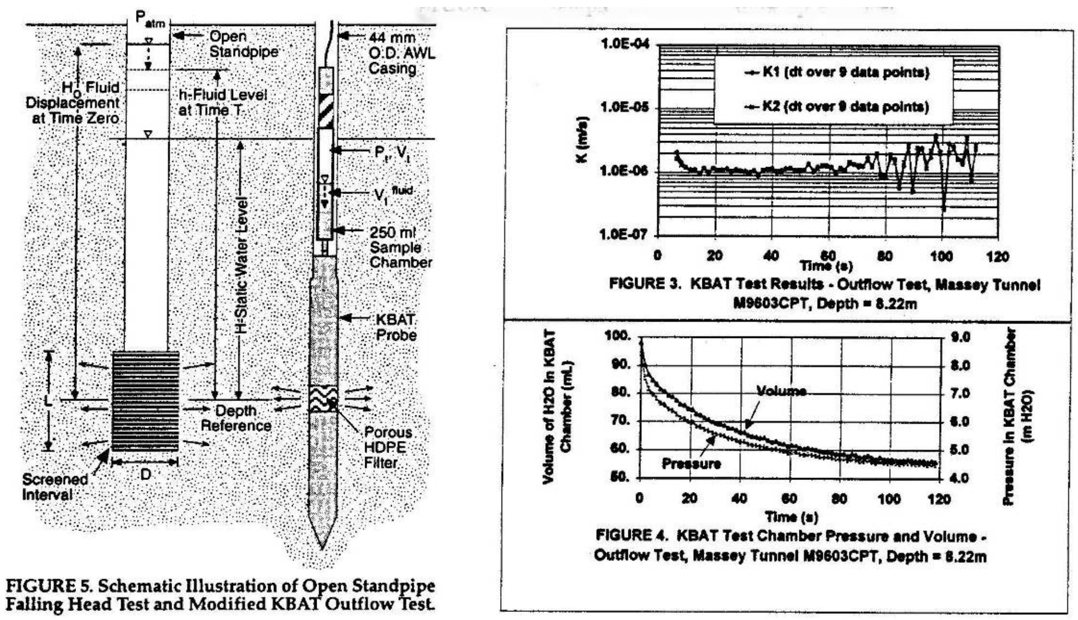

Figure 12: Falling head outflow test probe and typical results (Wilson, 1997)

Design Guide for Geotechnical Engineering

In following sections, the most appropriate in-situ test(s) is recommended for specific design applications. Table 1 summarizes these recommendations.

Table 1: Summary of Geotechnical Engineering Design Guide of Appropriate In-Situ Tests

Geotechnical Design Application	Soil Type	Suggested In-Situ Test Ranking (1 => most appropriate; 3 => least appropriate & N/A => not applicable)							
		SPT	DMT	CPT	PMT	BST	KSB	VST	FHPT
Shallow Foundations									
-settlement	cohesive	N/A	1	2[1]-3	1	N/A	N/A	N/A	N/A
	cohesionless	3	1	2[1]-3	1	N/A	N/A	N/A	N/A
-time rate of settlement	cohesive	N/A	2	2	N/A	N/A	N/A	N/A	1
-bearing capacity	cohesive	3	1	1	2	1	N/A	1	N/A
	cohesionless	3	1	1	2	1	N/A	N/A	N/A
Slope Stability	cohesive (total stress)	3	1	3	N/A	N/A	N/A	1	N/A
	cohesive (effective stress)	N/A	N/A	N/A	N/A	1	N/A	N/A	N/A
	cohesionless	3	1	2	N/A	1	N/A	N/A	N/A
Ground Improvement	cohesive	3	1	1[1]-2	1-2	N/A	1-2	N/A	N/A
	cohesionless	2[1]-3	1	1[1]-2	1-2	1-2	1-2	N/A	N/A
Deep Foundations									
-axial capacity	cohesive	1	2	1	1	2	N/A	2	N/A
	cohesionless	1	2	1	1	2	N/A	N/A	N/A
-lateral capacity	cohesive	3	1	N/A	1	N/A	N/A	3	N/A
	cohesionless	3	1	N/A	1	N/A	N/A	N/A	N/A

Note #1: Requires site specific correlations

Shallow Foundations

The engineer should always prove that a shallow foundation will not adequately support the load before recommending a deep foundation or ground improvement. Shallow foundations should be designed for sufficient bearing capacity and tolerable settlement. Bearing capacity depends on the soil's shear strength, while settlement depends on its deformation modulus. The settlement criterion generally controls design provided that the footings are wide enough.

