07 May 1984, 11:30 am - 6:00 pm

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SYNOPSIS A data bank containing records from 1000 load tests on driven piles was set up. A computer program was developed to access the data bank and perform capacity analyses using a variety of methods. Analyses using six methods in clay and three in sand are reported here. For piles in clay, the capacities were predicted with tolerable accuracy by all methods, whereas the scatter was large for all methods for piles in sand. Generally, capacities were higher for tapered piles then indicated by the analyses. Tensile and compressive side shear capacities were essentially the same. The capacities of open and closed ended pipe piles were predicted with equal accuracy. Limits on side shear and tip stresses were helpful in reducing overpredictions.

INTRODUCTION

Prediction of the axial capacity of piles is made difficult by a number of factors, e.g., severe remolding of the soil, changes in stresses due to pile installation, dragdown of soil from one layer into another, variations in pile installation procedures, and soil-pile interaction during loading. As a result of these problems, pile capacities are predicted using semi-empirical methods based on case histories. The accuracy of the prediction then depends on the size of the data base and on differences between conditions at the design site as opposed to average conditions existing at the sites of the case histories. If the case history data base is large enough, it may be subdivided in such a way as to minimize these differences and thus improve the accuracy of the prediction.

We have collected data for about one thousand load tests on driven, vertical piles. Loads were both compressive and tensile. The piles were of essentially all types and were installed using a variety of methods. Soils ranged from peat to gravel.

To determine the effectiveness of present static, axial pile capacity predictive procedures, eight of the most frequently employed methods were evaluated using appropriate subsets of our data base. In addition, two new methods were developed and evaluated. In this report, consideration will be restricted to piles in essentially pure clay or sand profiles.

The purpose of this paper is to examine broad aspects of application of these analytical methods. The more detailed considerations will appear in later papers.

ANALYTICAL METHODS

The axial capacity, Q, is:

$$Q = Q_s + Q_p \pm W_p \quad (1)$$

where Q_s and Q_p are the loads transferred to the soil in side shear and end bearing, respectively, and W_p is the weight of the pile (positive for tension, negative for compression). If Q_p is taken as the net tip capacity, then W_p was sufficiently small that it could be ignored.

Piles in Clay

For piles in clay, the tip capacity is estimated as:

$$Q_p = 9\ c_u\ A_p \quad (2)$$

where c_u is the undrained shearing strength and A_p is the tip area. For most piles in clay the tip capacity is less than 10 percent of the total capacity.

Side shear can be calculated using:

$$Q_s = \int f_s\ dA_s = \bar{f}_s A_s \quad (3)$$

where f_s and $\bar{f}_s$ are the local and average side shearing stresses, and A_s is side area. The shearing stresses were calculated using methods shown in Table 1. The present application of the methods is explained in detail in Olson and Dennis (1982). The methods are denoted by a four-character name, e.g., ALP1, which was used in computer programs.

For open ended pipe piles in clay the tip capacity was defined as the tip capacity of the steel tube plus the smaller of the tip capacity and side shear capacity of a full plug.

Piles in Sand

The methods used for piles in sand included the 1981 API standard (APIS), Meyerhof's (1956) method based on standard penetration resistances (STDP) and a quasi-static cone method (CON1) discussed by van der Veen (1953), Meyerhof

TABLE I. Summary of Equations used to Predict Pile Capacities in Clays

Method	Ref.	Equation	Parameters
ALP1	12	$\overline{f}_s=\alpha\ \overline{c}_u$	α=f(material type, shearing strength)
ALP2	13	$\overline{f}_s=\alpha\ \overline{c}_u$	α=f(shearing strength, pile penetration, soil profile)
APIC	1	$\overline{f}_s=\alpha\ \overline{c}_u$	α=f(plasticity, OCR shearing strength)
LAM1	16	$\overline{f}_s=\lambda(\sigma_v+2\overline{c}_u)$	λ=f(pile penetration)
LAM2	6	$\overline{f}_s=\lambda(\sigma_v+2\overline{c}_u)$	λ=f(pile/soil stiffness, OCR)

(1956), van der Veen and Boersma (1957), and Bogdanovic (1961).

In the API method, the unit tip stress (q_p) is:

$$q_p = \overline{\sigma}_{vo}\ N_q \tag{4}$$

where $\overline{\sigma}_{vo}$ is the free field vertical effective stress at the elevation of the tip, and N_q is a bearing capacity factor which is assigned values of 40, 20, 12, and 8 for sand, silty sand, sandy silt, and silt, respectively. The side shear is:

$$f_s = K\ \overline{\sigma}_{vo}\ \tan\delta \tag{5}$$

where K is the earth pressure coefficient (we used K equal to 1.0 and 0.7 for full displacement piles in compression and tension, respectively, and 0.7 and 0.5 for non-displacement piles in compression and tension, respectively) and δ is the pile/soil friction angle, taken as 30, 25, 20, and 15 for sand, silty sand, and silt, respectively. The API specification indicates that limits may be set on these values but no limits are specified.

