02 Jun 1993, 9:00 am - 12:00 pm

Analysis of Small Pipe Piles Using the Field Vane

G. A. Miller

University of Massachusetts, Amherst, Massachusetts

A. J. Lutenegger

University of Massachusetts, Amherst, Massachusetts

Proceedings: Third International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, June 1-4, 1993, Paper No. 1.39

Analysis of Small Pipe Piles Using the Field Vane

G. A. Miller
Research Assistant, Department of Civil Engineering, University of Massachusetts, Amherst, Massachusetts

A. J. Lutenegger
Associate Professor, Department of Civil Engineering, University of Massachusetts, Amherst, Massachusetts

SYNOPSIS Results of axial compressive load tests on three small diameter pipe piles driven in a varved clay deposit are presented. Predictions of the axial pile capacity were made using the α-method originally proposed by Tomlinson (1957, 1971) and incorporating undrained strength profiles determined with the field vane. Predicted and measured capacities are compared and discussed in light of the various factors which can affect the outcome such as vane geometry, vane testing procedure and interpretation, pile load testing conditions, and empirical relationships incorporated in predictions.

INTRODUCTION

This paper presents the results of an investigation to evaluate the use of undrained strength values determined from the field vane shear test for design of friction piles in a varved clay. Three small diameter pipe piles were installed in a lacustrine varved clay deposit at the Geotechnical Test Site on the University of Massachusetts (UMASS) at Amherst campus and load tested to failure in axial compression. In addition, an extensive program of field vane testing was performed and involved the use of vanes of different geometry. Standard vane testing procedures were used to obtain undrained shear strength profiles with all of the vanes. Parallel profiles in which the vane was allowed to consolidate at each test depth for 24 hours prior to shearing were also conducted. The purpose of the field vane testing program was to investigate the effects of different test procedures on the resulting undrained shear strength profile and evaluate the influence on the predicted capacities of the piles.

BACKGROUND

Field Vane Shear Test

The field vane test has long been a popular method used by practicing engineers and researchers for determining the undrained strength profile of soft clay deposits. Much research has been dedicated to this topic in the past 40 years and many methods of interpretation have been proposed to obtain soil shear strength from field vane measurements. Some of the best known work was performed by Bjerrum (1972) who presented vane correction factors which were backcalculated from embankment failures and were correlated with plasticity index (PI). Bjerrum suggested that a correction factor be applied to field vane strength values so that predicted behavior matched the observed failure behavior of the embankments. The correction factor varied from slightly greater than 1.0 for PI values less than 20 to as low as 0.6 for PI values on the order of 100.

An extension of Bjerrum's work is reflected in the more recent correction procedure proposed by Aas et al. (1986) which in addition to accounting for plasticity also, considers the geologic age of the deposit involved. Other investigators have proposed modifications to Bjerrum's work by taking into account the three dimensional nature of the slope failures from which the backcalculated correction factors were determined (e.g., Azzouz et al., 1983).

For computing undrained strength from the field vane test some investigators have suggested that the assumed shear stress distributions on the resulting failure surface created by the vane should have a more parabolic or triangular shape instead of the commonly assumed rectangular and uniform distribution (e.g., Donald et al., 1977). This appears to be especially applicable for the horizontal end surfaces and for work softening soils. Skempton (1948) suggested the effective diameter of the cylindrical failure surface is actually slightly larger (≈5%) than that formed by the vane. This was corroborated somewhat by radiograph work done by Arman et al. (1978), which revealed a considerable zone of disturbance around the vane.

Obviously, the choice of whether or not to apply a correction factor to field vane strengths can pose quite a dilemma, especially in highly plastic soils and when using undrained strength values for limit equilibrium problems other than embankment stability. That is, it may not be appropriate to use vane correction factors backcalculated from embankment failures for use in design with piles because of the dissimilarity in the volume of soil and kinematics involved in these two geotechnical situations.

Undrained Analysis of Piles

One of the most popular methods for determining the skin friction capacity of driven and bored piles in cohesive soils is the α-Method originally proposed by Tomlinson (1957, 1971). In this method, the unit skin friction mobilized along the pile shaft, f_s, is determined using the soil undrained shear strength, s_u, multiplied by an empirical coefficient α, defined as:

$$\alpha = f_s/s_u \quad (1)$$

where: $$f_s = (L_f - 9s_uA_e)/A_s \quad (2)$$

L_f = load at failure,

A_e = area of the pile tip,

A_s = area of the shaft.

