USE OF IN SITU PENETRATION TESTS TO AID PILE DESIGN & INSTALLATION

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All in situ penetration tests to some degree model the penetration of a driven pile. Hollanders originally developed the Dutch Cone penetration test (CPT) as a model pile for determining end bearing and side friction support capability for their driven displacement piles founded in sand layers underlying weak and compressible peat and clay. The dynamics of performing the standard penetration test (SPT) to some degree models the dynamics of a hammer driving a pile. Both tests have proven very useful for the empirical design of pile foundations—the SPT until recently mostly in the USA, and the CPT until recently mostly in Europe.

In the past ten years new developments have taken place that considerably expand the usefulness of the SPT and CPT model penetration tests. The profession has a new understanding of the statics and dynamics of the SPT. Calibration work in large chambers with the CPT and the new appreciation of pore pressure effects have made it possible to determine the engineering properties of soil that relate to pile driving and pile capacity in addition to simply treating the CPT as a model test.

In addition, a new generation of penetration tests has emerged, led by the Marchetti dilatometer test (DMT). The DMT allows introducing horizontal stress effects into pile capacity calculations and thus permits a better accounting for this very important variable in pile capacity and pile movement predictions.

Recent research provides a better understanding of old as well as new penetration tests. This improved understanding allows an improved theoretical basis for pile design. It also provides practical information of use to the contractor and engineer regarding pile penetration and pore pressure effects. In this paper the authors review new developments in penetration testing and some new uses of penetration test data for design. Then, by means of a case history involving the pile capacity predictions for the foundations for a large bridge in Florida, they illustrate some uses for the above in situ test methods.

THE SPT

As an introduction it is proper to say that the often maligned SPT, especially by foreign engineers, has gained some new stature with respect to using it to predict pile capacity. The May 1982 ESOPT II meeting in Amsterdam featured a contest to see who and what method could best predict the capacity of a pile driven prior to, but tested at the meeting. The contestants had SPT, CPT and even Menard pressuremeter test data available to them. L. Descourt from Brazil won the contest using a method based on SPT data! Clearly the SPT has value for the design of pile foundations. Some recent developments have added to this value.

Energy Calibrations

The research of Schmertmann[20], Kovacs[12], and Schmertmann and Palacios[21], has shown that the SPT N-value over the ordinary N-value range of interest varies inversely with the ENTHRU energy that enters the drill rods as a result of the SPT hammer impact. Comparing ENTHRU with the constant nominal hammer blow energy of 4,200 in-lb. gives the Energy Ratio, denoted ER_i. Kovacs and Salamone[13], by compiling their own work and the extensive work of other investigators, have confirmed that the ER_i values among a representative selection of SPT drill rigs varies from about 30 to 80 percent. The above implies a variability in N-value results by a factor of about 80/30 = 2.7 due only to variable ENTHRU. Thus, the engineer can now improve the quantitative use of N-value data by measuring and reporting ENTHRU information and adjusting N-values to some selected reference ENTHRU. (ENTHRU in the SPT test represents that part of the potential hammer energy that successfully reaches the sampler in the form of the initial compression wave. See Ref. 21 for additional details.)

The state-of-practice has just begun to measure ENTHRU, and adjust N-values by using a commercially available SPT energy Calibrator made by Binary Instruments Inc., Wellesley, Massachusetts. The ASTM Committee D18.02 has a task group in the final stages of preparing a Tentative Method D-4220 for SPT energy measurement for optional use as a supplement to simultaneously updated ASTM D-1586. *Our experience with such energy calibration indicates it will at present cost about $250-500, plus any travel costs, to calibrate a rig for ENTHRU and prepare a report.* It usually *involves no more than one hour of time added to the normal SPT operations of the rig.*

Statics of the SPT

Schmertmann[20] showed that in the absence of significant pore pressure effects during the SPT that the SPT end bearing and inside and outside soil friction resistances had similar magnitudes to the end bearing and local side friction resistances measured in the CPT The N-values had a direct proportionality to the sum of these equivalent static resistances around the SPT sampler.

The three 6-in. incremental values of SPT blowcount $N_{0\text{-}6\ in.}$, $N_{6\text{-}12\ in.}$, and $N_{12\text{-}18\ in.}$, progressively increase when sampling the same soil because of the progressive increase in sampler side wall friction while end bearing remains approximately constant. Table 1 shows comparative ratios of N, *termed the X-ratios, can give the engineer the information required to estimate the penetrated soil's equivalent CPT friction ratio, R_f. The next step in assessing the equivalent static cone bearing resistance, q_c,* to the SPT sampling involves using Fig. 1. Entering Fig. 1 with R_f permits a breakdown of total resistance against the SPT sampler into end bearing and inside and outside friction components. Using equation (1) to estimate the total equivalent static resistance for a given ER_i, and using Fig. 1 to separate the components, allows the engineer to estimate the equivalent static end bearing and side friction resistance forces during the SPT sampling. Dividing by end bearing and side friction areas then gives the ultimate end bearing pressure and side friction stress against the SPT sampler. This separation then permits either of the following methods for pile capacity analysis.