For the STDP method:

$$q_p = 4N \text{ tsf} \tag{6}$$

and:

$$\overline{f}_s = \overline{N}/50 \leq 1 \text{ tsf} \tag{7a}$$

for full displacement piles and:

$$\overline{f}_s = \overline{N}/100 \leq 0.5 \text{ tsf} \tag{7b}$$

for nondisplacement piles where N is the standard penetration resistance (ASTM D1586-67).

For the CON1 method:

$$Q_p = A_p q_c \tag{8}$$

and

$$f_s = q_c/200 \leq 1 \text{ tsf} \tag{9a}$$

or full displacement piles and:

$$f_s = q_c/400 \leq 0.5 \text{ tsf} \quad (9$$

for nondisplacement piles, where q_c is the co tip resistance.

INTERPRETATION OF LOAD TESTS

Two definitions of failure were used, "plunging" and "defined." Plunging failure occurre when a pile settled greatly and could carry n more load. For all piles in sand and many in clay, no plunging load could be defined becau the load-settlement curve never became vertic For such tests, plunging failure was taken as the maximum applied load provided the load-settlement curve had turned downwards. Defin failure was the force (Q) applied at a pile butt settlement (S) of:

$$S = \frac{QL}{AE} + 0.15 + 0.01D_b \quad \text{inch} \quad (1$$

where AE/L is the pile spring constant and D_b is the diameter of the base (Davisson, 1973). All analyses reported here used capacities at the defined failure point. For piles in clay the "plunging" capacity exceeded the "defined capacity by about 7% (range from 0 to 75%). For piles in sand the maximum applied load ex ceeded the defined failure load by an average of about 14% (range 0 to 52%).

CALCULATIONS AND PRESENTATION

Relevant data from load tests were stored on magnetic tape. A computer program was writte which allowed the user to select data accordi to a number of criteria (direction of loading pile type, pile diameter and length, pile shape, pile taper, pile displacement ratio, type of soil profile, range in strengths of soils, methods used to measure strengths of soils), to calculate the capacities of all accepted piles, to perform statistical calcul tions, and prepare appropriate diagrams. Linear regression analysis was performed for measured (Q_m) versus calculated (Q_c) pile capacities. The ratio Q_c/Q_m was calculated f each load test.

Because the values of Q_c/Q_m are log normally distributed, the mean was first calculated as the mean of $\ln_e(Q_c/Q_m)$ but was converted bac to a natural number for presentation. The standard deviation (σ_ℓ) was, however left in the natural log form. Linear regression was performed to obtain A and B of the equation:

$$Q_m = A + BQ_c \quad (1$$

as well as the standard error of estimate (SEE). The ideal formula will have the avera value of $Q_c/Q_m = 1$ but it should also have A = 0, B = 1, and SEE = 0.

Results of analyses are presented graphically as shown in Fig. 1.

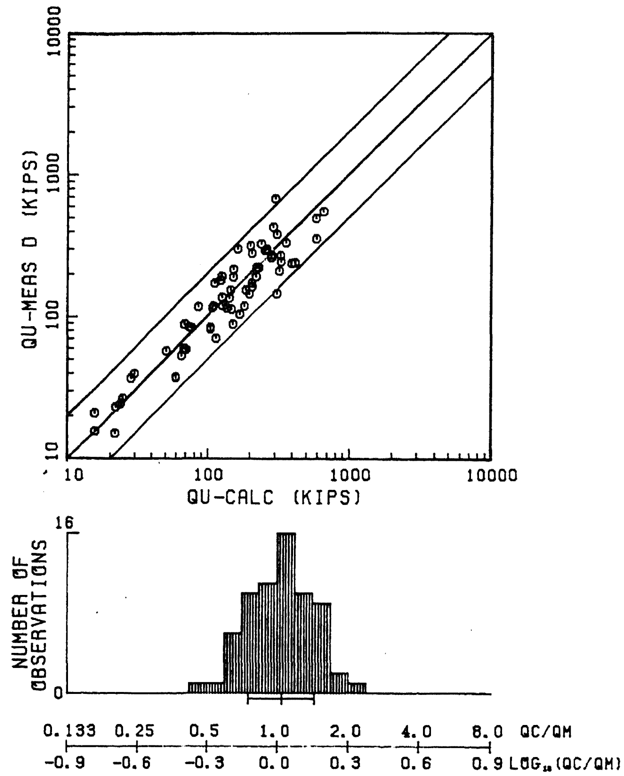

Fig. 1 Comparison of Measured and Computed Capacities for Full Displacement Untapered, Piles in Clay, Loaded in Compression, using Method APIC

QUALIFICATIONS

In comparing computed and measured pile capacities, the measured capacity is taken as "correct." However, measured capacities are influenced by a number of factors. We have eliminated from this set of analyses all piles that we know to be influenced by preboring, jetting, casing, and spudding, and piles with oversized coverplates. Tests were eliminated if they were preceded by lateral load tests. Of the 1000 tests, the numbers eliminated from the analyses reported here include 48 which were prebored, 7 that were jetted, 33 which had oversized coverplates, and 2 which were performed after a lateral load test. Of the remaining tests, 96 were on piles that had been loaded axially to failure before the test used here (20 of these were driven deeper before the final test). These 96 were included in the data set used here. Tests were eliminated if the setup time seemed inadequate, e.g., less than seven days for terrestrial sized piles in clay, but it is certain that higher capacities would have been measured if the set-up times had been increased. Some measured capacities are too high because of friction in the loading jack.

