Iranian Journal of Science & Technology, Transaction B, Engineering, Vol. 32, No. B2, pp 125-140
Printed in The Islamic Republic of Iran, 2008

BEARING CAPACITY OF DRIVEN PILES IN SANDS FROM SPT–APPLIED TO 60 CASE HISTORIES*

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Abstract– In recent years determining bearing capacity of piles from in-situ testing data as a complement of static and dynamic analysis has been used by geotechnical engineers. In this paper, different approaches for estimating the bearing capacity of piles from SPT data have been explained and compared. A new method based on the N-value from SPT is presented and calibrated. Data averaging, failure zone extension, and plunging failure of piles has been noticed in the proposed approach. A data base has been compiled including 43 full scale static pile load tests and 17 dynamic testings which were analyzed with the signal matching technique by CAPWAP. The SPT data were performed close to pile locations are also included in the data base. A comparison of current methods by error investigation with cumulative probability and Log-Normal approaches demonstrates that the proposed method predicts pile capacity with more accuracy and less scatter than other methods. Results of prediction with good agreement to measured capacities indicate that the proposed method can be used as an alternative for determining the bearing capacity of piles in geotechnical practice.

Keywords– Pile, bearing capacity, standard penetration test, SPT, static and dynamic load tests

1. INTRODUCTION

The prediction of the axial capacity of piles has been a challenge since the beginning of the geotechnical engineering profession. Several methods and approaches have been developed to overcome the uncertainty in the prediction. The methods include some simplifying assumptions and/or empirical approaches regarding soil stratigraphy, soil-pile structure interaction, and distribution of soil resistance along the pile. Therefore, they do not provide truly quantitative values directly useful in foundation design [1].

Bearing capacity of piles can be determined by five approaches as follows:

- Interpretation of data from full-scale pile loading tests,
- Dynamic analysis methods based on wave equation analysis,
- Dynamic testing by means of the Pile Driving Analyzer (PDA),
- Static analysis by applying soil parameters in effective stress or total stress approaches,
- Methods using the results of in-situ investigation tests, directly or indirectly.

In view of the uncertainties involved in the analysis and design of pile foundations, it has become customary, and in many cases mandatory, to perform full-scale pile loading tests. Such tests are expensive, time-consuming, and the costs are often difficult to justify for ordinary or small projects.

Dynamic analysis methods apply to driven piles, and are based on wave mechanics for the hammer-

*Received by the editors September 25, 2006; final revised form July 14, 2007.
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pile-soil system. The uncertainty in the hammer impact effect, as well as changes in soil strength from the conditions at the time of pile driving, and also at the time of loading, causes uncertainties in bearing capacity determination. Moreover, a wave equation analysis requires input assumptions that can significantly bias the results [1].

Dynamic testing methods are based on monitoring acceleration and strain near the pile head during driving. From these measurements, the pile capacity can be estimated by means of the Pile Driving Analyzer (PDA) and numerical analysis of the data. However, the PDA can only be used by an experienced person, and the test results apply essentially to the field-testing considerable situation. One considerable limitation is that the capacity estimation is not available until the pile is driven [2].

Static analysis methods estimate shaft and base resistances separately and differently. For shaft resistance, in cohesive as well as non-cohesive soils, considerable uncertainty and debate exist over the appropriate choice of the horizontal stress coefficient, K_s. Normally, bearing capacity theory is applied to estimate base resistance in non-cohesive soils. However, the theory involves a rather approximate Φ-N_q relationship coupled with the difficulty of determining a reliable and representative in-situ value of the Φ angle and the assumption of a proper shear failure surface around the pile tip. This creates doubts about relying on the bearing capacity theory in pile foundation design. Design guidelines based on static analysis often recommend using the critical depth concept. However, the critical depth is an idealization that has neither theoretical nor reliable experimental support, and contradicts physical laws [1].

In recent years, the application of in-situ testing techniques has increased for geotechnical design. This is due to the rapid development of in-situ testing instruments, an improved understanding of the behavior of soils, and the subsequent recognition of some of the limitations and inadequacies of conventional laboratory testing [3, 4].

The Standard Penetration Test, SPT, is still the most commonly used in-situ test. However, some problems and limitations are included with the SPT with respect to interpretation and repeatability. These are due to the uncertainty of the energy delivered by various SPT hammers to the anvil system and also with the test procedure.