[equation given in step 7 of Table 2] (1)

Many engineers think the CPT methods developed in The Netherlands for assigning end bearing and side friction of piles have a clear superiority over more empirical SPT methods. Breaking the SPT resistances into end bearing and local side friction permits the engineer to use the CPT methods with SPT data. Several references describe these methods in detail (10, 16, 17, 19) *and others (5, 11) show the generally good results obtained.*

Begemann R_f	W' weight of rods and hammer	soil type	$\Delta N/\Delta N_{(12-18")}$ X_1 (0-6")	X_2 (6-12")
1%	0	all with R_f	0.78	0.89
	$\neq 0$	loose sand	0.70	0.85
	$\neq 0$	dense sand	0.76	0.88
2 1/2%	0	all with R_f	0.61	0.81
	$\neq 0$	loose/weak	0.40	0.70
	$\neq 0$	strong/dense	0.59	0.795
4%	0	all with R_f	0.505	0.74
	$\neq 0$	NC day	0	0.2
	$\neq 0$	highly OC day	0.46	0.73
8%	0	all with R_f	0.415	0.71
	$\neq 0$	NC clay	0	0.5
	$\neq 0$	highly OC clay	0.37	0.69

TABLE 1 — Predicted Δ N Ratios For Sampler With Liners Removed
$C_1 = 1$, $C_2 = 0.7$ (see [20])

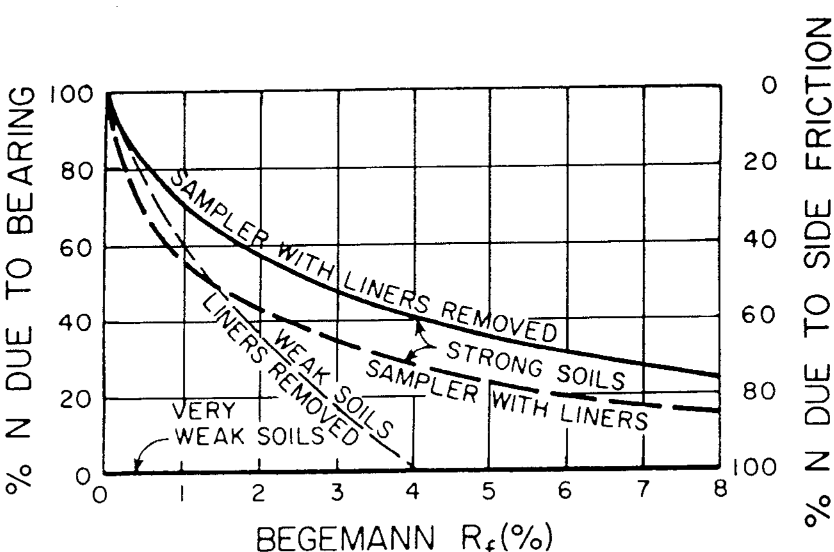


FIGURE 1 Relative Values Of QUASI - Static Components Of Sampler Penetration.

TABLE 2

A Rational Use of S.P.T. N-Values For Strength Parameters and Pile Capacity Design

(For N $\geq$ 10 to avoid any high positive pore pressure effects)

STEPS:

1. Collect N-values applicable to given soil layer.
2. Correct all N-values to a common ENTHRU[13, 21]
   (need to make measurements of SPT ENTHRU)
3. Make table listing $\Delta N_{0\text{-}6}$, $\Delta N_{6\text{-}12}$, $\Delta N_{12\text{-}18}$
4. Obtain ave> values of $X_1 = \Delta N_{0\text{-}6}/\Delta N_{12\text{-}18}$
   $X_2 = \Delta N_{6\text{-}12}/\Delta N_{12\text{-}18}$
5. Obtain friction ratio = R_F using above X-values and Table 1 herein.
6. Obtain SPT % end bearing using R_F and Fig. 1 herein, usually using the no-liner case.
7. Using ave> corrected N-value for layer under investigation, = N', and common ENTHRU %, obtain equivalent static force to penetrate soil with SPT sampler from

   F_{ST}(lbs) - 140 + 130 N' (ENTHRU/54%) (Ref. 20) eqn.(1)
8. Obtain equivalent static end bearing, $F_{ST,EB}$, by above F_{ST} X above % end bearing.
9. Obtain equivalent static side friction
   $Q_{SPT\ sampler} = F_{ST,EB}/(\text{end area sampler} \approx 10.7\ cm^2)$

Engineer now ready for strength determinations

To obtain c, ϕ, K

10. Use Durgunoglu & Mitchell bc theory[7]
11. Obtain (depth /B = 2 in.) ratio.
12. Estimate K,c,ϕ, for each soil layer based on the other available in situ and lab test data.
13. Use D&M theory to calculate Q_{SPT}
14. Revise c,o,K estimates until
    calc. Q_{SPT} = meas. Q_{SPT}
15. Then appropriate to use these final estimates with the D&M theory to help solve design pile bearing capacity.

Alternatively, further analysis of the ultimate end bearing resistance permits assigning an undrained shear strength to a soil if undrained conditions probably exist, as with clays, or the friction angle if drained conditions probably exist, as with sands. Then the engineer can use the same bearing capacity theories to evaluate the end bearing of displacement piles. Similarly, the engineer may make a rational calculation for side friction. However, these calculations involve the horizontal in situ stress before and after driving as major variables. The DMT, discussed subsequently provides such information for the analyses.

Dynamics of SPT

Schmertmann and Palacios[21], have shown that the stress wave theories of Fairhurst[8] provide a good explanation for the dynamic and pulsed penetration of the SPT sampler. As noted in the Introduction, the SPT penetration provides a model penetration of a pile as a result of a pile hammer blow. The state-of-the-art, and also largely the state-of-practice, requires the use of wave equation analysis to help with pile hammer selection, to control stresses during pile driving, and to help assure adequate pile penetration to provide the required static capacity. The engineer can also analyze the SPT sampler penetration problem using wave equation simulations. The wave equation parameters such as J_1 and J_2 (Refs. 9, 21) determined by analyzing the SPT can also then provide the J values for use when analyzing driven piles by wave equation methods. In this way the SPT can contribute its dynamic modeling capability in a rational way towards the better analysis of the driving of piles.