The value of α has been determined for various types of clay by different researchers (e.g. Tomlinson, 1957; Drewry et al., 1977; Semple and Rigden, 1984). In general, α was obtained by backcalculating f_s from interpreted pile load tests used in combination with measurements of the average undrained strength values over the pile length in question. Recently, it has become more common to correlate α to the average normalized undrained strength, s_u/σ'_{vo} (e.g., Randolph and Murphy, 1985) or to OCR as originally suggested by Wroth (1972). It is important to note that since undrained shear strength is not a unique soil property the resulting backcalculated α value will be dependent on the method used to determine s_u, the pile type, the pile load test procedure (fast or slow) and on the method of interpretation used to obtain the pile failure load. Because α is usually based on the average s_u calculated for the entire pile length, α will also depend somewhat on the pile length due to the load transfer and progressive failure that may occur along the pile shaft. This could be particularly significant to the resulting α value in situations where the soil is work softening, the undrained strength decreases or increases dramatically with depth and/or the piles are long.

Although there are a number of uncertainties associated with the α-Method, this procedure has proven to be very popular as evidenced by its widespread use, and shows up in nearly all foundation engineering texts. This is probably due in part to its simplicity and well established history. However, it should be noted that where empirical correlations are used, the reliability of associated design methods should be questioned particularly if the correlations are weak and involve a small data base focused on few soil types. For pile analyses in the present study, standard and common procedures for determining s_u from the field vane and α from the literature were used. This was done in the interest of practicality and to keep in line with procedures that are routinely used in practice.

TEST SITE CHARACTERISTICS

The study was conducted at the University of Massachusetts Geotechnical Test Site in Amherst, Massachusetts which is located in the Connecticut River Valley of western Massachusetts. The site is underlain by approximately 25 m (80 ft.) of lacustrine varved clay deposited into Glacial Lake Hitchcock during the past ice age. This deposit is considered to be geologically young (<10,000 yrs.) and is known locally as Connecticut Valley Varved Clay (CVVC). The thickness of individual varves is on the order of 2 to 8 mm.

A surficial layer of approximately 1.2 m (4 ft.) of mixed cohesive and sandy compacted fill is present above the CVVC deposit over most of the site. As shown in Figure 1, the CVVC deposit is made up primarily of silt and clay and has an overconsolidated crust. OCR values, determined from standard incremental loading oedometer tests on 76 mm diameter piston samples, range from approximately 12 at a depth of 1.5 m (5 ft.) to about 1.2 at a depth of approximately 10.5 m (34 ft.) and below. The mechanisms responsible for the overconsolidation in the crust include fluctuations in the water table, erosion of overburden and chemical weathering.

The CVVC is moderately plastic as shown by the Atterberg Limits profile and the natural water content reflects the increasing liquidity index and decreasing OCR of the soil. The undrained strength profile determined from the field vane and shown in Figure 1, is high near the top of the crust (≈100 Kpa, 2.08 ksf) but decreases rapidly to approximately 30 kPa (0.63 ksf) at a depth of 6 m (20 ft.). Between 6 m (20 ft.) and 18.3 m (60 ft.) the average undrained strength shows only a slight and erratic increase to a value of approximately 40 kpa (0.84 ksf) at a depth of 18 m (60 ft.).

The similarity in the shape of the OCR and s_u profiles results in a deposit which essentially exhibits normalized behavior. This behavior proved helpful in extrapolating undrained strength values in the upper part of the stiff crust where use of the standard field vane was not possible. As shown on Figure 1, the extrapolated values of s_u in the crust appear reasonable, however, additional testing in the crust is needed to corroborate the extrapolated strengths.

FIELD VANE TESTING AND ANALYSIS

Field Vane Testing

A field vane testing program was conducted at the test site to evaluate undrained shear strength. Initially, tests were performed using a Nilcon Vane Borer with a 130 mm x 65 mm (H/D=2:1) vane having rectangular blades of 2.0 mm thickness. Tests were performed according to ASTM Standard D 2573, at four different locations. These four profiles were used to establish baseline undrained shear strength values to which all other special field vane test results are compared. In addition, standard test procedures were used to perform field vane tests using a 3:1 vane of the same diameter and blade thickness as the standard vane and a 2:1 vane of the same diameter but with thicker blades. Table 1 gives the dimensions of the three vanes used in the field investigation.