Pile capacity determination by SPT is one of the earliest applications of this test that includes two main approaches, direct and indirect methods [5].

Direct methods apply N values with some modification factors. However, considerable uncertainty exists regarding filtering and averaging the data relating to pile resistance, failure zone around the pile base, use of total stress approaches, and capacity of piles with limited base penetration in dense strata.

Indirect SPT methods employ a friction angle and undrained shear strength values estimated from measured data based on different theories. In indirect methods, only soil parameters are obtained from SPT results and the methodology of the pile bearing capacity estimation is the same as for the static methods, and therefore involves the same sources of shortcomings [3].

2. IN-SITU TESTS FOR PILE BEARING CAPACITY DETERMINATION

Although there are some problems on the explicit interpretation of the results of SPT, this test is the most frequent in-situ test in geotechnical practice because of its simplicity and affordable costs. In this paper, five common SPT methods to estimate the bearing capacity of piles have been surveyed and presented in Table 1 [6-10].

Table 1. Current SPT direct methods for prediction of pile bearing capacity

No.	Method	Unit shaft resistance (KPa)	Unit base resistance (MPa)	Explanations
1	Aoki & De'Alencar [6]	$r_s=(ak/3.5)N_s$	$r_t=(k/1.75)\ N_b$ N_b: average of three value of SPT blows around pile base	Failure criteria : Vander veen method Energy ratio for N: 70% For sand: a=14 & k=1 ,For clay: a=60 & k=0.2
2	Shioi & Fukui [7]	$r_s=n_s\ N_s$	For driven piles: $r_t=(1+0.04(D_b/B))N_b \leq 0.3N_b$ For pipe piles: $r_t=0.06(D_b/B)N_b \leq 0.3N_b$	Energy ratio for N: 55% n_s=2 for sand and 10 for clay
3	Meyerhof [8]	$r_s=n_s\ N_s$	$r_t=0.4\ N_1\ C_1\ C_2$ N_1: N value at the base level	Failure criterion : Minimum slope of load-movement Curve Energy ratio for N: 55% Low disp. piles: n_s=1 High disp. piles: n_s=2
4	Briaud & Tucker [23]	$r_S=\dfrac{0.1}{\dfrac{1}{k_s}+\dfrac{0.1}{r_{s.\max}+r_{s.res}}}-r_{s.res}$	$r_t=\dfrac{0.1}{\dfrac{1}{k_t}+\dfrac{0.1}{r_{t.\max}-r_{t.res}}}+r_{t.res}$	Failure criteria: penetration of pile head equal 10% of pile Diameter
5	Bazaraa & Kurkur [10]	$r_s=n_s\ N_s$	$r_t=n_b\ N_b$ N_b: average of N Between 1B above and 3.75b under pile base, $N_b \leq 50$	n_s=2~4; n_b=0.06~0.2
	N_s: average value of N around pile embedment depth.			
	N_b: average value of N around pile base.			
	$C_1=((B+0.5)/2B)^n$; n=1, 2, 3 respectively for loose, medium and dense soil when pile diameter (B)>0.5 m, otherwise C_1=1.			
	C_2=D/10B when penetration in dense layer (D)>10B, otherwise C_2=1.			
	$k_t=1868400(N_b)^{0.0065}$, Nb average of SPT blow-count between 4B above and 4B under the pile base			
	$k_s=20000(N_s)^{0.27}$			
	$r_{t.max}=1975(N_b)^{0.36}$			
	$r_{s.max}=22.4(N_s)^{0.29}$			
	$r_{t.res}=557L\ ((k_s*p)/(A_t*E_p))^{0.5}$, L: length of pile, p: perimeter of pile, At: cross section area of pile, Ep :Elastic modulus of pile			
	$r_{s.res}=r_{t.res}\ (A_t/A_s)$, A_s: Surface area of pile			

When using these methods, the following inadequacies appear:

- All SPT-based methods to predict the pile bearing capacity ignore the excessive pore water pressure generated during the test and therefore the results may not be reliable in low permeable soils such as clays and silts. Since design procedures mainly involve considering the long term capacity of piles, SPT data generally is only applicable for sands or non-cohesive granular soils.