The calculated capacities are influenced by techniques used to measure soil properties. Strengths measured using laboratory tests were effected by the sampling, storage, trimming, and testing techniques used. In the case of sands, scatter in N values is well known, and assigning values of N_q and δ based on visual description is likely to lead to scatter.

Scatter in results also occurs due to inadequate definition of soil properties. In a few cases the nearest soil boring was several hundred feet away. In others the borings were close but the measured strengths scattered widely.

Load tests were assigned data quality factors (DQF) ranging from 1 for data of the lowest acceptable quality to 5 for data of the highest quality. Tests with DQF's of 1 were generally characterized by such factors as erratic soil conditions, no close soil borings, and inadequate soil testing. Tests with DQF's of 5 involved sophisticated soil tests, usually an instrumented pile, and often uniform soils. Data for about 5000 load tests were examined and only about 1000 were included in the data set. The major causes of rejection were lack of soil data and applications of peak loads much less than the plunging failure load.

Problems develop in some comparisons when the number of piles involved in an analysis is small because a disproportionate fraction of the tests may have come from a single site where conditions are not typical.

Finally, some of the data may be in error because of blunders, e.g., confusion between metric tons and English tons.

SOURCE

The results of extensive analyses were presented in a project report by Olson and Dennis (1982). Some of the numerical results differ slightly between this paper and the report because of reinterpretation of several of the load tests and further development of the program.

PILES IN CLAY

There were 279 tests on piles in profiles not containing any sand, with data quality factors of 2 or more.

Comparison of Methods

An initial set of analyses was performed using full displacement, untapered, piles in compression using defined failure. Data for the five methods of analysis are shown in Table 2. Method NCL1, shown in Table 1, will be discussed subsequently. Data from the APIC analyses are shown in Fig. 1. On the average, capacities are slightly underpredicted using ALP1, and overpredicted by the other methods, and the methods had similar amounts of scatter. The maximum scatter was about ±2 to ±2½ times.

Effects of Surface Roughness

The ALP1 method involves a separation of piles into two groups, vis. steel piles, and concrete and timber piles. A set of analyses were performed using untapered, full displacement, piles in both tension and compression, using data quality factors of 2-5, and defined

TABLE II. Summary of Analyses of Untapered, Full Displacement Piles, Loaded in Compression, in Clay, with Data Quality Factors of 2 through 5, at Defined Failure*

Var.	$\bar{x}$	SD	N	Min	Max	A kips	B	SEE kips
ALP1	0.93	.33	67	.53	1.74	1	1.12	64
ALP2	1.05	.36	67	.36	2.35	53	0.65	91
LAM1	1.09	.32	67	.44	2.23	31	0.73	77
LAM2	1.06	.32	67	.45	1.81	30	0.75	71
APIC	1.04	.32	67	.44	2.14	29	0.80	77
NCL1	1.00	.30	67	.51	2.12	38	0.76	74

failure for the three pile materials, using method ALP1. The results, summarized in Table 3, show that method ALP1 underpredicts the

TABLE III. Summary of Analyses for Untapered Full Displacement Piles in Clay, Loaded in Tension or Compression, with Data Quality Factors of 2 through 5, using Defined Failure and the ALP1 Method of Analysis

Var.	$\bar{x}$	SD	N	Min	Max	A kips	B	SEE kips
TIMB	.98	.37	7	.67	1.71	-22	1.52	20
CONC	1.05	.39	36	.54	1.74	-18	1.13	72
STLP	.84	.25	19	.53	1.35	33	1.04	67

capacities of steel pipe piles, but predicts capacities of timber (untapered) and precast concrete piles fairly well. No evidence could be found to show any influence of pile material on side shear capacity.

Comparison of Tensile and Compressive Capacities

Tension and compression tests were then considered separately. Data for untapered, full displacement piles in clay, with data quality factors of 2 through 5, and using defined failure, and the APIC method are summarized in Table 4. Unfortunately, there were only three tests in tension and all were from the same site. For that site, the average Q_c/Q_m in compression was 1.34 and in tension was 1.17. If the tip capacities in compression were calculated correctly, the data indicate a higher side shear in tension than in compression, an unlikely occurrence.