Settlement: Dilatometer and pressuremeter tests are static deformation tests and reliably measure the soil's static deformation modulus. Both tests can provide an initial tangent modulus representing a strain level in the elastic range of loading (about 0.5 to 1%), similar to the working load that most structures impose on soil. Menard developed empirical formulas for the PMT, based on numerous case studies, to compute settlement from the pressuremeter initial modulus. Settlement analysis with the DMT follows a more traditional approach, applying an elastic estimate of the expected stress increase profile to the DMT profile of the vertical modulus.

Schmertmann (1986) presented a method to compute settlement based on DMT results. He demonstrated the accuracy of the method in 16 case histories. Hayes (1986) had additional case studies again validating the method. In Schmertmann's "ordinary" method settlement is simply calculated using the following equation:

$$S = (\Delta\sigma)\,(\Delta H)/\,M \quad \text{..}(3)$$

Where S is the settlement; $\Delta\sigma$ is the increase in vertical stress; ΔH is the layer thickness and M is the constrained deformation modulus measured with the DMT. Because the modulus value is in the denominator of this equation, one cannot simply average the modulus value. Rather, the engineer should use the modulus from each test depth to represent a thin layer (thickness = test depth interval) at that depth. Schmertmann (1986) also presented a "special" method that attempts to account for lightly overconsolidated soils in which the modulus may decrease when the applied load exceeds the preconsolidation pressure. Generally, these two methods agree within 10%.

Penetration tests, such as the quasi-static CPT and the dynamic SPT, strain the soil to failure and therefore provide strength parameters that represent failure. The ratio of stiffness to strength increases significantly as overconsolidation increases (past stress history). As a result, modulus correlations with strength extrapolated from plastic (failure) behavior to elastic behavior necessarily include significant scatter and are usually chosen very conservatively. Site specific correlations with more accurate lab or in-situ tests can prove useful to reduce this conservatism.

Plate and conical test loads are methods to test the soil response to a directly-applied foundation stress. Plate load tests, usually a square plate 1 ft on a side, may need to be performed at several depths if the stress bulb from the plate is much smaller than the footing stress bulb. The conical test load (CTL) places the base of a cone of gravel or fill material directly on the surface of the test location, resulting in a full-scale stress increase beneath the center of the conical load. It is a convenient proof test and should be used more frequently (Schmertmann, 1993).

The time rate of settlement: In cohesive soils, excess pore water pressure is developed when the CPT or DMT probe is pushed into them. When the penetration stops, those pressures decrease. As the excess pore water pressure decreases, the engineer can measure the pressure and elapsed time. Like laboratory consolidation tests, the time for 50% dissipation to occur is computed and this value is needed to compute the coefficient of consolidation and coefficient of permeability in the horizontal direction, c_h and k_h. However, the method includes many correlation coefficients, making the accuracy of the method about one order of magnitude. A better method is the field falling head permeability test or KBAT outflow test, which provides a direct measurement of permeability using Boyle's law.

Bearing Capacity: The bearing capacity for the foundation can be evaluated using classic formulas, which have form similar to Meyerhof equation (Das, 1998):

$$q_u = (q_c + q_q + q_\gamma) = cN_c\lambda_{cs}\lambda_{cd}\lambda_{ci} + qN_q\lambda_{qs}\lambda_{qd}\lambda_{qi} + \tfrac{1}{2}\gamma BN_\gamma\lambda_{\gamma s}\lambda_{\gamma d}\lambda_{\gamma I} \quad \text{……………(4)}$$

where:

q_u = ultimate bearing capacity

c = cohesion

q = stress at depth of foundation = γD_f

γ = average unit weight of soil under footing (effective unit weight if submerged)

B = width (or diameter) of foundation

$\lambda_{cs}, \lambda_{qs}, \lambda_{\gamma s}$ = shape factors, based on footing plan dimensions

$\lambda_{cd}, \lambda_{qd}, \lambda_{\gamma d}$ = depth factors, based on width and embedment

