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AXIAL LOAD CAPACITY OF PILES IN SAND

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ABSTRACT

The axial load capacity of individual piles in cohesionless soils can be estimated at design time using a variety of methods. Because of the difficulties in modeling the process of pile driving, set-up, and loading of piles, useful methods are based on case histories of load tests. Perhaps the most common approach in current use is to specify a soil/pile friction angle, an earth pressure coefficient, a tip bearing capacity factor, and appropriate limits on side shear and end bearing. The various parameters may be made functions of soil classification, relative density, depth, or whatever other variables the investigator thinks are important.

In this paper, we compare several methods of analysis that have been in wide use, as well as a method based on continuous functions and a newer method developed by Jardine and coworkers, with measured capacities for untapered piles in tension and compression, in cohesionless soils, and try to draw conclusions about the relative merits of the methods.

KEYWORDS

piles, capacity, analysis, database, load testing

INTRODUCTION

Driven piles are widely used for the support of structures in both terrestrial and offshore environments. Axial load capacities of individual piles can be estimated using dynamic or static methods, with the static methods used at "design time" and dynamic methods used with test piles for design or with production piles for control.

Static methods may make use of a variety of approaches depending on the soil properties that are available. It is possible to make direct correlations of side shear and end bearing for piles with *in situ* soil properties, e.g., standard penetration or cone tip resistance (Meyerhof, 1976). Alternatively, a simple intuitive approach may be used in which the pile capacity is calculated as:

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

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 submerged in soil (positive for tensile loading and negative for compressive loading).

Side shear is calculated using:

$$Q_s = \sum_{L=1}^N f_s C \Delta L \quad (2)$$

where:

$$f_s = K \sigma' \tan(\delta) \leq f_{lim} \quad (3)$$

where f_s is the local side shear between the pile and the surrounding cohesionless soil (limited to a value f_{lim}), C is pile circumference, ΔL is the pile length in the L^{th} layer, K is the earth pressure coefficient, σ' is the free-field vertical effective stress in the middle of the L^{th} layer, and δ is the friction angle between the pile and the surrounding soil.

Similarly, the tip capacity is calculated using:

$$Q_p = q_p A_p \quad (4)$$

where:

$$q_p = \sigma' N_q \leq q_{lim} \quad (5)$$

where q_p is the net pressure between the pile tip and the subsoil (limited to q_{lim}), A_p is the area of the pile tip, σ' is the free field vertical effective stress at the pile tip, and N_q is a dimensionless bearing capacity factor.

The above equations are widely used for design. One example is their use by the American Petroleum Institute in their Recommended Practice 2A (API RP-2A). We will use RP-2A as an example of a design standard based on this approach.

API assigns the earth pressure coefficient (K) a value of 1.0 full displacement piles and 0.8 for non-displacement (open-ended pipe) piles, with tension and compression treated as the same. The assigned values are shown in Table 1.

Table 1 - API RP-2A Design Values for Cohesionless Soils

Relative Density	Soil	δ deg.	f_{lim} kPa	N_q	q_{lim} kPa
very loose	sand	15	1.0	8	40
loose	sand/silt				
medium	silt				
loose	sand	20	1.4	12	60
medium	sand/silt				
dense	silt				
medium	sand	25	1.7	20	100
dense	silt				
dense	sand	30	2.0	40	200
very dense	sand/silt				
dense	gravel	35	2.4	50	250
very dense	sand				

The wide use of API RP 2A has brought about numerous criticisms (Iskander and Olson, 1992; Pelletier et. al, 1993). For example, stresses around piles are greatly influenced by the presence of the pile so σ' may differ from the free-field vertical effective stress. Soil is dragged down from one layer into the interface between the pile and lower layers (Tomlinson, 1971). Particle sizes and densities of sands are reduced by the high normal stresses, large shearing deformations, and cyclic loading around a pile (Robinsky and Morrison, 1964). Time-dependent changes in pile capacities in sand apparently occur over substantial periods of time even after excess pore pressures have been dissipated (Tavanas, 1971; Armishaw and Cox, 1979). High radial gradients in sand density probably result from lateral movements of the pile during driving (pile shaking, Poisson ratio effects, etc.) (Szechy, 1961; Robinsky and Morrison, 1964). Exterior stresses are influenced by the behavior of the plug inside the pile but that behavior involves a dynamic soil-structure interaction that has not been quantified (Ugaz, 1988; Raines et al. 1992).