Once the standard test procedure was used to obtain strength profiles for the three vanes, another profile was obtained with each vane using a special test procedure. This special procedure allowed for a consolidation time of 24 hours between installation and shearing. The purpose of using this procedure was to allow excess pore water pressures, generated during

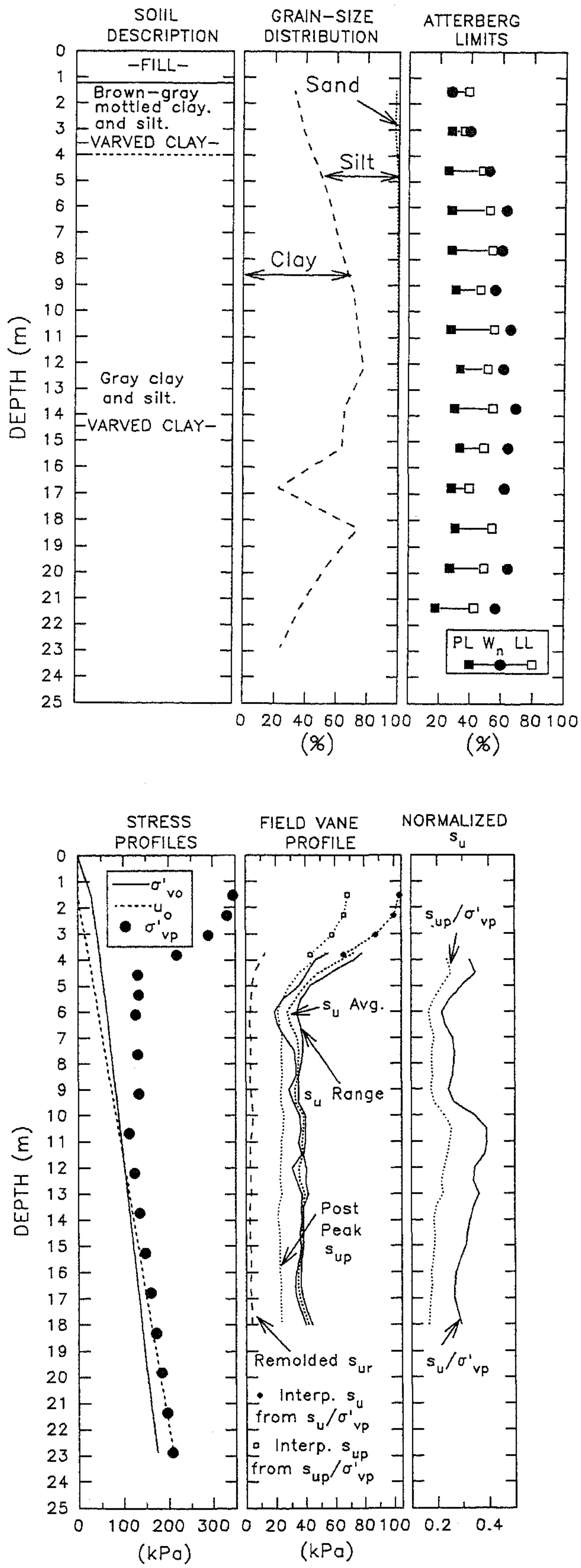

Figure 1. Site Soil Properties

Table 1. Field Vane Dimensions

Vane Type	Height, H (mm)	Diameter, D (mm)	H/D	Thickness, t (mm)
2:1 Standard	130	65	2	2.0
2:1 Thick	130	65	2	3.2
3:1 Standard	195	65	3	2.0

installation of the vane, to dissipate in order to evaluate how the resulting undrained strength profile would be affected.

The vane was advanced using 18 mm (0.75 in.) diameter rods which were pushed with the UMASS Mobile Hydraulic Pushing Rig. Immediately above the vane a special slip couple was attached to the rods which allowed for the determination of the rod friction for each vane test. The slip couple allows the rods above the vane to turn approximately 15° before the vane is engaged and this leaves a clearly discernible feature on the recording trace paper.

Each field vane test was performed to allow the determination of the peak and remolded vane strengths as well as the post peak strength. The remolded strength was obtained after first rotating the vane through ten complete 360° revolutions.

Figure 2 shows a typical trace obtained with the vane borer from a single field vane test. Measurements corresponding to the peak, post peak and remolded strength are indicated. The trace shown reveals the brittle nature of the CVVC at the test site as a drop from the peak value to a value which represents the post peak undrained strength of the soil. It may be reasoned that this post peak strength is the component of the peak undrained strength which does not depend on the initial structure of the soil but is greater than that which would be obtained from complete remolding of the soil fabric. Such structure may be due partly to a flocculated particle geometry or cementation.