THE CPT AND THE CUPT

Chamber Test Correlations

The relatively recent use of very large sized triaxial test chambers (samples 4 ft in diameter and 4 to 6 ft high), with controllable boundary conditions and designed for the insertion of penetrometers such as the CPT, has greatly expanded the research base for the correlation of CPT data against the engineering behavior of sands. Basically, testing in such chambers has allowed the identification of the major variables controlling CPT resistance and identified these as relative density, effective stress magnitude (especially horizontal stresses), and shape of the CPT tip. Chamber testing allows the preparation of duplicate specimens and the separation of density and stress effects—factors usually impossible to control and separate in the field. The following references provide some correlation information obtained from chamber testing: Schmertmann[18], Baldi et. al[1, 2]. An engineer may find such correlations of use when determining the need for a deep foundation, the best type of deep foundation, and designing the performance of driven piles should he or she choose that alternative.

Sand Friction Angle by Durgunoglu and Mitchell Theory[6]

Piles with significant end bearing support usually derive this support from layers of sand, cemented sand, gravel (or other dense cohesionless soils) or rock. If in cohesionless soil, then in accord with all bearing capacity theories of the Terzaghi type, the friction angle of the soil controls the bearing capacity. One of the latest, and perhaps also the best deep foundation bearing capacity theory available comes from the work of Durgunoglu and Mitchell (the D&M theory). This theory is presently recommended for the calculation of bearing capacity if one knows the friction angle, or for calculating the friction angle if one knows the bearing capacity in a cohesionless soil. To the writers' knowledge this represents the only theory that takes quantitative account of the in situ horizontal stress effects on bearing capacity.

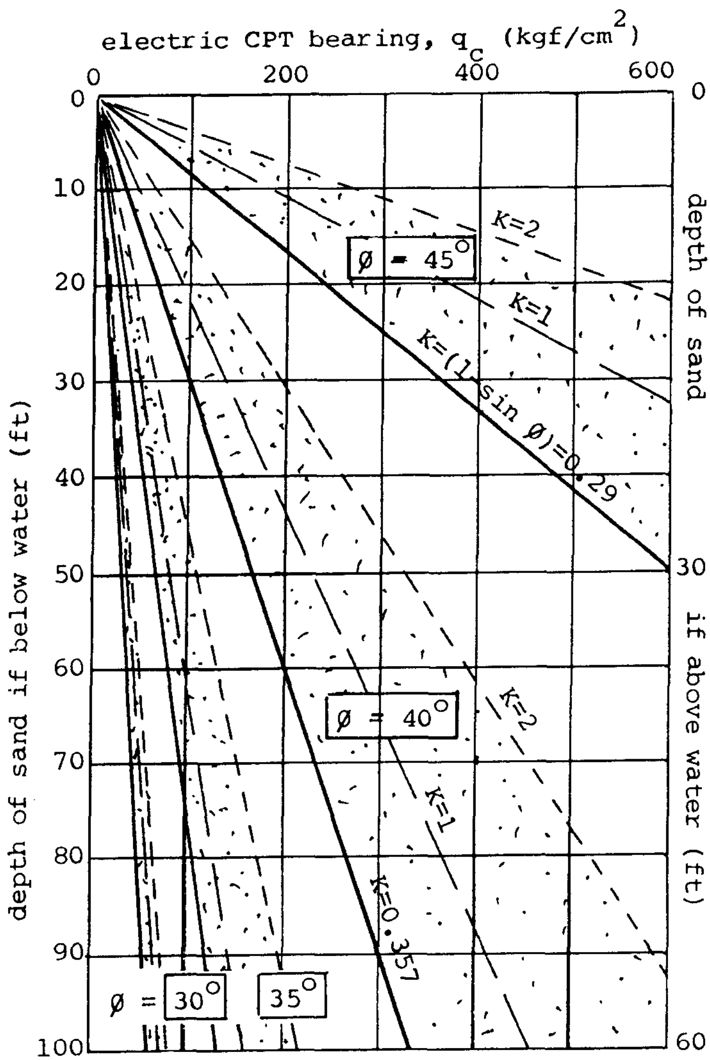
Although first developed for the shallow penetrometer exploration on the surface of the moon, recent developments that have included chamber test confirmations indicate that this theory, which relates friction angle and vertical and horizontal stresses to bearing capacity in sands, works reasonably well for the prediction for bearing capacity of deep foundations such as displacement piles. However, note that the curvature of the Mohr envelope means that the average, or secant, value of ϕ depends on some average stress level around the cone during the CPT. The engineer can attempt to correct for this effect, but stress levels in the CPT usually exceed those under a pile and thus using ϕ and bearing capacity values backfigured from CPT data should usually prove conservative.

Fig. 2 shows the D&M theory predicted values for friction angle for various CPT q_c values, depths, water table and horizontal stress conditions. The depths shown apply to effective soil unit weights of 105 and 62 lbs/ft^3 for the above- and below-water table conditions, respectively. Significantly different unit weights require correcting the depth scale by the ratio of the above to the actual unit weights (vertical scale actually for effective overburden pressure at depth of q_c). After using Fig. 2 to determine a ϕ value the engineer can then use the same D&M theory to predict the ultimate bearing capacity of pile tips driven into sands. However, soil layering effects dependent on pile sizes and the effects of vibrations from driving piles may reduce such ultimate capacity[6].

The CUPT and Pore Pressure Effects

The recent invention of the CPT cone tip that incorporates a pore pressure sensing element (the reason for the U in CUPT) has the potential for expanding the usefulness of CPT data, including the design of pile foundations. Wissa et. al.[25] and Torstensson[24] both simultaneously first reported the development of such a piezometer probe, or piezometer cone, or piezocone in 1975. In its most modern form it now permits the simultaneous measurement of cone resistance and local friction resistance as well as instantaneous pore pressure during penetration. The engineer can also observe this pore pressure dissipation with time after stopping the cone advance. The dissipation rate allows the engineer to estimate the rate of consolidation and permeability behavior of the soil surrounding the CPT tip[3, 4].