Attempts can be made to analyze the behavior of piles using finite element methods but such methods are currently both too expensive for onshore use (as opposed to use offshore where much larger piles are likely to be used) and too simplified to model the problem realistically. Further, complicated analyses have not been supported by a wide range of full scale field load tests and thus the accuracy of the final result cannot be ascertained.

Recently, Jardine and coworkers (Lehane and Jardine, 1994; Chow and Jardine, 1997) have used tests on well-instrumented piles to develop empirical formulas that differ from the API type of formulas. Although the parameters needed for use of Jardine's methods do not exist for most of our pile load tests, crude correlations can be used to provide insight into how well Jardine's methods work in comparison with previous

methods.

While supporting continued research in both dynamic and static methods, the authors believe that design will continue to be based mainly on a database of load test results and the experience of knowledgeable local engineers. This paper is concerned with an existing database for pile load tests in sand and with its use for prediction of pile capacities.

DATABASE

A database of load tests on piles of a variety of types, and in a variety of soils, was developed during the early 1980's and served as a basis for several methods of analysis of piles in sand (Olson and Dennis, 1982; Dennis and Olson, 1983). The American Petroleum Institute (API), which had partially funded development of the database, adopted design parameters different from the ones recommended by Dennis and Olson (1983) based on the database. Later efforts were made to improve on both the size of the database and on its interpretation (Olson and Al-Shafei, 1988; Olson, 1990).

The current study uses an updated data base of 76 load tests for untapered piles in cohesionless soils. Forty-eight tests were on steel pipe piles (13 were open-ended, 35 closed ended) and 28 were on precast concrete piles. The piles used in this study ranged in length from 3 - 42 m and in diameter (width) from 200 mm to 1.4 m. Set-up times (known for 30 of the tests) ranged from 14 hours to 30 days.

In applying the API recommended practice to predict the capacity of the piles in the database, it is necessary to adopt a standard method to define relative density. In the absence of API guidelines for relative density, we followed the classification recommended by Peck et. al (1974), where relative density was defined using standard penetration resistance, N , in blows per 30 cm (blows per foot). The N values were corrected for the effects of overburden pressure using the recommendation of Peck, et al. (1974). The method used to measure the "standard penetration resistance" was undefined in most cases so some degree of scatter may result from use of non-standard procedures.

Generally, for piles in sand, the load increases continuously as the pile tip settles so there is no sudden "plunging" failure and no unique peak load capacity can be defined. Several definitions of "failure" can be adopted under these circumstances. For the analyses reported herein, failure was defined as the peak applied load during the test, but tests were not accepted unless loads reached at least the value defined by Davisson (1973). Use of the peak load leads to scatter because some tests were stopped when some settlement-criterion was satisfied, whereas others were carried to relatively large settlements, and thus to relatively large loads. For the piles used in this study, the ratio of peak load to Davisson's load ranged from 1.0 (test was stopped when Davisson's criterion was satisfied) to 2.0 and averaged 1.2. The settlements at the peak load ranged from 6 mm to 800 mm, and averaged 25 mm.

Use of the existing database places limitations on the accuracy of any method developed from the database. The data are concentrated in medium to dense clean sands. When the 1990 API method of analysis was used, the calculation indicated that 63% of the side shear was in clean sand, and 28% in silty sands and sandy silts, with 4% in silt, 3% in sand/gravel mixtures, and 1% in clay. For the tip capacity, 89% was estimated to come from clean sand, 8% from sand/gravel mixtures, 1% from gravel, 2% from sand/silt mixtures, and none from silt or clay. Silty soils are not well represented because, geologically, the finer soils tend to occur at shallow depth, with sands at greater depth. The low effective stresses at shallow depth then lead to reduced frictional stresses. Piles are typically driven into firm material and thus logically tend to derive most of their side shear and tip capacities from deeper, thus coarser, soils. Significant vertical and lateral variations in the properties of cohesionless soils may result in soil borings not accurately reflecting the soil properties at the location of the load test. When standard penetration resistance are used for soil classification, variations in the efficiency of the hammers leads to scatter, especially when testing methods differ with respect to time or regionally. In a few cases, some of the soil was excavated from the site after soil borings were made and no borings were made after the excavation.