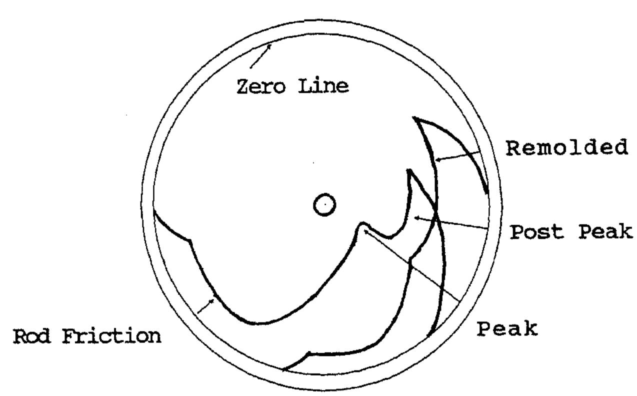

Figure 2. Typical Trace from the Vane Borer

For the CVVC it is postulated that the structured nature of the deposit is mainly the result of carbonate cementation and the layered structure of the varves themselves. In light of this structured behavior, for design of displacement piles installed in brittle soils it may be more appropriate to use the post peak undrained strength since installation of such piles partially or completely destroys the initial structure of the soil.

Field Vane Analysis

All of the field vane tests were analyzed using common procedures as outlined in ASTM Standard D 2573. The distribution of shear stresses on the right cylindrical failure surface formed by the vane were assumed to be uniform and rectangular. As mentioned previously, based on the results of finite element analyses of a vane in elastic and perfectly plastic mediums, Donald et al. (1977) have suggested that for work softening soils, such as CVVC, such an assumption may be inappropriate. For a 2:1 rectangular vane it was suggested that the shear stress distribution on the ends is closer to a parabola or rectangular shape, however, the rectangular distribution was assumed in this study because it appears to be the most commonly used in practice. A comparison was made by performing simple calculations of s_u for a 2:1 vane with the assumption of both rectangular and triangular end shear distributions. The resulting discrepancy in the calculated undrained strength was on the order of 10 % which is significant, however, the rectangular distribution gives the smaller value of s_u and hence is more conservative.

For the range of plasticity indices shown in Figure 1, the suggested correction factors (Bjerrum, 1976; Aas et al. 1986) vary from slightly less to slightly greater than 1. Therefore, it was felt the correction factors would not appreciably affect the undrained strength values and so were not incorporated in the pile capacity predictions. Figure 3 shows the undrained shear strength profiles resulting from the field vane testing program.

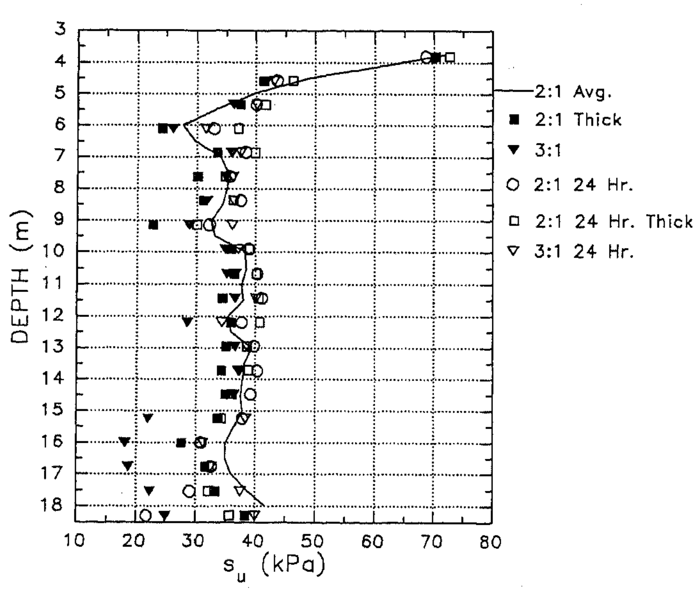

Figure 3. Field Vane Strength Profiles

PILE INSTALLATION AND TESTING

Pile Installation

Three small diameter pipe piles were installed at the test site using a standard exploratory drill rig. A 7.6 cm (3 in.) diameter hole was first prebored through the surficial fill to a depth of 1.2 m (4 ft.) so that pile penetration would begin at the top of the overconsolidated clay crust. The piles were then driven in 3.05 m (10 ft.) sections using a 1.3 kN (300 lbs.) casing hammer. Welded splices ground flush to the pile were made between sections. During driving the blow counts were recorded and the amount of plugging was measured for each 0.3 m (1 ft.) of penetration. Table 2 gives the geometry and installation data for each of the three piles used in this study.