- Among the five SPT methods presented in Table 1, Shioi & Fukui [7] and Bazaraa & Kurkur [10], do not specify any failure criterion for bearing capacity determination. This fact can be confusing in prediction; therefore, a failure criterion should be pointed out.
- In all SPT methods, an arithmetic average of N values around the pile base and along the pile body are related to pile base bearing capacity and pile shaft resistance, while the variation of SPT N values in peaks and troughs can significantly bias the results.
- All methods have very limited failure zones. This factor strongly affects the calculated pile bearing capacity, thus this zone must be carefully chosen to properly estimate the pile base bearing capacity. Aoki & De'Alencar [6], Shioi & Fukui [7] and Meyerhof [8] methods do not specify this zone and as a result, choosing a consistent value for N is done with some uncertainty.

In Briaud & Tucker [23] and Bazaraa & Kurkur [10] methods, the energy ratio of N values was not specified, however this index is directly related to the pile bearing capacity and affects the results.

3. PROPOSED DIRECT METHODS FOR PILE CAPACITY FROM SPT

A new method has been developed for pile bearing capacity estimation, based on the results of standard penetration tests in granular soils [11, 12].

The failure criterion of this method is plunging. It occurs when a pile has rapid movement under a sustained or slightly increased load. An ideal plunging failure diagram is shown in Fig. 1. The first part of this diagram, called the semi-elastic portion, demonstrate the elastic interaction of the soil-pile system that continues to point A, where the pile head movement could be very small. The second part of this diagram illustrates the plastic behavior and spans from point A to point B which is called the semi plastic portion, where the pile head movement increases rapidly with a small rate of load increment [13].

This definition is sometimes inadequate because plunging failure requires large movement, and the ultimate load is often less a function of the capacity of the pile-soil system, and rather a function of the man-pump systems [14-16].

In Fig. 2, several types of load-settlement diagrams are shown. The ideal plunging here has only occurred in diagram C. In diagram D, the residual resistance is assumed to be equal to the plunging failure load. In other cases some interpretation criteria must be employed.

There are several methods to predict the pile failure or ultimate load from pile load test results. Three of them, Davisson offset limit load, 80% Brinch Hansen criterion and Chin-Kondner are suggested by most of the geotechnical engineering handbooks [17, 18]. Based on the analyses by Fellenius [14-16], the best method to simulate the plunging failure is Brinch Hansen's 80% criterion. In another analysis conducted by some other authors, using a database consisting of 30 case studies of a pile load test that accomplished plunging failure, six interpretation methods were compared, and the same results as those obtained by Fellenius were concluded [11].

In most cases, the value of N presents a relatively wide range of variations due to the heterogeneity of soil layers. In order to obtain proper unit shaft and base resistances it is very important to consider the variations of soil resistance properties by presenting an average value for N. Since unit shaft and base resistances are related to the average value of N, this value should be a pertinent representative. Usually two methods of averaging, arithmetical and geometrical, are used to find the mean value of a series of numerals.

The arithmetical average is calculated as follows:

$$N_a = \frac{(N_1 + N_2 + ... + N_n)}{n} \tag{1}$$

In which N_a is the arithmetical average of N_1 to N_n. The geometrical average (geo mean) is calculated as follows:

$$N_g = (N_1 \times N_2 \times ... \times N_n)^{1/n} \tag{2}$$

In which N_g is the geometrical average of N_1 to N_n.