ANALYSES BASED ON API RECOMMENDATIONS

Analyses were performed for piles in the data base using the 1993 API RP 2A recommendations. Measured and predicted capacities are compared in Fig. 1. The ratio of calculated-to-measured axial load capacity (Q_c/Q_m) ranged from 0.04 to 3.57.

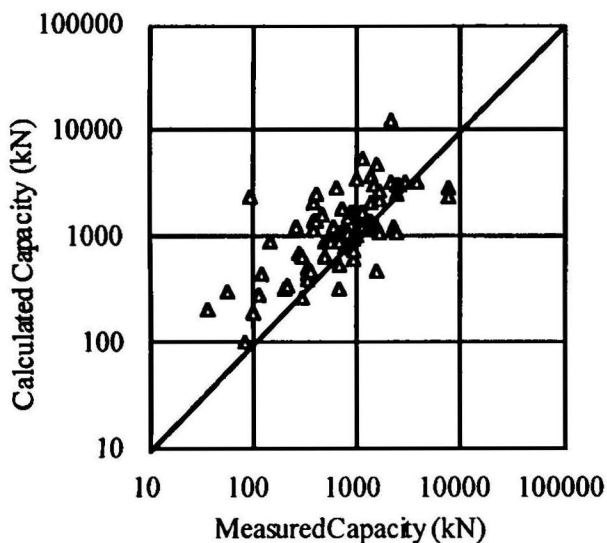


Fig. 1 Comparison of Calculate and Measured Pile Capacities Based on API RP-2A (1993)

The mean $\log(Q_c/Q_m)$ was -0.173 (antilog is 0.67), indicating that this method underpredicts capacities on the average, and the standard deviation of $\log(Q_c/Q_m)$ was 0.35. The large scatter indicates that the method can be unsafe in some cases and uneconomical in others. It may be noted, however, that

"unsafe" for individual piles in sands implies unexpectedly large, but stable, deformations, not collapse.

Use of the API method led to underprediction of capacities of short piles (Fig. 2) and overprediction of long piles. There were only three tests with pile penetrations exceeding 30

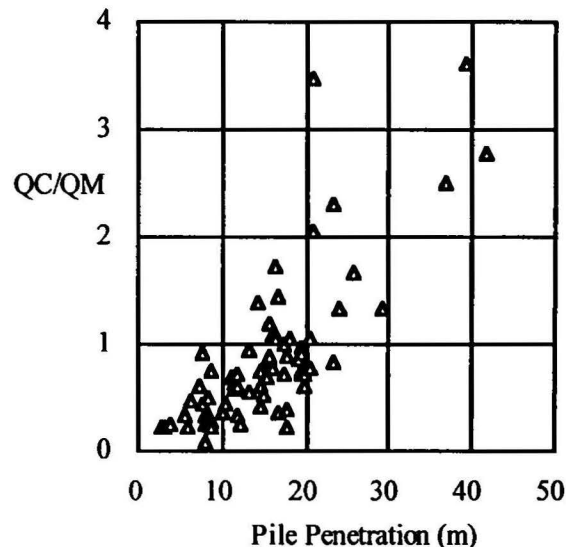


Fig. 2 Effect of Pile Penetration on the Q_c/Q_m Ratio Based on the API (1993) Standard

meters. Two of them were from Japan and the "sand" may have been a volcanic sand that behaves differently from the silica sands around other piles. Further, one might surmise that sands around piles with such large penetrations must be unusual in some respect because, otherwise, it would not be possible to achieve such large penetrations. Nevertheless, capacities were generally underpredicted for piles penetrating less than 20 meters and generally overpredicted for longer piles indicating defects in the method. It should be noted that the API recommended practice is intended for use with large offshore piles where small penetrations are unlikely.

REVISION OF THE API METHOD

A logical approach to improving the API method would be to use the same formulation but refine the parameters (Olson, 1990). The design parameters were expressed as functions of the standard penetration resistance (corrected for the effect of overburden) instead of using more qualitative descriptive terms and the lateral coefficient of earth pressure was made a function of the standard penetration resistance as follows:

$$K = 0.16 + 0.015 N \quad \text{for partial displacement piles}$$

$$K = 0.70 + 0.015 N \quad \text{for full displacement piles}$$

Extensive trial analyses were performed to obtain parameters that would fit the data base better. As an example, the parameters selected for clean sands are shown in Table 2.