Comparison of Open Ended and Closed Ended Pipe Piles

For open ended pipe piles, the tip capacity was taken as the smaller of the tip capacity of a closed ended pipe, and the tip capacity of the steel end of the open tube plus the side shear capacity of a full plug. The results of the analyses, summarized in Table 5, indicate that

TABLE IV. Summary of Analyses for Untapered, Full Displacement Piles in Clay, using Data Quality Factors of 2-5, Defined Failure, and the APIC Metho of Analysis

Var.	$\bar{x}$	SD	N	Min	Max	A kips	B	SEE kips
TENS	1.17	.17	3	1.00	1.40	120	.12	9
COMP	1.04	.32	67	.44	2.14	29	.80	77

TABLE V. Summary of Analyses for Open and Closed Ended Steel Pipe Piles in Clay, using Data Quality Factors of 2-5, Defined Failure, and the APIC Method of Analysis

Var.	$\bar{x}$	SD	N	Min	Max	A kips	B	SEE kips
Open	1.17	.34	21	.65	2.16	-25	1.00	157
Closed	1.15	.28	19	.68	2.14	-28	.79	74

the method used for open ended pipe piles gives results comparable to those for closed ended pipe piles.

Effects of Pile Taper

A separate analysis was performed for tapered piles, taken collectively, using method APIC. The data are compared in Table 6. The higher capacity of tapered piles in general is suggested by the lower mean value of Q_c/Q_m. Detailed consideration of effects of taper will be presented in a later paper.

TABLE VI. Comparison of Statistical Data for Tapered and Untapered Piles in Clay using Method APIC, using Defined Failure and DQF = 2-5

Var.	$\bar{x}$	SD	N	Min	Max	A kips	B	SEE kips
tapered	0.91	.41	37	.31	1.91	22	.99	61
untapered	1.04	.32	67	.44	2.14	29	.80	77

Method NCL1

More detailed examination of the Q_c/Q_m ratios revealed that the values tended to be site specific, but further that they tended to correlate with the sampling and testing procedure used. The effect of sampling procedure was first investigated by backcalculating the developed values of α as:

$$\alpha = (Q_m - 9\ c_u A_p)/A_s \bar{c}_u \qquad (12)$$

(see Eq. 1-3) using c_u from unconfined compression tests. The developed values of α plotted against the undrained shearing strength in Fig. 2, where solid symbols denote use of samplers

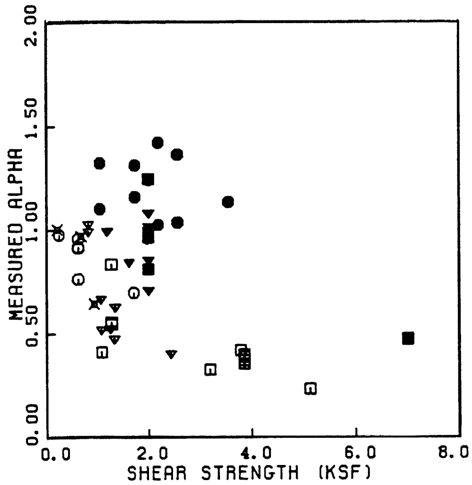

Fig. 2 Comparison of Alpha, Back Calculated from Measured Pile Capacity, with Undrained Shear Strength, Determined using Unconfined Compression Tests, for Steel Pipe Piles. Solid symbols denote cases where samplers were driven and were typically smaller than 3-inch whereas the open symbols apply for samplers at least equal to 3-inch pushed Shelby tubes.

which were generally smaller than three inches in diameter, had high area ratios, and were driven, and hollow symbols denote cases where the samplers were generally three inches in diameter or larger, were thin walled, and were pushed. The lack of overlap between the two data sets indicates the dominant influence of sampling technique.

A significant source of scatter in the correlations used previously clearly involves the mixing of sampling techniques. It is also clear that the testing technique influences the undrained strengths and that the empirical methods should utilize a standard technique for sampling and testing. Unfortunately, the development of a standard is made difficult by lack of data in the case histories on critical soil properties, e.g., sensitivity. The standard to be preferred would probably be unconsolidated-undrained triaxial compression tests using samples trimmed from three-inch, or larger, thin-walled, pushed samplers. However, most of the case histories involve unconfined compression tests so we have adopted that as an expedient standard. To obtain the standard strength, the measured strength is multiplied by a correction factor, F_c. Based on very limited and incomplete data we have used F_c of 0.6 for field vane tests, 0.9 for unconsolidated-undrained tests on samples of good quality, and 1.65 for unconfined compression tests on samples taken using thick-walled driven samplers(values in the paper by Dennis and Olson, 1983a, are incorrect). These factors are known to vary with depth, degree of fissuring, sensitivity, and a variety of other factors and the values cited should not be used indiscriminatly.