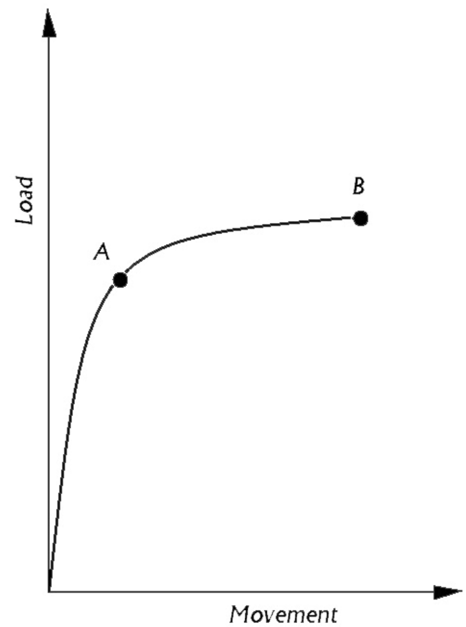

Fig. 1. Ideal plunging failure load-set diagram

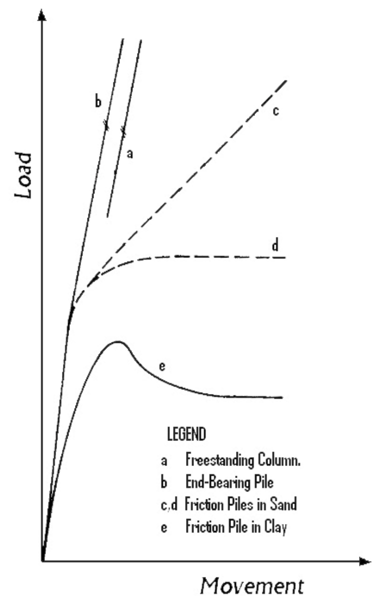

Fig. 2. Various types of load-set diagrams [14]

For example, 26.8 and 15 are the arithmetic and geometric averages, respectively, of 2, 3, 15, 14, 15, 17, 15, 17, 70 and 100. This example demonstrates that the value of the geometrical average is closer to the predominant values of these numerals, since the arithmetical value is highly affected by the values 70 and 100.

As a result, using the geometrical average method to obtain the logical representative of N values seems to be more accurate and relevant [1]. It should be noticed that the SPT values used for the geometric average should be at a constant spacing.

In order to obtain the unit base resistance of piles from standard penetration test results, the failure zone and failure mechanism should be specified around the base of the pile. The object is the simulation of the punctuate failure at the bottom of the pile. Eslami & Fellenius [1] used a model for local failure simulation, which is a spiral logarithmic surface starting at the base of the pile, and ending at one point on its body. The height and depth of this spiral logarithmic surface can vary between four to nine times the pile diameter (4-9B) at the upper part of the pile, and between one and 1.5 times the pile diameter (1-1.5B) below the base, depending on the soil friction angle. In case the confining soil is heterogeneous, this failure pattern can not be generalized for the failure that occurs around the pile base (Fig. 3).

In this study, a process of trial and error was pursued among the presumed patterns in order to reach a suitable failure pattern regarding the log-spiral rupture surface. The failure surface patterns were considered to be 2B, 4B and 8B, both at the upper and lower parts of the pile base level, while the last failure surface pattern was considered 8B at the upper part and 4B at the lower part of the pile, in which B is the pile diameter. This criterion seems to lead to consistent output in comparison to the investigations of Eslami & Fellenius [1].