Table 2 - Parameters Recommended by Olson (1990) for Analysis of Capacities of Piles in Sand

Soil Type	Range in N Values, blows/300m m	δ (deg)	f_{lim} (kN)	N_q	q_{lim} (kN)
Sand	0-4	20	4.4	50	180
	5-10	30	4.9	120	530
	11-30	35	8.4	120	840
	31-50	40	11.6	120	840
	51-100	40	16.4	130	890
	over 100	40	16.7	220	2360

Analyses were performed for piles in the current database using the Olson (1990) method (Fig. 3). The logarithmic mean Q_c/Q_m was 0.023 (antilog was 1.05) and the logarithmic standard deviation was 0.20, a reduction of 43% compared with the API value. The range in Q_c/Q_m was from 0.38 to 4.24. This method also overpredicted the capacities of long piles (Fig. 4).

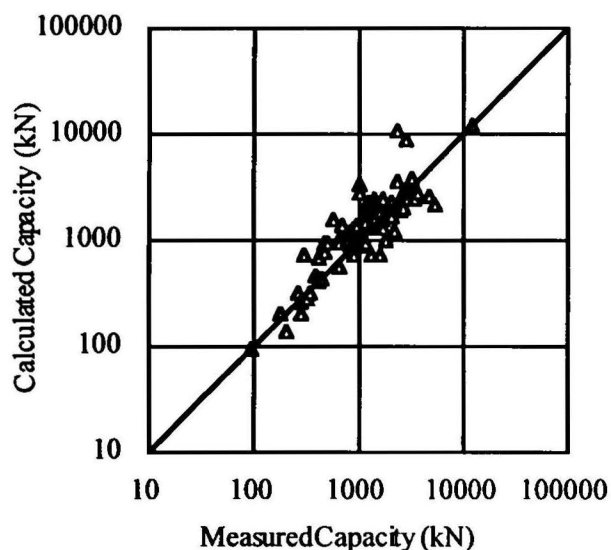


Fig. 3 Comparison of Calculated and Measured Pile Capacities Based on Olson (1990)

The selected parameters tended to mimic previous methods, e.g., by assigning greater strengths to gravels than sands. Recent experiments (Lehane and Jardine, 1994) indicate that measured soil/steel friction angles of coarse-grained soils may actually be less than for finer-grained, but still cohesionless, soils and thus suggests that some of the pre-existing ideas about soil/pile behavior should be reconsidered.

A further problem develops when a table of coefficients is used. It was possible to assign properties that worked well for cases where there might be only a few pile load tests, e.g., there is only one test in silty sand with N values exceeding 100 blows/300 mm, so properties can be assigned that work well for that one test but do not make sense when compared with properties for say silty sand with an N of 90 blows/300 mm. The logical way out of such a dilemma might be better to

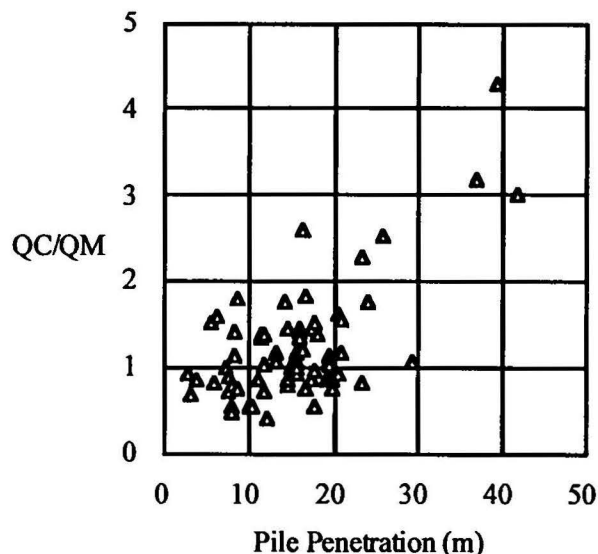


Fig. 4 Effect of Pile Penetration on Q_c/Q_m Based on Olson (1990) Method

formulate relationships between input properties such as standard penetration resistance, and properties used in the analysis (K , δ , f_{lim} , N_q , q_{lim}) in a simpler manner using continuous functions as opposed to using tables of properties.