$\lambda_{ci}, \lambda_{qi}, \lambda_{\gamma i}$ = load inclination factors, based on inclination

N_c, N_q, N_γ = bearing capacity factors, based on friction angle

The engineer must evaluate the soils' shear strength to calculate the bearing capacity. The BST can accurately measure the drained shear strength properties. The DMT provides the friction angle in cohesionless soils by back calculation based on the thrust measured during penetration and the normal stress and side shear acting on the DMT. In cohesive soils, the DMT provides well-documented correlation with the undrained shear strength. For the CPT, the friction angle is fairly well correlated with tip resistance based on tests performed in large triaxial chambers. The undrained shear strength of cohesive soils is also commonly correlated with the CPT tip resistance using a factor that varies between 10 and 20, depending on the geology and sensitivity of the clays. Shear strength correlations with SPT N-values tend to be conservative and crude.

The bearing capacity can also be predicted using empirical correlations with the net limit pressure from pressuremeter tests using the following formula:

$$q_{ult} = (k)(p^*_{Le}) + q_o \quad \text{……………………………………………………………..(5)}$$

Where q_{ult} is the ultimate bearing capacity,

k is a pressuremeter bearing capacity factor,

p^*_{Le} is the equivalent net limit pressure near the foundation level, and

q_o is the total stress overburden pressure at the foundation level.

Note that for soils exhibiting strong anisotropic behavior, the orientation of the failure plane developed during an in-situ test may prove important for predicting the shear strength along the failure plane for the footing. The DMT, BST, PMT, and VST force a failure in the vertical plane and are sensitive to lateral stress variations, which can be beneficial to the bearing capacity analysis. The CPT and SPT cause failure to occur due vertical loading and may provide a better model of the actual load behavior.

Slope Stability: Slope stability analyses generally address two limit states, total stress (undrained) and effective stress (drained). Sands are generally permeable enough to be considered as drained with the exception of earthquake or other dynamic loading conditions. For clays, the engineer should analyze the slope using both drained and undrained shear strength properties. Overconsolidated clays tend to have high undrained shear strengths and are more critical when using drained shear strengths, while normally consolidated clays tend to have lower undrained shear strengths and are more critical when using undrained shear strengths. Overconsolidated clays often have residual shear strengths that are significantly lower than peak shear strengths. Residual strengths should be used in the analyses when there are preexisting failure surfaces or slickensides that are oriented in the direction of the critical failure surface.

The BST is the only in-situ test that measures the drained shear strength of cohesive soils. Some preliminary testing has been performed to measure the residual shear strengths by repeatedly shearing the soil. The BST can also accurately measure the drained angle of internal friction for cohesionless soils, provided that there are no particles larger than 1 cm in diameter.

The DMT and CPT can also provide reasonable measurements of the friction angle for cohesionless soils, while SPT data provides very conservative estimates. The VST and DMT provide good estimates for the undrained shear strength of cohesive soil; the CPT is correlated with undrained shear strength, which depends on correlation coefficients that typically range from 10 to 20; and the SPT again tends to provide conservative estimates. As noted above for bearing capacity, orientation of the failure plane may again prove important for strongly anisotropic soils.

Ground Improvement: Often soils are improved so that the structure can be safely supported on shallow foundations by previously inadequate soils. Loose granular soils are densified, usually by a dynamic method. Soft cohesive soils are usually preloaded, often using wick drains to shorten the consolidation time. The end result is that soils' deformation moduli and shear strengths are increased. Often soils are tested before and after the improvement effort to evaluate its effectiveness. The SPT variability and relative insensitivity to ground improvement changes make it a relatively poor choice for this type of testing, and lab testing of field samples cannot provide the quantity of data required to verify improvement of the overall mass of material.

In cohesionless soils, ground improvement techniques often both increase lateral stresses and compact the soil. These changes lead to both a greater friction angle and increased stiffness as any excess pore pressures rapidly dissipate. They also may encourage an "ageing" process that further increases the shear strength and stiffness. The amount of improvement that occurs depends on the dynamic effort and the distance away from the dynamic source. The improved soils will be fairly heterogeneous in both the vertical and horizontal directions. A large number of tests are needed to confirm that the soils have been adequately improved at all desired locations.