Table 2. Pile Installation Details

Pile Number	Outside Diameter, D (mm)	Inside Diameter, ID (mm)	Penetration Depth, P (m)	Plug Length (m)
2	60.3	55.0	3.05	1.21
4	60.3	55.0	7.62	2.86
5	60.3	55.0	10.67	4.38

Pile Load Testing

Pile load tests were conducted approximately 1 year after installaion. Load testing was conducted using a procedure similar to that outlined in ASTM Standard D 1143 under the "Quick Load Test Method". Axial compressive loads were applied to the pile butt using an Enerpac 220 kN (25 ton) single acting hydraulic jack. The load was monitored with a Geokon 3000-300-2 Load Cell connected to a Measurements Group P-3500 Strain Indicator, which together gave a resolution of approximately 70 N (16 lbs).

Deflections at the pile butt were monitored using three dial gages with a resolution of 0.025 mm (0.001 in.) which were placed 120° apart and equidistant from the pile center. Static axial loads were applied to the pile butt in increments equal to approximately 5 to 10 % of the pile capacity and deflections were monitored immediately after and at 2, 5, 10 and 20 minutes following application of a load increment. The reaction to loading was provided by two drilled anchor shafts which supported a steel I-Beam. A ball and socket swivel device was placed between the beam and the loading jack to minimize eccentric loading on the pile butt.

Load/displacement curves for first time monotonic compressive loading of the three piles are shown in Figure 4. The failure load or ultimate capacity was interpreted as the load at the point of intersection between the initial and final tangents of the load displacement curve as indicated in Figure 4. According to Kulhawy and Hirany (1989) this method of interpretation is a reasonable one for cases where the transition from linear to non-linear behavior is small and the curves rapidly approach an asymptotic value, as is the case for the three piles tested for this study. Table 3 gives the interpreted failure loads and displacements at failure for the three piles tested.

Table 3. Interpreted Pile Load Test Results

Pile Number	Failure Load, L_f (kN)	Failure Deflection, δ_f (mm)
2	14.6	1.22
4	21.9	2.86
5	29.0	2.20

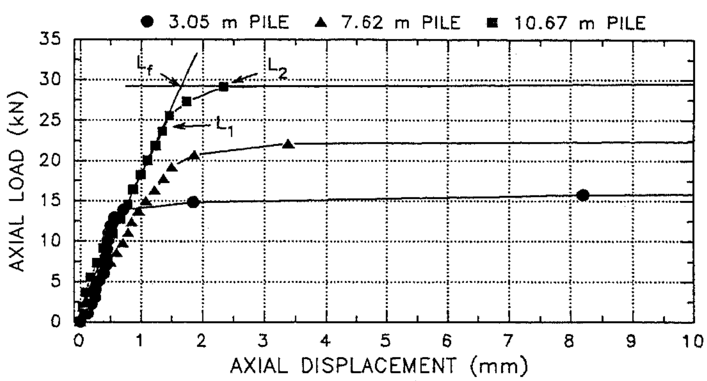

Figure 4. Pile Load Test Results

PILE ANALYSIS AND PREDICTIONS

Pile Analysis

Assuming undrained conditions ($\phi'=0$), the skin friction at failure along a pile with a constant outside diameter, D, can be estimated using the equation:

$$F_s = \pi D \Sigma f_{si} l_i \qquad (3)$$

where:

F_s = the total skin friction force developed on the pile at failure,

f_{si} = αs_{ui} is the unit skin friction developed along pile segment i,

s_{ui} = is the average undrained strength of the soil along segment i prior to pile installation,

α = is an empirical factor and,

l_i = is the length of pile segment i.

Although, there are a number of foundation engineering texts with charts available for selecting α (e.g. Poulos and Davis, 1980; Bowles, 1988; Fang, 1991), for this paper α was interpreted from the popular "Foundation Engineering Handbook, Second Edition" (Fang, 1991, Figure 18.14). The α values reported here are recommended by the American Petroleum Institute (API) for offshore structures and are the result of work done by Randolph and Murphy (1985). This α correlation was selected because it is conservative relative to other correlations and has become well accepted. α values are given as a function of the soil undrained strength normalized by the effective overburden pressure. The equations for α recommended by API are:

$$\alpha = 0.5(s_u/\sigma'_{vo})^n \qquad (4)$$

where:

s_u = the initial soil undrained shear strength,

σ'_{vo} = the initial overburden pressure,

$n = -0.25$ for $s_u/\sigma'_{vo} > 1$ and

$n = -0.50$ for $s_u/\sigma'_{vo} \leq 1$.