ANALYSES USING CONTINUOUS FUNCTIONS

The starting point is logically a log function, e.g., for example:

$$y = A + B \log(x) \quad (6)$$

where y is the dependent variable, in this case being δ , f_{lim} , N_q , or q_{lim} , x is the independent variable, here the standard penetration resistance N , and A and B are parameters whose values depend on the parameter being evaluated. This function did not fit the design parameters used in either API (1993) or Olson (1990) but plots versus $\log(N)$ using those methods were illogical and probably resulted from the small sample sizes used in developing the earlier methods.

Values of A and B were sought for each dependent variable (δ , f_{lim} , N_q , and q_{lim}) separately for untapered precast concrete and steel pipe piles, in both tension and compression, and for steel pipe piles that were both open ended and closed ended (six sets of trials). Hundreds of trial analyses were performed but it was found that no rational sets of parameters could be found. In some cases, the sample size was too small, e.g., only four tests for open-ended pipe piles in compression, and four for precast concrete piles in tension. Problems developed because some of the data sets were dominated by tests at a single site. Finally, analyses were performed using the following parameters:

$$\delta = 20 + 8 \log(N) \quad \text{degrees} \quad (7)$$

$$f_{lim} = 67 \log(N) \text{ kPa} \quad (8)$$

$$N_Q = 40 + 60 \log(N) \quad (9)$$

$$q_{lim} = 3.4 + 38 \log(N) \quad \text{mPa} \quad (10)$$

$$K = 0.16 + 0.015N \quad \text{non-displacement piles} \quad (11)$$

$$K = 0.70 + 0.015N \quad \text{full displacement piles} \quad (12)$$

It was necessary to force high values of side shear at low stress levels, to account for high capacities of short piles, but to impose severe limits on stresses to prevent massive overprediction for long piles.

The measured and computed capacities are compared in Fig. 5. The mean $\log(Q_c/Q_m)$ was -0.009 (antilog was 0.98) and the standard deviation of $\log(Q_c/Q_m)$ was 0.25. The influence of pile penetration on the ratio Q_c/Q_m is shown in Fig. 6.

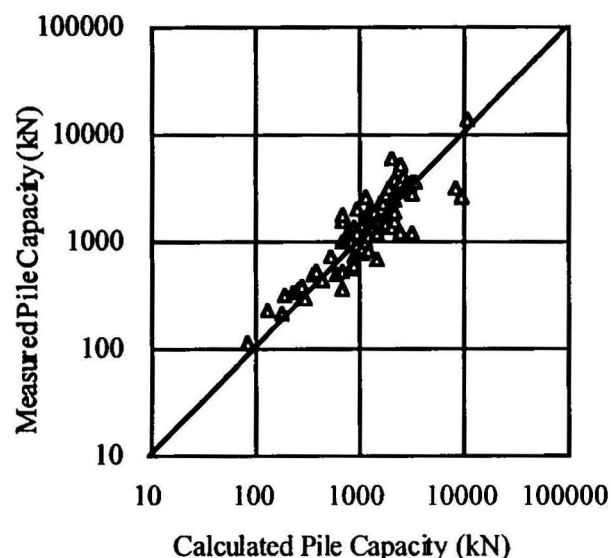


Fig. 5 Comparison of Calculated and Measured Pile Capacities using a Set of Continuous Functions

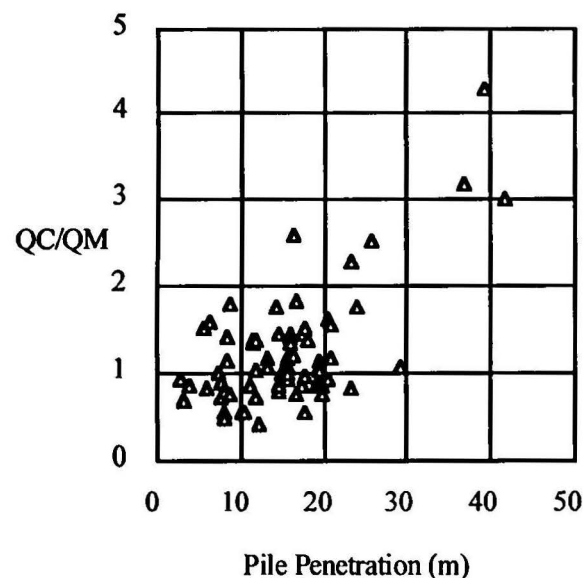


Fig. 6 Effect of Pile Penetration on QC/QM for One Set of Continuous Functions

The use of continuous functions led to a reasonable mean value of Q_c/Q_m and to a scatter only slightly more than in a previous method that was less constrained to use rational parameters. The problem remains of the depth dependency of Q_c/Q_m . The inability to find rational parameters to eliminate the depth effect suggest that there may be something wrong with the general formulation of the equations. Gregerson et al. (1973) drove an instrumented pile to a depth of eight meters and ran a load test. The apparent distribution of side shear is shown in Fig. 7