In-situ tests with high shear strain and disturbance effects measure ground improvement poorly because they destroy the improvement during the test. Because the DMT, and possibly the PMT, accurately measure both the soil's deformation modulus and the at rest lateral pressure with minimal ground disturbance, they provide the best choice to determine whether sufficient ground improvement has been performed (see Figure 12). By performing a few DMT and CPT soundings close to each other, a site specific correlation can sometimes be developed to more reliably compute the deformation modulus from the CPT tip resistance (Schmertmann, et al., 1986). Then, because a CPT sounding requires only about half the time needed for a DMT sounding, the CPT can provide the bulk of the verification tests, saving time and reducing testing costs.

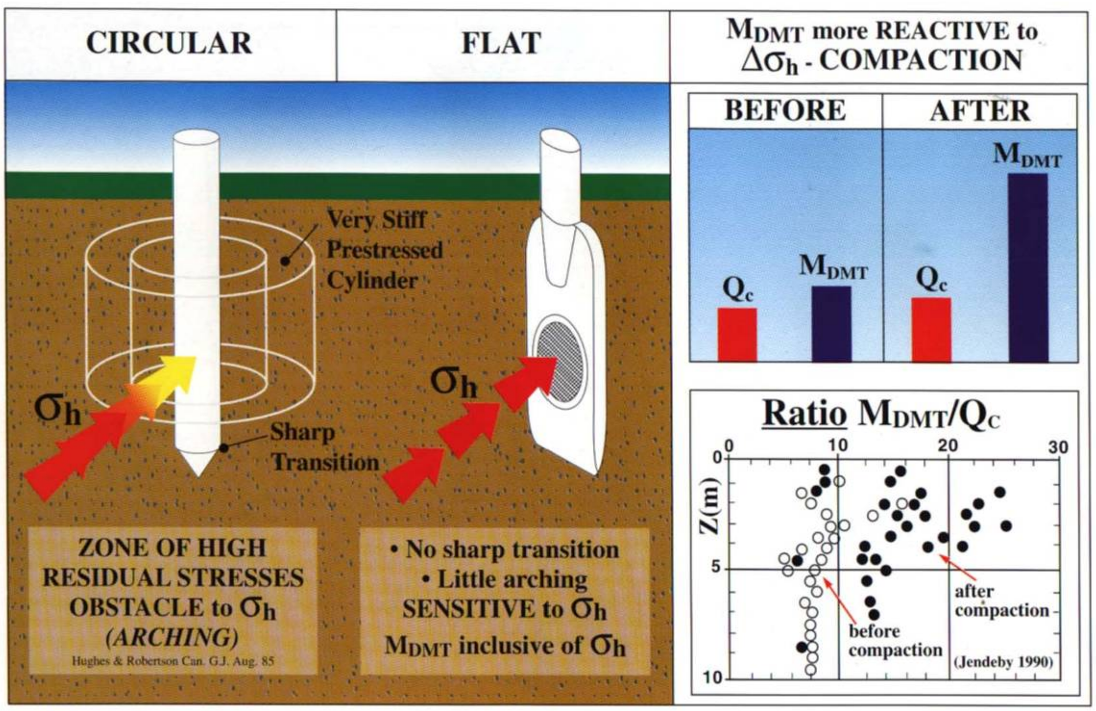

Figure 13: Better measurements of ground improvement using DMT instead of CPT (Marchetti, 1998)

When the ground improvement uses cement or chemical grouting, there may be cemented layers that cannot be penetrated using direct push methods of DMT or CPT. Pressuremeter tests should be done to measure the deformation modulus and serve as the calibration test. A site-specific correlation between the PMT and the SPT could then increase testing productivity. The minimum acceptable N_{60}-value should be chosen based on the comparison with the acceptable PMT modulus. Penetration may also not be possible if the improved materials contain rock fragments or concrete/urban debris. In this case a slotted casing pressuremeter is required.

In cohesive soils, ground improvement is often monitored by measuring the decrease of in-situ pore pressures as the soil consolidates under the applied pre-load. However, this will only confirm the completion of the consolidation process. In-situ tests are then required to confirm the improvement of strength and stiffness both of which the DMT and PMT can verify. Alternatively, the CPT, BST, and VST can verify strength improvement. The CPT tip resistance can again be calibrated with the DMT modulus, and then the bulk of the testing can be performed with the quicker, cheaper CPT. PMT results could also verify improvement, but with greater cost due to additional time of testing and analysis effort.