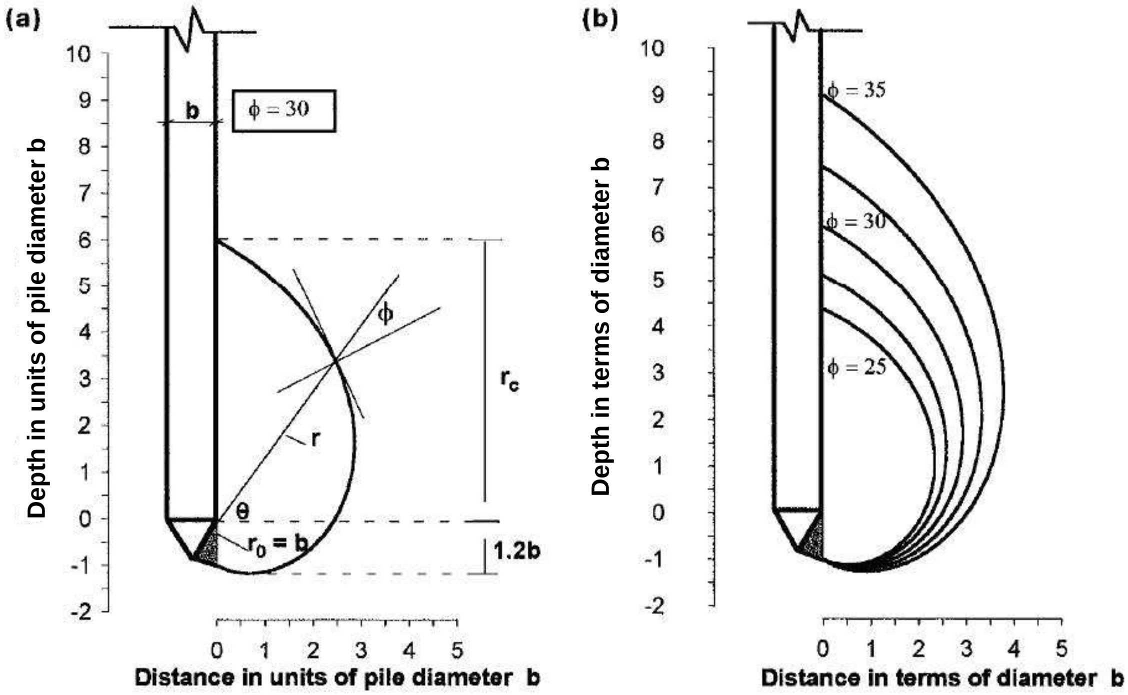

Fig. 3. a) Schematic view of spiral logarithmic failure surface around the base, b) failure surface for various values of Φ [1]

Besides, the new method is developed regarding the geometric mean of N-values around the pile, failure zone extending 8B above, and 4B below the pile base level and plunging failure based on Brinch Hansen 80% criterion by calibration of 15 case histories. The proposed formula for bearing capacity of the pile is:

$$Q_u = Q_t + Q_s = 385 * N_{g.b} * A_b + 3.65 * N_{g.s} * A_s \tag{3}$$

Where Q_u (kN) is the ultimate bearing capacity of the pile, Q_t the pile base capacity, Q_s the pile shaft capacity, and $N_{g.b}$ is the geometrical average of N values within the zone surrounded by a spiral surface with 8B at the upper part and 4B at the lower part, near the base. Also, A_b (m^2) is the section area of the pile, $N_{g.s}$ is the geometrical average of the N values along the pile, and A_s (m^2) is the pile cross section surface area. Since the applied loading tests results are from piles generally in sandy soils, the proposed formula should, preferably, be used for piles driven in cohesionless soil types.

4. COMPARISON WITH CPT-BASED METHODS

There is no doubt that CPT data are more reliable than SPT to estimate the bearing capacity of a pile. It is because the CPT is simple, fast, and relatively economical. It provides continuous records with depth, and the results are interpretable on both an empirical and an analytical basis. Therefore, the CPT is becoming the preferred type of penetration test for pile analysis. By analogy of a cone penetrometer as a model pile, the measured cone resistance and sleeve friction can be employed to estimate the unit base and shaft resistances, respectively. Among several methods to estimate the bearing capacity of piles based on CPT, the Eslami & Fellenius [1, 3] approach is a recent method that represents a more reliable prediction for pile bearing capacity.

Validation of this method by a data base consisting of 102 full scale pile load tests demonstrates that their methodology has the highest accuracy and the lowest scatter for predicting the pile bearing capacity among other CPT and CPTu based methods.

The preliminary assumptions to represent the new SPT method are the same as Eslami & Fellenius [1]. Their method suggests that the pile unit base and shaft resistances be determined from the effective cone resistance, q_c, as follows:

$$r_t = C_t q_{cg} \tag{4}$$

$$r_s = C_s q_{cg} \tag{5}$$

r_t and r_s are unit base and unit shaft friction resistances; C_t is a correlation factor to estimate the base bearing capacity that is equal to 0.98; and C_s is a correlation factor to estimate the skin bearing capacity equal to 0.01 for sandy soil. The q_{cg} is the geometrical average of q_c values within the influence zone surrounding the pile base almost 8B to 12B; and $q_{c.g}$ is the average of cone point resistances.

Robertson *et al*. [4] suggested the values of the $(q_c/p_a)/N_{60}$ ratio shown in Table 2 for each soil classification zone. These values provide a reasonable estimate of SPT N_{60} values from CPT point resistance, q_c.