The end bearing for the piles used in this study was calculated as 9 x s_u multiplied by the end area of the pile and assuming the soil plug was an integral part of the pile. Thus, for an open, circular pile the end area is $\pi D^2/4$ where D is the outside diameter of the pile. Observations made of the soil plug before and after the load tests indicated no plug movement which supports the assumption that the entire end area of the pile contributed to base bearing capacity.

Pile Ultimate Capacity Predictions

The results of the predictions for the ultimate capacity of the three piles used in this study are presented in Table 4 along with the measured capacities interpreted from the load-displacement curves. All predictions were computed using the average undrained strengths of the four standard 2:1 vane profiles. Predictions of the end bearing capacity and skin friction were made using peak undrained strengths and skin friction was also predicted using the post peak undrained strength. Predictions of skin friction were made by: 1) summing incremental values of skin friction along the pile shaft using local α and s_u values; and 2) calculating a global skin friction using the average undrained strength and vertical effective stress at the pile midpoint. Differences in incrementally computed and global predictions were negligible. The predictions reported in Table 4 are those computed from incremental values of skin friction.

Estimates of end bearing for the three piles were very low as compared to the skin friction and were therefore neglected in the predictions. Field load tests of cone tips on adjacent piles, at the same depths as the pile tips in this study, have verified these predicted low capacities. Furthermore, the cone tip load tests showed that the ultimate tip capacity is mobilized at displacements much larger than the butt displacements measured at failure for the piles in the present study. These observations suggest that elimination of the ultimate tip capacity from the predictions is justified for

Table 4. Comparison of Predicted and Measured Pile Capacities

Pile Number	Q_t (kN)	L_f (kN)	Q_u (kN)	Q_u^* (kN)
2	1.4	14.6	20.4	15.1
4	0.9	21.9	41.3	32.8
5	0.9	29.0	57.9	46.4

Q_t = predicted ultimate tip capacity

L_f = interpreted load at failure

Q_u = predicted ultimate pile capacity using peak s_u

Q_u^* = predicted ultimate pile capacity using post peak s_u

this study, however, for cases where the piles end in stiffer material this may not be appropriate.

As shown in Table 4, the capacities were significantly overpredicted using the peak undrained strength values and the overprediction was more severe for the longer piles. The predicted capacities based on post peak undrained strengths were much better especially for Pile 2, however the amount of overprediction was still significant for Piles 4 and 5. The amount of overprediction using both post peak and peak undrained strength measurements appears to be a function of the penetration depth of the piles. Behavioral dependence of friction piles on penetration depth has been observed by other researchers as well (e.g., Tomlinson, 1971; Vijayvergiya and Focht, 1972; Janbu, 1976; Meyerhof, 1976; Flaate and Selnes, 1977).

DISCUSSION OF RESULTS

The selection of the appropriate undrained strength profile to be used in the prediction of pile capacity is extremely important in light of the many uncertainties involved in the empirical method of analysis. Figure 3, which presents the undrained strength profiles determined with different vanes and procedures illustrates the problem. It can be seen that by allowing a consolidation period following vane installation, the resulting strength values over most of the profile are higher than the average values obtained by standard test procedures. In the authors' opinion, these consolidated strength values more accurately depict the "true" in situ undrained strength of the soil. In general, an average increase of about 12 % over the standard values is indicated. Other researchers (Torstensson, 1977; Roy and Leblanc, 1988) have shown that this consolidation may provide an increase of as much as 20 % over conventional test procedures.

Additionally, since the insertion of the vane creates an unknown amount of disturbance to the soil structure, the "true" in situ strength may be higher still. The degree of this influence is probably related to the sensitivity of the soil. La Rochelle et al. (1973) have shown that the degree of disturbance can be related to the vane geometry by the perimeter or area ratio. They have demonstrated that an increase of 10-20 % may result when extrapolating to a "zero disturbance" condition. Therefore, the use of a thicker vane should give strength values which are lower than a thinner vane of the same diameter. This trend is shown in Figure 3. However, any correction factors used to increase the field vane strength to account for both disturbance and installation pore pressures would produce an even greater overprediction of pile capacity.