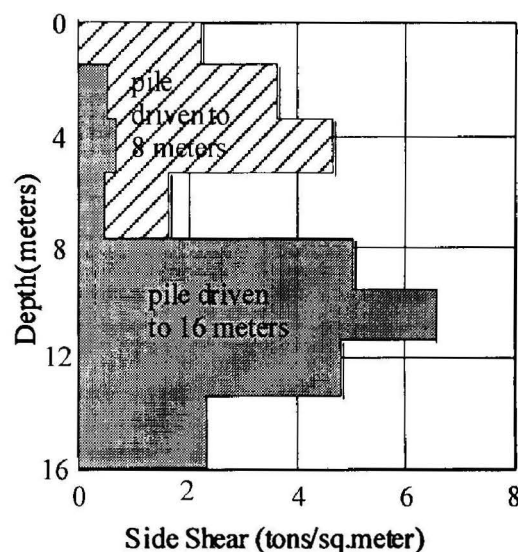


Fig. 7 Variation of Side Shear with Respect to Depth for Instrumented Precast Concrete Piles in Loose Sand (Gregerson et al, 1973)

(labeled "pile driven to 8 meters"). The side shear did not rise more-or-less uniformly from a negligible value at the surface to a peak value at the base, as postulated in the above methods. An additional eight meters of instrumented pile section was added and the resulting pile driven to a depth of sixteen meters. Again, the side shear peaked at some height above the base (Fig. 7). Perhaps of more interest, the apparent side shear in the upper section decreased sharply. Data such as these suggest that the formulations used in the above methods may be fundamentally flawed.

JARDINE'S METHOD

Jardine's methods have involved fitting functions to data he collected using a pushed, closed-ended, model steel pipe pile and then the resulting functions were applied to a select set of well documented case histories of load tests. For piles in cohesionless soils, the radial stress after reconsolidation (σ'_{rc}) is assumed to be given by:

$$\sigma'_{rc} = 0.029 q_c (\sigma'_{vo}/P_s)^{0.13} (h/R^*)^{-0.38} \quad (13)$$

where q_c is cone tip resistance, σ'_{vo} is the free-field vertical

effective stress, P_a is atmospheric pressure, h is the vertical distance from the point where the stress is being calculated to the pile tip, and R^* is the tip radius for a closed-ended pile and, for an open ended pile is:

$$R^* = \sqrt{R_{outer}^2 - R_{inner}^2} \quad (14)$$

Chow and Jardine (1997) correct the lateral stresses for dilatency of the soil during loading but note that the effect is generally small. Their equation for the change in normal stress due to dilatency contained parameters not available for our database (local soil shear modulus and pile roughness). We made estimates of these parameters and found that the correction was negligible so it will not be discussed further here. The lateral stress of soil on pile at failure is then:

$$\sigma'_{rf} = \sigma'_{rc} \quad (\text{compression}) \quad (15)$$

$$\sigma'_{rf} = 0.72 \sigma'_{rc} \quad (\text{tension}) \quad (16)$$

Finally, the side shear at failure, τ_{rf} , is given by:

$$\tau_{rf} = \sigma'_{rf} \tan(\delta) \quad (17)$$

LeHane and Jardine (1994) presented a curve of δ versus D_{50} for cohesionless soils on steel and found that δ was generally higher for the finer soils than for coarser cohesionless soils, contrary to usual expectations.

Chow and Jardine (1997) assume that pile plugging occurs if:

$$D_{inner}/D_{CPT} \leq 0.083 q_c/P_a \quad (18)$$

and for plugged piles, the tip capacity (Q_b) is given by:

$$Q_b = q_c \pi R_{outer}^2 [0.5 - 0.25[\log_{10}(D/D_{CPT})]] \quad (19)$$

where D_{CPT} is the diameter of the cone and D is the pile tip diameter.

For our data base, standard penetration resistances were available but not usually q_c . For the cases in which both N and q_c were known in a soil layer, we compared the two graphically and found, on the average that q_c (ksf) = 10 N or q_c (kPa) = 0.201 N and we used those values in reducing data.