Values of developed α were then plotted against the corrected undrained strength (Fig. 3) and

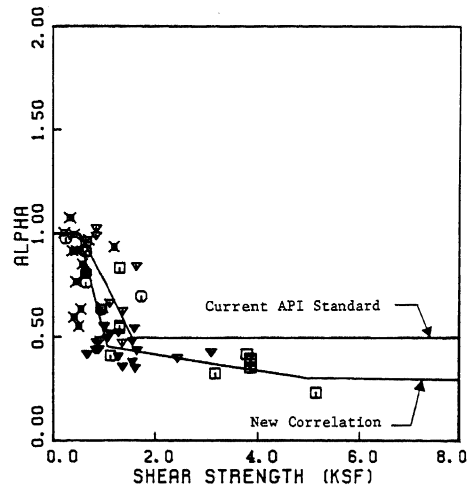

Fig. 3 Relationship Between Alpha and Shearing Strenth, for Straight Sided Piles (No H-Piles). Hollow symbols are for untapered steel pipe piles and the solid symbols are for untapered precast concrete piles.

a simple relationship was fit through the points:

$c_u F_c$ (ksf)	0	600	1200	5000	∞
α	1.0	1.0	0.5	0.3	0.3

The α methods have a general tendency to underpredict capacities of long piles (Fig. 4). The

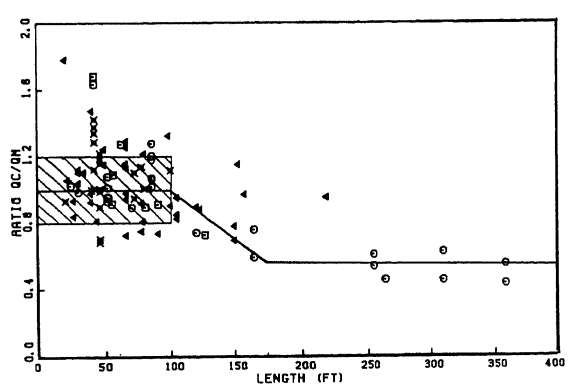

Fig. 4 Comparison of the Ratio of Calculated to Measured Pile Capacity (Q_c/Q_m), using Method NCL1, and Pile Length

predicted capacities were therefore corrected by multiplying by a length correction factor,

F_L. Values of F_L were found to be about as follows (Dennis and Olson, 1983a):

Pile Penetration (feet):	0	100	175	∞
F_L:	1.0	1.0	1.8	1.8

The data in support of these numbers are fragmentary due to lack of field tests on long piles.

The revised method, which we have termed NCL1 so it could be referenced in the computer program, has the side capacity calculated as:

$$Q_s = \bar{c}_u F_c F_L A_s \tag{13}$$

and tip capacity as:

$$Q_p = 9\ c_u F_c A_p \tag{14}$$

The degree of improvement in predicing pile capacities using method NCL1, varies considerably with the data set used in the comparison, being small in cases where F_c and F_L are one. For the data set used here (Table 2) the degree of scatter was reduced by only 7-20% whereas in the data set used by Dennis and Olson (1983a) and Olson and Dennis (1983) which were larger because they used cases for which no load-settlement curve was available, and some piles in interstratified sands and clays, the scatter was reduced by up to 63%. Graphical data comparing measured and predicted capacities for the data set used here are presented in Fig. 5.

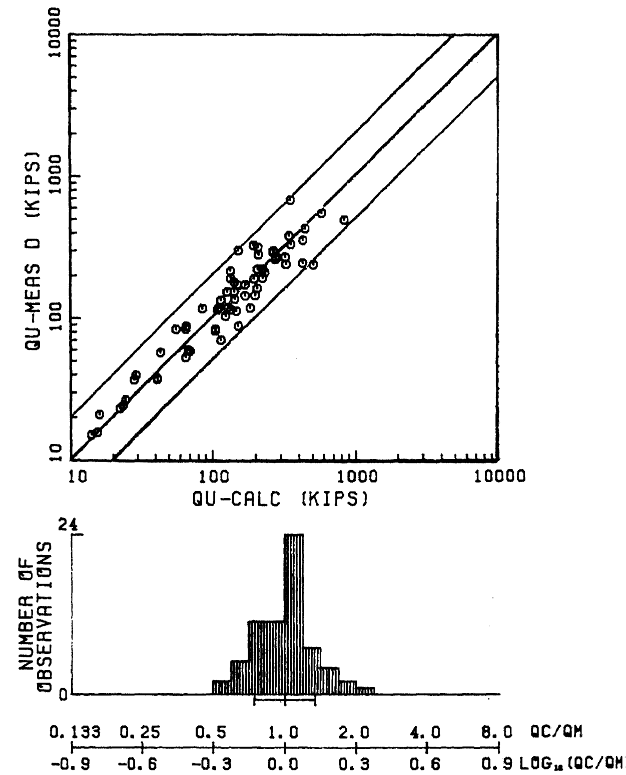

Fig. 5 Comparison of Measured Capacities, at Defined Failure, and Capacities Calculated using Method NCL1, for Untapered Full Displacement, Piles in Clay

PILES IN SAND

Some piles penetrated through layers of soft clay and derived nearly all of their calculated capacity in underlying sand. To include these in the data set, all piles were analyzed but tests were rejected if more than 20% of the calculated side capacity came from the clay. The capacity in the clay was calculated using method NCL1. Data quality factors were 2-5.