Deep Foundations

Axial Capacity: Both the SPT and the CPT provide good models for determining the vertical capacity of a deep foundation, with the SPT generally better for driven piles. While numerous analytical methods have been developed to determine vertical capacity, the methods that directly use N_{60} or the CPT tip resistance are more accurate than classic methods that use shear strength parameters determined from empirical correlations. Because both tests provide a depth profile of test results (more data points with the CPT), the engineer can also prepare depth plots of total pile capacity, side resistance, and end bearing. Furthermore from those plots, the engineer can make a contour map of the required tip depth for the entire site. (see Failmezger & Bullock, 2004). In stronger soils, the SPT provides more reliable test results. The CPT may reach refusal in strong thin layers that will not stop either the SPT or a pile. The SPT also provides the best tool to determine the drivability of a pile, and is the most likely test to recognize potential capacity reduction due to dynamic penetration in lightly cemented soils and sensitive clays. Correlations with both SPT and CPT usually include a database of comparisons with static load testing.

If the CPT or SPT cannot penetrate the foundation materials (soil or rock), then the PMT can be performed to calculate vertical capacity. Numerous pressuremeter tests have been performed in conjunction with pile load tests and correlation coefficients have been refined for the PMT-based analytical methods.

Historically, engineers have grossly underestimated the vertical capacity of rock sockets, primarily because they have not been able to accurately measure the rock’s shear strength or run a load test on the rock to failure. The rock borehole shear test is a new method to determine the rock’s shear strength. Classic shear strength capacity equations can be used to predict the vertical capacity. Osterberg load tests should be used to measure the rock socket failure capacity and to refine correlations with the RBST. While this is an area of research, site specific correlations can be used now.

Negative Skin Friction: When the soil surrounding a pile moves more than the design pile settlement, then negative skin friction or "downdrag" occurs. The axial capacity of the pile will not decrease, but undesirable foundation settlement may occur as the capacity is "remobilized". This often happens when fill is placed on a site that contains soft compressible soils. The engineer must determine the neutral point, where the negative skin friction ends and the positive skin friction begins. As above, the pile's side resistance can be estimated from SPT or CPT testing. However, the engineer must also accurately compute settlement to quantify identify the zone over which the soil settles more than the pile. The best way to do this is with DMT data. By cumulatively calculating settlement from the bottom of the sounding to the ground surface, the depth where the design settlement occurs can be determined. Above this depth is negative skin friction and below it is positive skin friction. Only a small amount of movement (<0.25 inches) is required to fully mobilize friction, whether positive or negative.

Lateral Capacity: Correlations have been developed to estimate the deformation behavior of laterally loaded piles from strength parameters, but tests that actually measure both strength and stiffness will provide superior design parameters. Because both the DMT and PMT test the soil horizontally, they are the best methods to evaluate lateral capacity. The engineer can determine accurate P-y curves from those methods and use them with numerical computer programs such as LPILE and COM624. The Dilatometer is the best choice if it can be pushed, because a continuous P-y profile can be easily established. The pressuremeter is needed for harder soils and for rock.

Conclusions

Though usually testing less than 0.01% of the overall mass of soil and rock supporting a foundation, in-situ tests generally investigate a much greater volume of soil more quickly than possible for sampling and laboratory tests. Thus they provide both cost savings and increased statistical reliability for foundation design. The additional foundation cost from poor geotechnical design greatly exceeds the additional cost of these tests to obtain better engineering design. Therefore, performing appropriate in-situ tests to support more reliable design should prove economical on every significant project.

The type of information required for a particular design application should drive the choice of an in-situ test. The in-situ test chosen should compare favorably with the application, including stress path, test type (static vs. dynamic), orientation (lateral vs. vertical), level of stress (elastic vs. plastic), and the controlling design parameters (strength, stiffness, stress).

For shallow foundation design, the DMT, CPT, and PMT provide bearing capacity parameters in all penetrable soils, but only the DMT for penetrable soils and the PMT for harder/stronger soils provide reasonable settlement estimates. (The CTL provides a good full-scale proof test of settlement.)