One of the outstanding advantages of the CUPT comes from its ability to detect many of the fine details of soil layering not otherwise detectable. Such details may influence pile design and performance—especially with



Example: say q_c = 200 kgf/cm^2 at 40 ft depth
water table at 10 ft depth
K uncertain, between (1-sin ϕ) and 2
find range of estimated triaxial ϕ
if K = (1-sinϕ), ϕ= 42° for surface water table
= 39° for w.t. below 40 ft
= 41° for 10 ft depth to water
if K = 2 , ϕ= 38 to 36° depending on water
= 37.5° for 10 ft depth to water
answer: 37.5-41° , depending on K

FIGURE 2 DURGUNUGLU & Mitchell Theory Predictions of Triaxial ϕ In Sands Based on Knowing The CPT Bearing Capacity, q_c, The Depth of q_c, and The Lateral Stress Coefficient, K.

respect to such pore pressure related problems as pile freeze, soil heave and the lateral effects of the cumulative pore pressure increase around piles during their installation. However, the engineer can only use the CPT in soils penetrable by quasi-static methods with the equipment available. This limitation may not permit penetrating into those layers intended to support end bearing piles.

If the CUPT detects a large buildup of positive pore pressures resulting from the displacement of soil by the cone and overhead rods, then the same will probably occur in the soil displaced by a pile or pile group. If the CUPT detects a slow dissipation of this pressure, then the same will likely occur around the piles. Such a situation would probably predict very easy driving conditions and a large freeze effect taking considerable time to occur. The CUPT results permit, in principle, a calculation of this time to occur. Similarly, the CUPT generation of negative pore pressures would predict higher than expected driving resistance and a negative freezing effect (the relaxation and reduction of bearing capacity with time). If the CUPT data showed a rapid dissipation of either positive or negative pore pressures with time, then the calculation for this rate for the real pile foundation would probably also yield relatively rapid positive or negative freeze effects. Many engineers will appreciate a logical prediction of pile pore pressure and freeze effects to rationally account for these effects in design, construction and proof testing. At present the engineer must just wait and see what happens and adjust accordingly.

The reader should note that there is not yet a standard for the position and shape of the piezometer element on the cone tip. Most of the currently available designs place this element as a thin band immediately above the base of the cone. This location tends to give the maximum negative pore pressure effect and perhaps thus discriminates with the current maximum sensitivity between negatively and positively dilatent soils. About 2/3 of the current designs use this location. Some designs have the element approximately midway between the point and the base of the cone. This position tends to give the highest positive pore pressure values and subdues negative values. It perhaps gives the most appropriate values for subsequently discussed pore pressure corrections to CPT q_c data. Other designs, including the original ones by Wissa and Tortensson, place the piezometer at the point of the cone. This location probably gives intermediate effects between the other two locations and gives the finest soil layering detail.

If the CUPT detects a large buildup of positive pore pressures resulting from the displacement of soil by the cone and overhead rods, then the same will probably occur in the soil displaced by a pile or pile group. Most importantly, sands with high silt contents can develop significantly large positive or negative pore pressures. These subtract or add directly to the effective stresses calculated when assuming hydrostatic pore pressure conditions. But, watch out for some CUPT tip designs where the pore water pressure does not act over the full cross sectional area of the cone tip—these require special correction factors which the manufacturer must supply or the user must find them by calibration.

Because CPT resistance depends on effective stresses, the engineer can most accurately evaluate soil properties from the CPT by first correcting

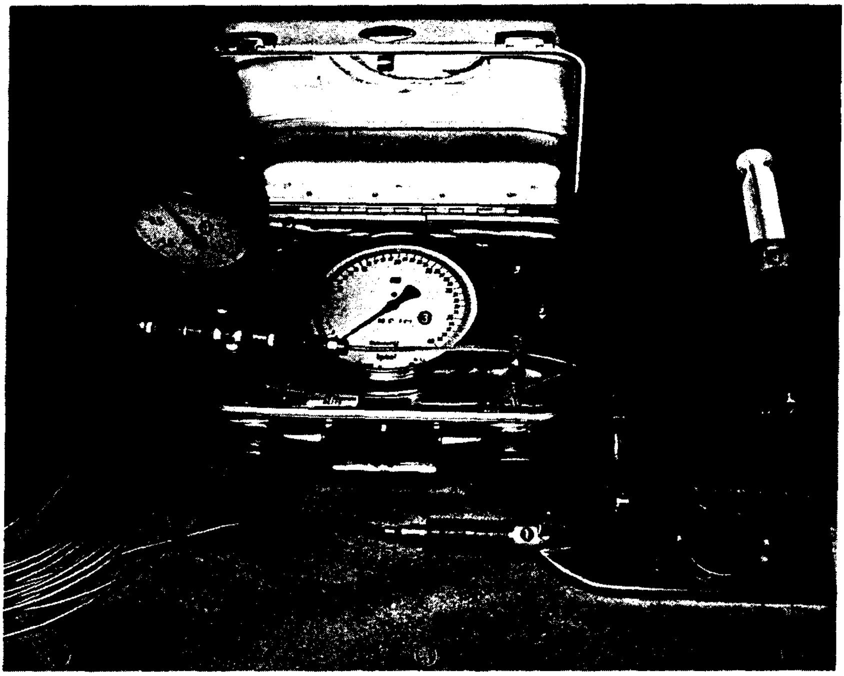


FIGURE 3 – MARCHETTI DILATOMETER
(Shown Set Up for Membrane Calibration)

1. *Blade with thread connection to rods (boring or CPT)*
2. 60 mm membrane
3. Control box and pressure gage
4. Special equipment for calibration
5. Nitrogen tank plugs in here
6. Valve to control rate of pressure increase
7. Tubing to carry pressure to blade

for any significant pore pressure effects. The writers cannot now recommend an analytical method to accomplish this correction. Do it in the field by varying the rate of penetration for the q_c measurement and extrapolate for the q_{co} at a rate of zero. As a quide for what to expect, two such experiences by the senior writer gave (q_{co}/q_c) = 1.7 and 2.4 when $(\Delta u/\sigma_{vi}'$ = ratio of pore pressure increase during standard q_c to initial effective overburden pressure) = 0.19 and 0.26, respectively, in normally consolidated, Florida marine silty fine sand and organic clayey silt, respectively. A pore pressure decrease during q_c will require reducing this q_c to obtain q_{co}. The engineer should use the corrected q_{co} value in such correlation figures as Fig. 2 herein, which do not include pore pressure effects.