APIS Method

First, an analysis was performed for all untapered, full displacement piles. Statistical data are presented as line 1 in Table 7 and the results are plotted in Fig. 6. The method predicts about the right capacity on the average but it tends to overpredict capacities of large piles. It was clear that the largest overpredictions were for large piles in calcareous sands. A second analysis (line 2 in Table 7) was performed with piles in calcareous sands removed. The quality of the prediction improved slightly. The overprediction of some capacities can be reduced by setting limits on the average side shear and end bearing. A third set of analyses were performed using a limit of 2 ksf in side shear and 200 ksf in end bearing and calcareous sands removed. Predictions were again improved slightly (line 3, Table 7).

It seems clear that a high level of accuracy is not to be obtained with the APIS method. The method was developed for use with large offshore piles under circumstances that have generally precluded efforts to measure in situ properties of the sand. The method is here applied to terrestrial sized piles where density effects are likely to be more important. However, for piles iwth calculated capacities (no calcareous, limits on stresses) above 500 kips, Q_c/Q_m averaged 2.21 (22 tests) and ranged from 0.69 to 7.23. The method was selected for use here as an example of methods based on Eqs. 4 and 5.

Method NSA1

A decision was made to try to revise the values of δ, N_q, and K to fit the data set better. The revised method (Olson and Dennis, 1983; Dennis and Olson, 1983b) was called NSA1 for use in the computer program.

In method NSA1, K is taken as 1.0 for both tension and compression, for full displacement piles, and 0.8 for H piles. The side shear capacity from Eq. 5 is multiplied by a reduction factor, F_{SD}, given by:

$$F_{SD} = 1/[0.6 \exp (D/60B)] \tag{15}$$

where D and B are the depth of embedment in sand and pile diameter, respectively. The tip capacity (Eq. 4) is multiplied by a factor, F_D, given by:

$$F_D = 1/(0.15 + .008D) \tag{16}$$

TABLE VII. Summary of Analyses of Piles in Sand

Line No.	$\bar{x}$	σ	N	Min	Max	A kips	B	SEE kips
1	1.19	.92	104	.08	14.16	274	.16	308
2	1.12	.86	101	.08	9.03	249	.24	296
3	1.08	.83	101	.08	7.23	218	.34	287
4	1.09	.57	109	.19	6.43	167	.43	243
5	1.08	.57	106	.19	6.43	170	.43	246
6	1.02	.54	105	.19	5.20	156	.50	239
7	.96	.44	30	.37	2.12	287	.52	217
8	.96	.56	37	.32	5.20	269	.29	207
9	1.08	.30	18	.57	2.01	-1	1.00	55
10	1.14	.48	51	.32	3.75	83	.61	261
11	.97	.52	47	.32	3.28	218	.34	197
12	.48	.51	42	.17	1.88	194	.45	100
13	.96	.70	72	.15	3.32	115	.54	154
14	.93	.65	14	.19	3.00	13	.80	56
15	.55	.68	52	.14	2.56	187	.48	116
16	.96	.51	21	.25	2.07	21	.89	87

where D is again the depth of embedment. Values of N_q and δ are taken from Table 8.

The descriptors are qualitative in keeping with the belief that a design method, such as this one, is mainly useful when only qualitative data are available on soil properties.

Method NSA1 was used with (1) the whole data set of untapered piles in sand, (2) with calcareous sands removed, and (3) finally with case (2) with average side shear and tip stresses limited to 2 ksf and 200 ksf, respectively, and the results are summarized in Table 7, as lines 4-6 respectively. Although the scatter is less than for method APIS, it is still large (compare with Table 2). For piles with calculated capacities above 500 kips (no calcareous sands, limits on stress), Q_c/Q_m averaged 1.74 (N=25) with a range from 0.44 to 5.20.

Open and Closed Ended Pipe Piles

A set of analyses were performed for open ended (line 7, Table 7) and closed ended (line 8, Table 7) steel pipe piles in sand, with calcareous sands eliminated and side and tip stresses limited to 2 ksf and 200 ksf, respectively, using method NSA1. The data are comparable, in large part because the calculations indicated the piles were plugged.

Tension and Compression

For 18 full displacement, untapered, piles in sand (no calcareous sands, limits of 2 and 200 ksf on side shear and end bearing), using method NSA1, the average Q_c/Q_m was 1.08 (line 9, Table 7) and the predictions were better than for compression tests (line 8, Table 7) in

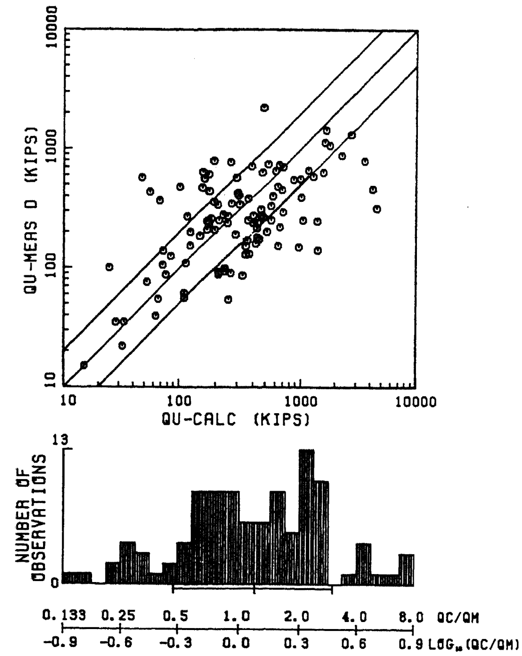

Fig. 6 Comparison of Capacities Measured at Defined Failure and Calculated using Method APIS for Untapered, Full Displacement, Piles in Sand