For slope stability design, the BST should be used to quantify the drained shear strength parameters; the DMT to determine the drained friction angle of cohesionless soils and undrained shear strength of the cohesive soils; and the VST for the undrained shear strength of the cohesive soils.

For ground improvement verification the DMT provides the best sensitivity, but the CPT tests the mass of foundation material more efficiently if it can be correlated with the dilatometer results at the site.

For deep foundation axial capacity, the SPT or CPT should be used if they can penetrate to the depth desired. The PMT should be used in harder soils. For lateral capacity of deep foundations, the DMT should be used where it can be pushed and the PMT should be used in the harder soils.

References

Annual Book of ASTM Standards, *Section 4 Construction, Soil and Rock*, 2008, Vols. 4.08 and 4.09, 2008, ASTM International, West Conshohocken, PA.

Bjerrum, L., 1972, "Problems of soil mechanics and construction on soft clays," *Proceedings from 8th ICSMFE, Vol. 3*, Moscow, pp. 111-159.

Briaud, J-L, 1992, *The Pressuremeter*, A.A. Balkema, Brookfield, VT.

Bustamante, M. and Gianeselli, L., 1982, "Pile bearing capacity prediction by means of static penetrometer CPT", *Proceedings of the Second European Symposium on Penetration Testing*, Amsterdam, pp. 493-500.

Campanella, R. G., 2008, "Geo-environmental site characterization", *Proc. Geotechnical and Geophysical Site Characterization*, Huang, A. B. and Mayne, P. W., editors, London, pp. 3-15.

Crapps, David K., 2006, "The use of DMT data for lateral load analyses", *Proceedings from the Second International Flat Dilatometer Conference*, Washington, D.C., pp. 190-197.

Das, B. M., 1998, *Principles of Geotechnical Engineering 4th Edition*, PWS Publishing Company, Boston, MA.

Failmezger, R. A. and Bullock, P. J., 2004, "Individual foundation design for column loads", *Proc. Geotechnical and Geophysical Site Characterization*, Viana da Fonseca, A. and Mayne, P., editors, Rotterdam, pp. 1439-1442.

Failmezger, R. A. and Bullock, P. J., Handy, R. L., 2004, "Site variability, risk, and beta", *Proc. Geotechnical and Geophysical Site Characterization*, Viana da Fonseca, A. and Mayne, P., editors, Rotterdam, pp. 913-920.

Failmezger, R., Zdinak, A., Darden, J., Fahs, R., 2005, "Use of rock pressuremeter for deep foundation design", *Symposium International ISP5/Pressio*, Paris, pp. 469-478.

Failmezger, R. A., White, D. J. and Handy, R. L., 2008, "Measurement of effective stress shear strength of rock", *Proc. Geotechnical and Geophysical Site Characterization*, Huang, A. B. and Mayne, P. W., editors, London, pp. 335-339.

Handy, Richard L., 2008, personal communication

Handy, Richard L., Remmes, Bernard, Moldt, Steven, Lutenegger, Alan J., and Trott, Gary, 1982, "In Situ Stress Determination by Iowa stepped blade", *Journal of the Geotechnical Division*, Proceedings of the American Society of Civil Engineers, Vol. 108, No. GT11, pp. 1405-1422

Handy, R. L., Schmertmann, J. H., Lutenegger, A. J., 1985, "Borehole shear tests in a shallow marine environment", *Strength testing of marine sediments: laboratory and in-situ measurements, STP 883*, American Society for Testing and Materials, Philadelphia, pp. 140-153.

Handy, Richard L., 1986, "Borehole shear test and slope stability", *Use of InSitu Tests in Geotechnical Engineering*, Samuel P. Clemence, editor, ASCE, New York, pp. 161-175.

Handy Geotechnical Instruments, Inc., 2004, “Instructions Ko Stepped Blade”

Hartmann, Burt, 2008, personal communication

Kulhawy F. H. and Mayne, P. W., August 1980, *Manual on Estimating Soil Properties for Foundation Design*, EPRI EL-6800, Electrical Power Research Institute, Palo Alto, CA.