Note that research has shown that clean fine sands and all more permeable (coarser) sands produce negligible pore pressure effects during CPTs using the standard 1-2 cm/s rate of penetration. Significant pore pressure effects may occur in very silty sands or sandy silts, and probably do occur in loose and dense silts and clays.

MARCHETTI DILATOMETER TEST (DMT)

Description

Marchetti developed this test and has described it in detail[14, 22]. Fig. 3 shows the DMT equipment. Briefly, the test involves the successive repetition of two steps. First, the operator advances the DM blade vertically through the soil to the test depth. This blade consists of a flat stainless steel wedge 14 mm thick, 94 mm wide, and about 200 mm long. The bottom 50 mm of the width of the blade gradually tapers to form a sharp (approx. 16°) cutting edge. The operator can advance the blade into the soil either by pushing (as with CPT equipment) or driving (as with SPT equipment), and from either the ground surface or the bottom of a borehole beginning at any depth.

The second step in the sequence involves an expandable stainless steel diaphragm centered on one of the vertical faces of the blade, 60 mm in diameter, and with its center 90 mm above the cutting edge. The outside face of the membrane lies on the same plane as the flat face of the dilatometer blade. The operator first increases the gas pressure on the blade side of the membrane until it just lifts off its seating and begins to move away from the blade and into the soil. This liftoff pressure represents the DMT A-reading. The operator then continues to expand the membrane by increasing the gas pressure until the membrane has moved 1 mm into the soil. The pressure required to just obtain this movement represents the DMT B-reading. Then, by means of suitable correlations provided by Marchetti and others[14, 23], the engineer can interpret the A and B-readings for a variety of soil properties including in situ horizontal stress conditions, strength and modulus. After obtaining a set of A and B-readings at a test depth, the operator then repeats step 1 and advances the DMT blade to the next test depth, generally an 0.15 to 0.30 m (6 to 12 in.) deeper, and repeats step 2, etc. until reaching the final test depth of interest or until reaching the limit of penetration ability. Deeper testing may then require boring support to allow the DMTs to continue from the bottom of a borehole.

The writers first introduced this test in the USA in 1979 after its development in Italy beginning about 1974. It is now used routinely. To date only a few engineers and universities have become familiar with the equipment and use it in practice. However, the writers have included it in this paper because of its speed, economy and practicality and because they consider it potentially very useful for pile analysis.

Measures Horizontal Stress

The ability of the DMT to tell the engineer something about in situ horizontal stresses represents one of its key advantages. To date, only the pressuremeter test (PMT) offers a similar advantage. However, the PMT takes much more time to perform, costs considerably more per test, and also introduces the uncertainties of the effects of soil disturbance associated with making the required borehole in which to test. The latter introduces an important element of operator technique. In contrast, the DMT requires no borehole and introduces a small but constant amount of disturbance into the soil with each test. Operator technique has only a minor effect on DMT results. Although a perfectly performed PMT may in theory provide results superior to those from the DMT, real-life comparisons suggest that at least sometimes the DMT actually provides results, including in situ horizontal stress, of equal or superior quality.

Heretofore the static analysis formula methods for predicting pile friction capacity have always had the uncertainty of what horizontal stress to choose acting on the side of the pile after the insertion of the pile. The DMT provides significant data on this point. The horizontal stresses before the blade insertion come from the correlations established by Marchetti[14]. After the blade insertion, which introduces a horizontal displacement of 7 mm outward from the axis of the blade, the A-reading gives a direct measure of the total horizontal stress acting on the vertical face of the blade. If this 7 mm expansion brings the soil to its limit pressure, then the A-reading minus pore pressure represents the same limit pressure against the vertical sides of a displacement pile. If the blade displacement does not reach the limit pressure, then the A-reading represents a lower bound, and therefore conservative, estimate of the total horizonal pressure against the pile. Knowing this horizontal pressure, and the cohesive and frictional strength components in the soil, permits the desired calculation of the ultimate static side friction resistance of the pile.

Of course, even with dilatometer test data some uncertainty still exists with respect to horizontal stresses against a driven pile. The point angle of the pile vs. the dilatometer, the axi-symmetric pile displacement vs. the approximate plane strain for the dilatometer, the driving of the pile vs. the quasi-static pushing or different-driving of the dilatometer, and the stress distribution along the pile vs. that measured only at one point on the dilatometer, all complicate transferring dilatometer results to the pile. Also, the pile material may not match the frictional character of the smooth steel of the dilatometer and thus require another correction. Despite these complications, and perhaps others not noted, making an actual dilatometer measurement of horizontal stress and attemping a correction for the above noted effects, if needed, should usually give superior results compared to the current common practice of just assuming a horizontal stress coefficient for static pile analyses. Using dilatometer data should reduce the uncertainty.

Pile Bearing Capacity Prediction

It is now known that the in situ horizontal stress conditions play a dominant part in determining the CPT bearing capacity in sands, and indeed all soils[1]. Fig. 2 shows how q_c predicted from the D&M theory increases with increasing K. The D&M theory has the possibly unique capability of including horizontal stress effects in evaluating bearing capacity from given strength parameters, or vice versa. All penetration tests, including displacement piles, have their bearing capacity strongly influenced by the in situ horizontal stress conditions. Knowing these conditions at least in principle allows a more accurate calculation of pile support capability—preferably using the same D&M theory as used to establish the design strength parameters from one of the aforementioned penetration tests.