For the same database used previously, the average value of $\log(Q_c/Q_m)$ was -0.034 (antilog was 0.92) with a logarithmic standard deviation of 0.22. The scatter between Q_c and Q_m was comparable to previous analyses (Fig. 8) and there was still a depth dependency of Q_c/Q_m (Fig. 9).

DISCUSSION

The usual formulation of equations used to estimate axial load capacity of piles in cohesionless soils seems to be fundamentally in error in that it presupposes that the side shear

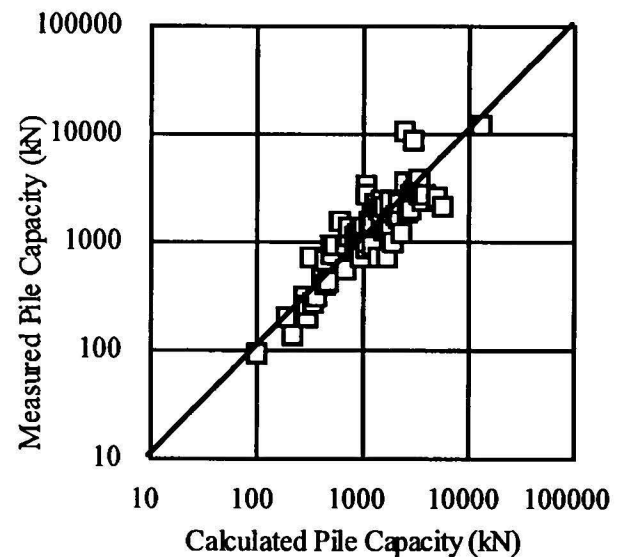


Fig. 8 Comparison of Measured and Calculated Capacities Based on Jardine's Method

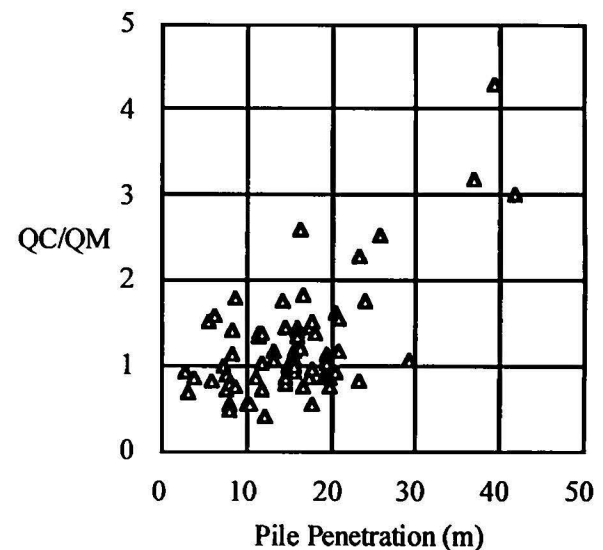


Fig. 9 Depth Dependency of Q_c/Q_m for Jardine's Method

increases with depth in direct proportion to the free-field effective overburden pressure, but then imposes a limit on side shear. The limit seems artificial and existing field data indicate that the side shear tends to maximize at some height above the base. Jardine's method provides a non-linear variation in side shear but it provides a maximum side shear at the pile tip as opposed to at a shallower depth as found by Gregersen et al. (1973). From a purely practical point of view, for American practice, it would be better to formulate the equations using N in place of q_c . As in the past, the major need remains to develop a data base of a large number of well documented pile load test case histories.

CONCLUSIONS

The following conclusions are drawn from these analyses:

- The standard formulation used to estimate side shear for

piles in cohesionless soils seems oversimplified. A formulation in which side shear increases more rapidly with respect to depth, but peaks above the pile tip, seems preferable.

- For all of the methods considered here, the capacity of short piles is underpredicted and of long piles is overpredicted. The problem is least severe with the Olson (1990) method but that is probably due to the fact that the method was developed using an earlier version of the current database and then used to predict capacity of piles in the same database.
- Overprediction of capacities for long piles may indicate problems in the various formulations but it may also simply reflect different soil properties that would allow such long piles to be driven, e.g., presence of volcanic sands.
- The use of continuous functions with the standard approach seems like a useful simplification but use of functions more like those used by Jardine seems preferable, although the functions should be developed using input variables representative of local practice.

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