Lunne, T., Robertson, P. K. and Powell, J.J.M, 1997, *Cone Penetration Testing in Geotechnical Practice*, Blackie Academic and Professional, London, England.

Lutenegger, Alan J., 2006, “Cavity expansion model to estimate undrained shear strength in soft clay from dilatometer”, *Proceedings from the Second International Flat Dilatometer Conference*, Washington, D.C., pp. 319-326.

Marchetti, Silvano, 1980, “In Situ Tests by Flat Dilatometer”, *Journal of Geotechnical Engineering*, Vol. 106, No. GT3, pp. 299-321.

Marchetti, Silvano, 1998, “Dilatometer Brochure”, Geotechnical and Geophysical Site Characterization ISC’1 Conference in Atlanta

Mirza, C., 1998, “The high cost of low bid cost site characterization”, *Proc. Geotechnical and Geophysical Site Characterization*, Robertson, P. K. and Mayne, P. W., editors, Rotterdam, pp. 227-232.

Monaco, P., Totani, G., Calabrese, M., 2006, “DMT-predicted vs observed settlements: a review of available experience”, *Proceedings from the Second International Flat Dilatometer Conference*, Washington, D.C., pp. 244-252.

Pennsylvania Department of Transportation/Federal Highway Administration, 1988, *Report Number: FHWA-PA-87-024+84-24, Guidelines for Using the CPT, CPTU and Marchetti DMT for Geotechnical Design*, Volumes 1-4.

Robertson, P. K., Davies, M. P., and Campanella, R. G., March 1989, “Design of Laterally Loaded Driven Piles Using the Flat Dilatometer,” *Geotechnical Testing Journal*, GTJODI, Vol. 12, No. 1, pp. 30-38.

Schmertmann, J.H., 1975, "Measurement of In Situ Shear Strength", *Proceedings of the Conference on In Situ Measurement of Soil Properties, Vol. 2*, ASCE, NY, NY.

Schmertmann, J.H., 1978, "Use the SPT to Measure Dynamic Soil Properties? - Yes, But..!", *Dynamic Geotechnical Testing*, STP 654, ASTM, Philadelphia, PA.

Schmertmann, J.H., 1979, "Energy Dynamics of the SPT", *JGED*, GT8, ASCE, NY, NY.

Schmertmann, John H. and Crapps, David K., 1983, “Use of in situ penetration tests to aid pile design and installation”, *Proc. Geopile ’83 Conference*, Dougherty and Hunt editors, Hollywood, FL

Schmertmann, John H., 1985, “Measure and use of the insitu lateral stress,” *The Practice of Foundation Engineering*, The Department of Civil Engineering, Northwest University, pp. 189-213

Schmertmann, John H., 1986, “Dilatometer to compute foundation settlement,” *Use of InSitu Tests in Geotechnical Engineering*, Samuel P. Clemence, editor, ASCE, New York, pp. 303-321.

Schmertmann, J., Baker, W., Gupta, R. and Kessler, K., 1986, "CPT/DMT QC of Ground Modification at a Power Plant", *Use of InSitu Tests in Geotechnical Engineering*, Samuel P. Clemence, editor, ASCE, New York, pp. 985-1001.

Schmertmann, John H., 1991, “The mechanical aging of soils”, *Journal of Geotechnical Engineering*, Vol. 117, No. 9, pp. 1288-1330.

Schmertmann, John H., 1993, “Conical Test Load Measuring Compressibility”, *Journal of Geotechnical Engineering*, Vol 119, No. 5, pp. 965-971.

Schmertmann, John H., Hayes, John A., 1996, “Observations from Osterberg cell tests in bored piles,” Central Pennsylvania Section ASCE and Pennsylvania Department of Transportation

Skempton, A. W., 1986, “Standard penetration test procedures and the effects in sands of overburden pressure, relative density, particle size, ageing and overconsolidation,” *Geotechnique* *36*. No. 3, pp. 425-447.

Terzaghi, K., and Peck, R.B., 1948, *Soil Mechanics in Engineering Practice*, John Wiley & Sons, NY, NY.

Wilson, Daryl and Campanella, R. G., 1997, “A Rapid In-Situ Hydraulic Conductivity Measurement in Sands using a UBC Modified BAT Penetrometer,” *Proc. 50th Canadian Geotechnical Conference*, Ottawa