Horizontal Subgrade Reaction Coefficient (k_h) and Pile Deflections

The DMT also gives the engineer a direct model determination of k_h. The aforementioned 7 mm horizontal plate expansion into the soil increases horizontal stresses from the K_o value to the A-reading value. The DMT thus gives us the values for a stress increment required to produce a displacement increment. Dividing stress by displacement produces the k_h value appropriate to a 94 mm wide plate. The engineer can then use the conventional Terzaghi formula to extrapolate to wide square or round piles. A number of easily available references explain how to use k_h values in pile deflection problems, for example Poulos and Davis (Ref. 15, pp. 164-177).

Pile Settlements and Deflections by Elastic Methods

Since the availability of the Poulos and Davis book[15] it has become increasingly popular to estimate the settlement and deflections of single piles, and pile groups, using suitably modified elastic methods. Chapters 5 and 6 in this book discuss settlement calculations; Chapter 8 discusses horizontal deflection calculations using elastic methods. The biggest problem when using these methods involves determining an appropriate equivalent elastic modulus for each of the significant soil layers. The difference between the B- and A-readings in the DMT, which produces the membrane deflection of 1 mm, gives directly an in situ modulus measurement that the engineer can relate, by suitable correlations developed by Marchetti[14], to in situ modulus and compressibility behavior. The engineer can use such DMT-determined moduli directly in the Poulos and Davis elastic methods. Note that the Menard pressuremeter only produces a "pseudo-elastic" modulus which usually has a significantly lesser value than the soil's equivalent elastic modulus.

EXAMPLES FROM THE SKYWAY BRIDGE INVESTIGATION

Scope of Investigation

In 1980 one of the two existing parallel Sunshine Skyway Bridges, spanning five miles across the mouth of Tampa Bay and owned by the Florida Department of Transportation, collapsed as a result of impact from a storm-driven freighter. In 1982 the Florida DOT began a fast-tracked program to design and construct a replacement structure with a world-record segmental-concrete main span of 1,200 ft and a channel vertical clearance of 175 ft designed by Figg & Muller, Inc., Tallahassee, Florida. Schmertmann & Crapps, Inc., together with Figg & Muller, Inc., completed the geotechnical investigation and foundation design in about one year. This included the main piers with a maximum vertical and horizontal design static load of about 40,000 and 6,000 tons respectively, and more than 300 transition and trestle piers, to support four lanes of traffic designed to conform to interstate standards. The total geotechnical investigation cost about $3 million, roughly divided equally between a driven pile and a drilled shaft load test program

Layer Nos. New	Layer Nos. Previous	General Soil Type	ave γ pcf used	design equations for $K_o, c_{tsf}, \phi^o = f(N, Z')$
1.	1, 2, 3, 4	SAND, sand/shell, shell hash, silty sand, cemented sd.	110	$K_o = 0.736(\frac{Z}{N})^{-0.681}$ (0.3 min) $C_{tsf} = 0$ $\phi^o = 38.60 - 2.220 \ln(\frac{Z'}{N})$ (45° max)
5.	5, 6, 7, 8, 14, 15, 16	SAND/CLAY mixes, clayey sand, sandy clay, lyrs. sand in clay or clay in sd. silts, clayey silts	120	$K_o = 2.798 - 0.840 \ln(\frac{Z'+20}{N})$ $c_{tsf} = 0.498 - 0.237 \ln(\frac{Z+20'}{N})$ $\phi^o = 34.61 - 2.747 \ln(\frac{Z+20'}{N})$
10.	9, 10, 11, 12, 13	CLAYS, silty clay, lightly cement. clay, clays with only minor sand,	115	$K_o = 3.385 - 0.754 \ln(\frac{Z+100}{N})$ $c_{tsf} = 2.074 - 0.649 \ln(\frac{Z+100}{N})$ $\phi^o = 30.31 - 2.840 \ln(\frac{Z+100}{N})$
20.	17, 18, 19, 20	ROCKS, limestones, sandstones, highly cemented, sands, calcareous clays	130	$K_o = 7.495 - 6.594(\frac{Z+60'}{N})$ (min = 1) $c_{tsf} = 0.416 - 5 \ln(\frac{Z+60'}{N})$ (min = 1) $\phi^o = 36.36 - 3.510 \ln(\frac{Z+60'}{N})$ (min = 36°)

Z = depth from mudline ft
N = ave. corrected SPT blowcount

TABLE 3 Soil Layers Used for Transition and Trestle Pier Designs.

and a soil exploration program that included more than 150 borings, a very large in situ testing program, and a small laboratory testing program.

The in situ test program, done from barges in up to 35 ft of open water, involved the extensive use of penetration tests. About 2,000 SPTs and 1,000 DMTs were performed. Because the deep foundation bearing layers included primarily very hard clays and calcareous clays with irregular limestone inclusions and strengths, the CPT or CUPT were not tried in such strong and variable soils.

The penetration tests were supplemented with in situ borehole tests. Williams & Associates, Inc. of Clearwater, Florida, performed 150 Menard PMTs under our direction, primarily under the main piers and the adjacent major transition piers. Geotechnical Test Systems, Inc., of Ames, Iowa, performed about 50 borehole shear tests at similar locations. Dr. Richard Handy acted as our consultant for the performance and interpretation of these shear tests.

Analyses of Driven Pile Capacity and Settlement

Two methods of analysis were used based primarily on the penetration test (SPT) results and then compared with the "ground truth" revealed by the load test program. In this program 13 driven piles of various types were taken to failure or near-failure (the program also included five tests on drilled shafts). The two methods, used as a check on each other, had some similarities and some differences.