An inherent problem with the use of an empirical approach to design, in this case the α-Method, is that the data base used to formulate the method may make use of different testing techniques to arrive at the proposed correlations. Tomlinson (1957) primarily used the results of unconfined compression, UU, and quick triaxial compression, QU, tests to provide estimates of the undrained shear strength in the development of the method. The authors are aware of only limited attempts to refine the α-Method by isolating the techniques used to obtain s_u (e.g., Dennis and Olson, 1984). Interestingly, Bjerrum (1973) presented a correlation between α and s_u obtained from the field vane. Since the field vane normally gives strength estimates which are higher than either UU or QU tests in soft clays, a corresponding lower value of α would be needed to match pile load tests. This was noted by Dennis and Olson (1983) who found that piles with the largest values of Q_{calc}/Q_{meas} (calculated/measured pile capacity) involve the use of either field vane tests or laboratory tests on high quality samples. It is interesting to note that in this study the API correlation worked well using post peak s_u for the shorter pile which is completely embedded in the stiff clay crust. This result is contrary to the longer piles which have a considerable portion of their embedded length in the soft clay. Based on the previous discussion, this may be attributed to the fact that the field vane values in stiffer material more closely match UU strengths which were used to develop the correlation. The results of pile load tests presented in this paper suggest that additional work is needed to develop accurate α factors for undrained strengths determined by field vane tests. For the CVVC in this study, a substantial reduction in existing α factors would be required in order to accurately predict the capacity of piles which penetrate the softer CVVC.

The fact that the API correlation used does not provide satisfactory results for all the predictions made in this study may be due to several factors. First, the dimensions of the test piles were considerably smaller than those used to establish the API data base. Secondly, the API correlation was developed for offshore piles primarily using undrained strength values determined from unconfined compression tests (Semple and Rigden, 1984). Thirdly, the API correlation is based primarily on marine clays and may not be suitable for lacustrine varved clays. Finally, the scatter of the data (Randolph and Murphy, 1985) used to establish the API α equations contributes to the uncertainty inherent in this method.

CONCLUSIONS

As part of an ongoing study being conducted at the University of Massachusetts, the α-Method of analysis utilizing the well known API empirical relationships was evaluated. This was accomplished using field vane undrained strengths and load test results from three small diameter piles installed in the Connecticut Valley Varved Clay. From the results of the investigation, the following conclusions can be drawn.

1.) The ultimate pile capacities interpreted from compression load tests performed on three test piles were significantly lower than predicted capacities using peak undrained strength values determined from conventional field vane tests and suggested α values.

2.) Using the post peak undrained strength from the same vane profiles, the predicted capacities were closer to measured values but still significantly greater than the measured capacities for the two longer piles.

3.) The predicted capacity, based on post peak undrained strengths, for the shortest pile matched the capacity interpreted from the pile load test. Thus, for the shorter piles embedded in the stiff CVVC crust, using the α-Method with post peak undrained strengths and API α values, provided a good estimate of the actual capacity. However, more data from other pile load tests are required to substantiate this conclusion.

4.) For the CVVC which exhibits brittle behavior, use of the post peak undrained strength determined from the field vane produced better predictions for the piles studied. It is reasonable that the post peak field vane strength is more representative of the mobilized strength along the pile. Use of the post peak strength may account for the loss of the component of shear strength, derived from the initial soil structure, that accompanies the installation of displacement piles.

5.) To accurately predict pile capacity, a refinement in the α-Method of pile analysis is needed in order to account for the test method used to evaluate the soil undrained shear strength.

LIST OF SYMBOLS

α = correlation coefficient
A_e = pile end area
A_s = pile shaft area
D = pile or vane diameter
δ_f = pile failure displacement
F_s = shaft friction force
f_s = unit shaft friction
H = vane height
ID = pile inside diameter
l = incremental shaft length
L_f = interpreted pile failure load
LL = liquid limit
n = correlation exponent
OCR = overconsolidation ratio
P = penetration length
PI = plasticity index
PL = plastic limit
Q_t = pile ultimate tip capacity
Q_u = pile ultimate capacity
σ'_{vo} = vertical effective overburden pressure
σ'_{vp} = vertical preconsolidation pressure
s_u = undrained shear strength
s_{ur} = remolded undrained shear strength
s_{up} = post peak undrained shear strength
t = vane blade thickness
w_n = natural water content

REFERENCES

Aas, G., Lacasse, S., Lunne, T. and Hoeg, K. (1986), "Use of In Situ Tests for Foundation Design on Clay," Proc. In Situ 86, pp. 1-30.

API (1984), "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms," American Petroleum Institute Publication RP-2A, Dallas Texas.

Arman, A., Poplin, J.K. and Ahmad, N. (1978), "Evaluation of the Vane Shear Test in Louisiana," Engineering Research Bulletin No. 118, Division of Engineering Research at Louisiana State University, Louisiana.