TABLE VIII. Values of N_q and δ for Method NSA1

Soil Description	δ, deg.	N_q
Very Loose Siliceous Sand Medium Silt Loose-Medium Calcareous Sand* Medium Sandy Silt	15	8
Dense Silt Silty Sand Medium-Dense Calcareous Sand* Loose Siliceous Sand	20	12
Dense Sand Silt Medium Siliceous Sand Medium Silty Sand	25	20
Dense Siliceous Sand Very Dense Silty Sand	30	40
Very Dense Siliceous Sand Dense Gravel	35	50

that the regression line is almost perfect and the standard error of estimate is small.

Effects of Length

As suggested by the values of the regression coefficients, there was a tendency to overpredict capacities of larger piles. The average value of Q_c/Q_m, calculated using method NSA1, was 1.02 for piles penetrating less than 50 feet (N = 43), 1.15 for penetrations of 50-100 feet (N = 52), and 1.83 for penetrations over 100 feet (N = 52), and 1.83 for penetrations over 100 feet (N = 9), all for full displacement, untapered, piles in non-calcareous sands, with side shear and end stresses limited to 2 ksf and 200 ksf respectively.

Effects of Pile Type

Separate analyses were performed for precast concrete (line 10, Table 7) and steel pipe (line 10, Table 7) piles with no calcareous sands and with limits on average side and tip stresses of 2 ksf and 200 ksf, respectively. The NSA1 method tended to overpredict capacities of precast concrete piles which is surprising considering that the concrete piles tended to penetrate comparatively short distances.

Effects of Taper

Method NSA1 was used with tapered, full displacement, piles in sand, with local side shear limited to 2 ksf and tip capacity to 200 ksf. The results (line 12, Table 7) indicate that the tapered piles have significantly higher capacities than untapered piles. The detailed effects of taper will be considered in a separate paper.

Method STDP

The popularity of the standard penetration test in the past is indicated by the fact that of the 1004 tests in the existing data set, the only measure of soil properties was the standard penetration resistance in 438 tests (44% of the data). Of the 336 tests in pure sand profiles, 298 (89%) had standard penetration values.

Unfortunately, the standard penetration test is not standardized world wide. Accordingly, only data from countries following U.S. standards were used.

The results of analyses for untapered, full displacement, piles in sand, in compression and tension, are summarized as lines 13 and 14 in Table 7, respectively, and the compression test data are shown in Fig. 7. As expected, the accuracy of the predictions is low, as indicated by the large values of standard deviation, and by the regression coefficients, but the mean values are close to one. There was a general tendency for the Q_c/Q_m ratio to be large for coarse sands and gravels, and to decrease markedly as grain size decreased.

Data for tapered piles in compression (line 15, Table 7) indicate a marked increase in capacity for tapered piles.

Method CON1

Methods of analysis utilizing the quasi-static cone penetration test are in wide use in parts of the world having extensive deposits of loose

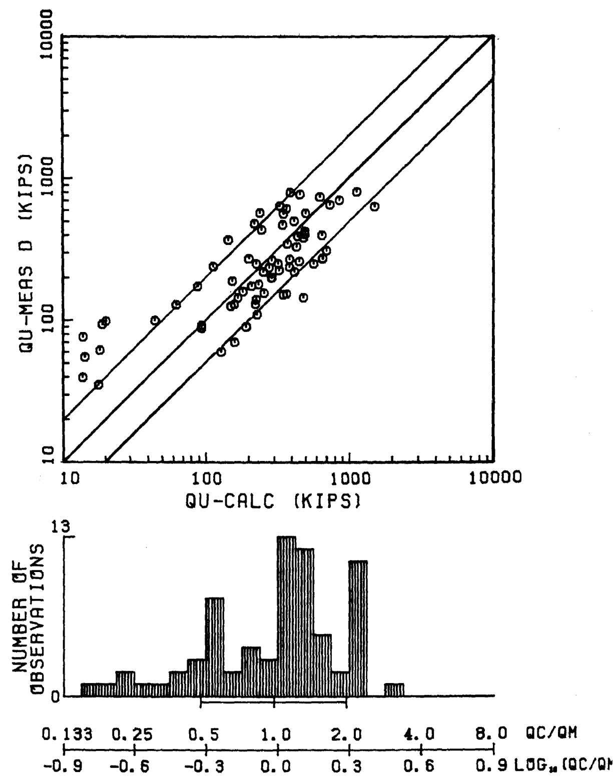

Fig. 7 Comparison of Measured Capacities, at Defined Failure, and Capacities Calculated using Method STDP, for Full Displacement, Untapered, Piles in Compression, in Sand

sand but have not been widely used in the U.S. Only twenty one load tests could be found with cone data. The mean value of Q_c/Q_m (line 16, Table 7) is close to one, and the scatter is smaller and the regression coefficients are better than for most other methods, for piles in sand. It is unfortunate that more load tests could not be found with cone data.