The first, called "Method B," used only the SPT N-value results in each of four soil types: sands, sand-silt-clay mixes, clays, and limerocks. We then used the empirical correlations for end bearing resistance and side friction presented in the Florida DOT Research Bulletin 121[16]. The second, "Method A" (Or "PCAP" method for the computer program), also used the very large number of SPT N-values for reference, together with a similar set of four soil types. But the PCAP method of analysis for pile capacity involved the theoretical calculation of bearing and friction. As it turned out, both methods predicted pile capacity with about the same reliability. While satisfying from the point of view of the two methods of analysis checking each other, we believe this result somewhat coincidental because the site soils proved exceptionally variable and this site variability probably set limits on the reliability of any method of analysis. This empirical method seems to work acceptably well in Florida conditions and can be used easily. It may not work as well in other geologic conditions. We believe the theoretical PCAP Method-A has much more general validity and has the potential for even better accuracy than demonstrated in this project. The subsequent discussion relates to the PCAP method.

Table 2 summarizes the steps used to help assign the pile capacity design soil properties of K, c and ϕ needed in the PCAP method. It was desirable to somehow make a rational use of the large volume of N-value data obtained in the investigation. From other in situ and some lab tests better designed for the purpose, it was known just what K, c and ϕ values seemed appropriate for some of the soil layers found at the Skyway site. N-values were available for these layers. The method outlined in Table 2 was used as a rational basis for converting N-values into appropriate K, c and ϕ values. Note that the

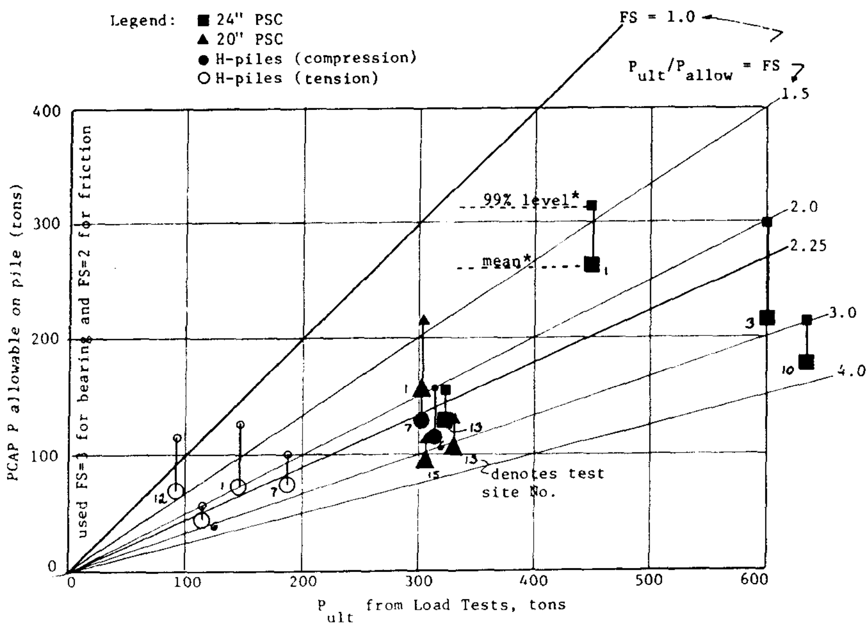
TABLE 4

STEPS IN THE "PCAP" METHOD OF PILE CAPACITY ANALYSIS

1. Choose an SPT boring log representative of the location of the pile-support bridge pier (usually the closest boring(s)).
2. Correct *all* N-values to the reference ENTHRU, giving N' values.
3. Divide the boring log into the four soil types developed in Table 3.
4. Further subdivide the log into layers within each soil type so as to readily assign an average N' to each layer. This often produced 5' deep layers because of the 5' spacing between N-values in the borings and the often abrupt variability in N-values.
5. Use the equations developed for Table 3 to assign a K, c and ϕ value appropriate to each layer.
6. Use the D&M theory to calculate the bearing capacity of a pile (or drilled shaft) as if the entire boring consisted of the same soil as that layer. Use appropriate modifications for different pile materials shapes, and methods of installation.
7. Use the layering adjustment procedure (based on Dutch CPT methods and noted/referenced in text) to obtain pile bearing capacity in the actual layered system at each tip elevation of interest.
8. After estimating appropriate K_p, c_p and ϕ_p values along the pile/soil interface after installation, based on the assigned K, c and ϕ values *within the soil before the installation, calculate the pile friction resistance* in each layer. (K generally increases, and c and ϕ decrease as a result of the pile.)
9. Make a reduction-adjustment of the pile friction in the bearing layer similar to the layering adjustment made in the end bearing resistance. Do this for each tip elevation of interest.
10. Total all side friction resistances for layers above the tip. Divide by the factor of safety for friction to get the allowable design value.
11. At each tip elevation of interest add the end bearing at that elevation, divided by the factor of safety for the allowable design end bearing.
12. Add the allowable end bearing and side friction and subtract the self-weight of the pile. The total equals the allowable vertical load of that pile. Work this out for a range of tip elevations to get the optimum combination of allowable capacity and required length.
13. Adjust, as required, for special considerations such as group action, consolidation settlement, negative skin friction, transient loads, special installation methods, etc.

rational basis for establishing the layer design properties involved the use of the aforementioned D&M bearing capacity theory, which properties were used later with the same D&M theory for the calculation of pile capacities—in this way minimizing any errors associated with the theory itself.

The Table 2 procedure was applied only to those layers where there was other information data as well as SPT data. Thus, in these layers reasonable combinations of depth = Z, K, c and ϕ could be matched with the N' values actually measured. This in turn allowed us to establish site-specific empirical equations to determine K, c and ϕ appropriate to any soil type and any N' value for use over the majority of the site where only Z and N' values were known. *Table 3 shows final results, as developed for the transition and* trestle pier designs for which there was a variety of driven piles among the foundation alternates. For organizing purposes, the site layering was initially *divided into 20 layers based on both soil description and N-values.* This was simplified into only four based on soil type, as shown in Table 3. The equations in Table 3 then took care of N-value differences within each type.