ASTM (1992) "Annual Book of ASTM Standards," Vol. 4.08.

Azzouz, A.S., Baligh, M.M. and Ladd, C.C. (1983), "Corrected Field Vane Strength for Embankment Design," Journal of Geotechnical Engineering, ASCE, Vol. 109, pp 730-733.

Bjerrum, L. (1972), "Embankments on Soft Ground," Proc. of the Specialty Conf. on Performance of Earth and Earth-Supported Structures, ASCE and Purdue University, Lafayette, Indiana, Vol. 2., pp. 1-54.

Bjerrum, L. (1973), "Problems of Soil Mechanics and Construction on Soft Clays and Structurally Unstable Soils (Collapsible, Expansive and Others)," Proc. of 8th ICSMFE, Vol. 3, pp. 111-158.

Bowles, J.E. (1988), "Foundation Analysis and Design," McGraw-Hill, Inc., USA.

Dennis, N.D. and Olson, R.E. (1983), "Axial Capacity of Steel Pipe Piles in Clay," Geotechnical Practice in Offshore Engineering, ASCE, S.G. Wright, Ed., Austin, Texas, pp. 370-387.

Donald, I.B., Jordan, D.O., Parker, R.J. and Toh, C.T. (1977), "The Vane Test-A Critical Appraisal," Proc. 9th ICSMFE, Vol. 1, pp. 81-88.

Drewry, J.M., Weidler, J.B. and Hwong, S.T. (1977) ,"Predicting Axial Pile Capacity for Offshore Platforms, Petroleum Engineer, Vol. 41.

Fang, H.Y. (1991), "Foundation Engineering Handbook," Van Nostrand Reinhold, USA.

Flaate, K. and Selnes, P. (1977), "Side Friction of Piles in Clay," 9th ICSMFE, Vol. 1, pp. 517-522.

Janbu, N. (1976), "Static Bearing Capacity of Friction Piles," 6th ECSMFE, Vol. 1.2, pp. 479-488.

Kulhawy, F.H. and Hirany, A. (1989), "Interpretation of Load Tests on Drilled Shafts Part II: Axial Uplift," Proc. of Foundation Engineering: Current Principles and Practices, ASCE, Vol. 2, pp. 1150-1159.

La Rochelle, P., Roy, M. and Tavenas, F. (1973), "Field Measurements of Cohesion in Champlain Clays," Proc. 8th ICSMFE, Vol. 1, pp. 229-236.

Meyerhof, G.G. (1976), "Bearing Capacity and Settlement of Pile Foundations," Journal of Geotechnical Engineering, ASCE, Vol. 102, No. GT 3, pp. 197-228.

Poulos, H.G. and Davis, E.H. (1980), "Pile Foundation Analysis and Design," John Wiley & Sons, Inc., USA.

Randolph, M.F. and Murphy, B.S. (1985), "Shaft Capacity of Driven Piles in Clay," Proc. 17th Offshore Technology Conference, Houston, Texas, OTC 4883, pp. 371-378.

Roy, M. and Leblanc, A. (1986), "Facteurs Influencant la Mesure au Scissometre," Bulletin de Liaison des Laboratoires des Ponts et Chaussees, France, Vol. 144, pp. 25-35.

Semple, R.M and Rigden, W.J. (1984), "Shaft Capacity of Driven Pipe Piles in Clay," Symposium entitled Analysis and Design of Pile Foundations, ASCE, San Francisco, California, pp. 59-79.

Skempton, A.W. (1948), "Vane Test in The Alluvial Plains of River Froth Near Grange Mouth," Geotechnique, Vol. 1.

Tomlinson, M.J. (1957), "The Adhesion of Piles Driven in Clay Soils," Proc. 4th ICSMFE, Vol. 2, pp. 66-71.

Tomlinson, M.J. (1971), Some Effects of Pile Driving on Skin Friction," Proc. of Conf. on Behavior of Piles, ICE, London, pp. 107-114.

Torstensson, B.A. (1977), "Time-Dependent Effects in the Field Vane Test," Proc. of International Symposium on Soft Clay, Bangkok, Thailand, pp. 387-397.

Vijayvergiya, V.N. and Focht, J.A. (1972), "A New Way to Predict Capacity of Piles in Clay," 4th Offshore Technology Conference, Houston, Vol. 2, pp. 865-874.

Wroth, C.P. (1972), Discussion in Spec. Conf. on Performance of Earth and Earth-Supporting Structures, ASCE, New York, New York, Vol. 3, pp. 231-234.