APPLICATIONS IN DESIGN

Design should generally be in terms of probability of failure rather than in terms of factor of safety. In the absence of load tests, the designer should first correct the calculated capacity, Q_c, for bias, for example, by dividing by $\overline{x}$ from Tables 2-7 to obtain a corrected calculated capacity, Q_c':

$$Q_c' = Q_c/\overline{x} \tag{17}$$

Then the design load on a single pile, Q_d, should be taken as:

$$Q_d = \exp\ (\ell n_e Q_c' - \beta\ \sigma_\ell) \tag{18}$$

where σ_ℓ is the natural logarithmic standard deviation, and β is the number of standard deviations from the mean corresponding to a given probability of failure. For probabiliti of failure of 1% and 0.1%, β = 1.28 and 2.33, respectively. Alternatively, a factor of safety corresponding to any given probability of failure is:

$$F_s = \exp\,(\beta\, \sigma_\ell) \tag{19}$$

For example, if the APIC method is used for analysis, and a closed ended steel pipe pile in clay has a calculated capacity of 300 kips, then from Table 6:

$$Q_c' = 300/1.04 = 288 \text{ kips}$$

and for a 0.1% probability of failure

$$F_s = \exp[(2.33)(.32)] = 2.11$$

so

$$Q_d = 288/2.11 = 136 \text{ kips}$$

On the other hand, if the same pile is used in sand with method APIS (Table 7, line 3) then

$$Q_c' = 300/1.08 = 278 \text{ kips}$$

$$F_s = \exp[(2.33)(.83)] = 6.92$$

$$Q_d = 278/6.92 = 40 \text{ kips}$$

The application of this approach to individual piles without load tests was discussed by Kay (1976, 1977) and Olson and Dennis (1983), and to more general cases by Baecher and Rackwitz (1982).

SUMMARY AND CONCLUSIONS

The purpose of this paper was to examine the broad aspects of the relationships between predicted and measured axial load capacities of driven piles. No consideration is given to H piles nor to piles installed by jetting, preboring, or spudding. Detailed analyses using methods discussed here are left for more specialized papers.

In comparison of measured and computed capacities, the measured capacities may be significantly in error. In this paper, failure was specified at a "defined" failure point. The plunging load was about 7% greater for piles in clays. Piles in sand do not experience plunging failure.

For the methods considered for piles in clay, the ALP1 method underpredicts capacities of steel pipe piles. Most methods overpredict capacities slightly. A major cause of scatter involved the methods used to obtain and test soil samples. These methods should be standardized for use in empirical design procedures, or factors need to be used to relate strengths measured using different techniques. The data indicate no effect of pile surface roughness on capacities. The methods seem to predict tensile capacities as well as compressive capacities. Capacities of open ended pipe piles were predicted with the same precision as for closed ended pipe piles. Tapered piles had slightly higher capacities than untapered piles. The ALP1 and ALP2 methods tend to underpredict capacities of long piles.

For piles in sand, the most distinguishing feature of all of the methods was the large amount of scatter between measured (Q_m) and predicted (Q_c) capacities, even when the average value of Q_c/Q_m was near one. Methods like the APIS method have inherent scatter for short piles because of the lack of adjustment for sand density. Calcareous sands yield greatly reduced pile capacities. The design methods work equally well for open ended and closed ended steel pipe piles, probably because the open ended pipes became plugged. Piles seemed to develop about the same side shear in tension as in compression. A limit of 2 ksf on side shear and 200 ksf on tip capacity helped improve correlations of measured and computed capacities but only slightly. Tapered piles (point down) had substantially higher capacities then did untapered piles. There was a tendency to overpredict capacities of long piles.

The available data can be used in reliability analyses to select factors of safety in keeping with the demonstrated accuracy of the predictive method.

For piles in the terrestrial environment, the empirical methods, for piles in clay, are apparently accurate enough that for many projects they can be used for design. For piles in sand, the scatter is larger and efficient designs for projects of moderate to large size require use of dynamic methods or load tests.

ACKNOWLEDGMENTS

The data base and earlier versions of the computer program was developed with financial support from the American Petroleum Institute. The project advisory committee consisted of J.D. Allen (Arco), D.D. Engle (Union), L.M. Kraft, Jr. (McClelland), J.D. Murff (Exxon), B.S. Murphy (Amoco), J.R. Ruser (Shell, Chairman), G.E. Strickland (Chevron), and R.L. Thomas (Phillips). The analyses used here were performed with a revised program and corrected data base not financially supported by API. The opinions expressed in this paper are strictly those of the authors.

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