*NOTE: Including soil variability in the PCAP calculations for the allowable P produces a probability distribution for P. The smaller point labelled "99% level" shows the P for which 99% of the calculated Ps in that distribution fall below the point. The larger point, labelled "mean", shows a lesser value of P where 50% of the Ps fall below.

FIGURE 4 Comparisons of PCAP (Method A) Predicted Values For Allowable Pile Capacity at Mean and 99% Probability (Rosenbleuth Method) With Load Test Values of Ultimate Pile Capacity and the Conventional Factor of Safety.

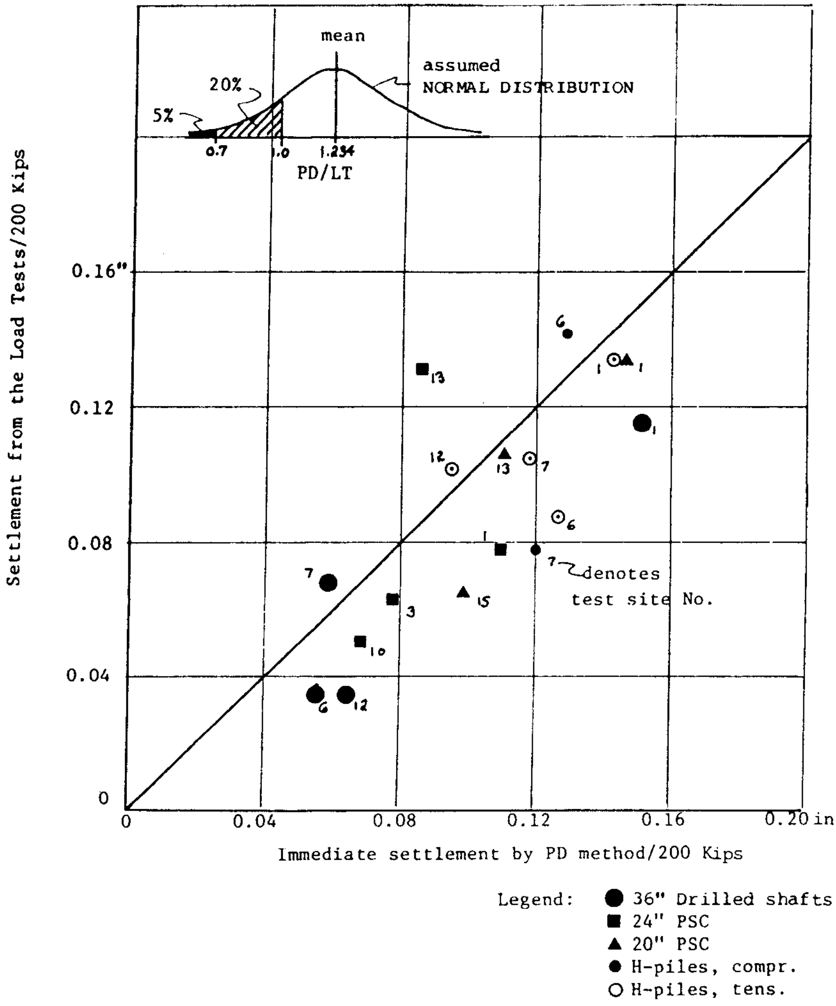


FIGURE 5 Comparison of Predicted (PD Method [15]) and Load Test Immediate Settlements in Allowable Load Range.

Pile Capacity: The PCAP theoretical approach was applied to the calculation of side friction and end bearing for the actual piles. Table 4 lists the steps. A five-layered simplification (up to two below the tip layer, the tip layer, and up to two above) was used for the layer system analysis, developed by the Dutch for the CPT (described in Refs. 5, 10, 11, 16, 17, 19 and noted above under "Statics of the SPT").

Pile Settlement: The DMT provided most of the elastic modulus values for the pile settlement calculations using the Poulos-Davis methods. Elastic and non-linear finite element model studies of the settlement of single piles and pile groups was made. The various in situ penetration tests provided data to help set up the soil layer properties used in these models. However, these models fall outside the scope of this paper.

Fig. 4 shows the bottom-line results between Method-A predicted pile capacities for the load tested piles vs. the capacities measured. Fig. 5 shows a comparison between the computed and measured "elastic" pile settlement in the design load range. Figs. 4 and 5 present these results in a probabilistic manner, as suggested by consultant Dr. Milton Harr. It was concluded that there was a 99 percent probability that actual ultimate pile capacity would exceed design requirement and an 80 percent probability that using the Poulos-Davis elastic settlement analysis will not underpredict the settlement of single piles. These probabilities were considered adequate, based on the current state-of-practice with respect to the use of probabilistic methods. Note that actual design was with the conventional deterministic safety factors of two for side friction and three for end bearing. The probabilistic study was made as an additional check.

CONCLUSIONS

From knowledge and use of new and improved-old in situ penetration tests, and experience with using such tests to design and predict pile foundation performance, the authors have the following conclusions:

1. Better understanding of the statics and dynamics of the SPT will permit improved pile design installation and performance predictions based on this well established and cost effective in situ penetration test.

2. The Dutch cone penetration test already has an excellent reputation for the prediction of driven pile capacities. The new introduction of piezocone (CUPT) now permits a better assessment of the effects of soil layering detail on pile performance, as well as an evaluation of probable pore pressure and freeze effects resulting from driven pile displacements.

3. The new Marchetti dilatometer test offers an economical and rugged in situ penetration test that provides information about horizontal stresses, soil strength and soil modulus to use for both ultimate capacity calculations and vertical and horizontal movement calculations.

4. An extensive pile load test program at the Skyway Bridge Replacement site allowed checking some of the methods discussed herein against field performance, with favorable results.

5. It is believed that in situ penetration test results for the analysis of pile capacity and movement, combined with wave equation analysis for pile driving and confirmation of pile capacity, represents the state-of-the-art at this time.

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