Table 2. The ratio of $(q_c/p_a)/N_{60}$ for various types of soil [4]

Zone	Soil behaviour type	$(q_c/p_a)/N_{60}$
1	Sensitive fine grained	2
2	Organic material	1
3	Clay	1
4	Silty clay to clay	1.5
5	Clayey silt to silty clay	2
6	Sandy silt to clayey silt	2.5
7	Slity sand to sandy silt	3
8	Sand to slity sand	4
9	Sand	5
10	Gravely sand to sand	6
11	Very stiff fine grained	1
12	Sand to clayey sand	2

Since driven piles in sand and silty sand soils have been studied here, the value of the $(q_c/p_a)/N_{60}$ ratio is about 4 from Table 2. Rearranging this equation, and noting that p_a (air pressure) is almost 100 kPa, the following equation can be derived:

$$q_c\,(kPa) = 400 * N_{60} \quad q_c\,(kpa) = 400\,N_{60} \tag{6}$$

Substituting in Eslami & Fellenius's [1] equation:

$$r_t\,(kpa) = 392\,N_{60} \tag{7}$$

$$r_s\,(kpa) = 4\,N_{60} \tag{8}$$

It can be seen that the latter equation is very similar to the new SPT method, and indicates that the proposed method is a fairly consistent approach to evaluate the bearing capacity of piles based on the SPT N-value of CPT-SPT equivalent data.

5. DATA BASE AND CASE RECORDS

A data base was compiled of 60 pile case histories and SPT borehole results close to pile locations. The cases comprise 43 full scale pile load tests and 17 dynamic tests with CAPWAP analysis. These case histories were collected from 18 sources reporting data from 26 sites from 7 countries. Table 3 summarizes the main case records data for reference, pile characteristics, pile loading test results, and soil profiles.

Table 3. Summary of data base records

No.	Case	Reference	Site location	Material	b(mm)	Length (m)	R_{ult} (kN)	Soil profile
1	A&M 14	[1, 11]	L.M.S, USA	HP, St.	246	8.5	590	Clay & Sand
2	A&M 39	[1, 11]	L.M.S, USA	HP, St.	310	19	1370	Clay & Sand
3	A&M 40	[1, 11]	L.M.S, USA	Sq, Conc.	350	16	1070	Clay & Sand
4	A&M 41	[1, 11]	L.M.S, USA	HP, St.	310	12.4	520	Clay & Sand
5	A&M 49	[1, 11]	L.M.S, USA	Sq, Conc.	400	14.7	1170	Sand
6	A&M 66	[1, 11]	L.M.S, USA	Sq, Conc.	350	25	1560	Clay & Sand
7	A&N1	[1, 11]	VIC, Australia	Sq, Conc.	450	14	3850	Sand & Limestone
8	A&N2	[1, 11]	VIC, Australia	Sq, Conc.	450	13.75	4250	Sand & Limestone
9	A&N3	[1, 11]	VIC, Australia	Sq, Conc.	355	10.2	1300	Silt & Sand
10	A&P1	[11]	Asaloye, Iran	P, St.	1424.4	14.6	6450	Sand
11	A&P2	[11]	Asaloye, Iran	P ,St.	1424.4	14.6	1470	Sand
12	A&P3	[11]	Asaloye, Iran	P ,St.	1424.4	18.5	2550	Sand
13	ALABA	[1, 11]	Alabama, USA	HP, St.	310	36.3	2130	Silty clay & Sand
14	BOOSH1	[11]	Booshehr, Iran	P, St.	457	24	2230	Silty clay & Sand
15	BOOSH2	[11]	Booshehr, Iran	P, St.	457	24	1200	Silty clay & Sand
16	B.A.1	[19]	B.Abbas, Iran	P, St.	1000	15	2880	Clay & Sand
17	B.A.2	[19]	B.Abbas, Iran	P, St.	1000	18	3500	Clay & Sand
18	B.A.3	[19]	B.Abbas, Iran	P, St.	1000	15	3000	Clay & Sand
19	B.A.4	[19]	B.Abbas, Iran	P, St.	1000	18	3000	Clay & Sand
20	B.A.5	[19]	B.Abbas, Iran	P, St.	1000	15	2000	Clay & Sand
21	B.A.6	[19]	B.Abbas, Iran	P, St.	1000	18	2000	Clay & Sand
22	B.A.7	[19]	B.Abbas, Iran	P, St.	1000	18	5000	Clay & Sand
23	B.A.8	[20]	B.Abbas, Iran	P, St.	1200	20.5	6300*	Clay & Sand

Table 3. (Continued).

24	B.A.9	[20]	B.Abbas, Iran	P, St.	1200	20.5	6605*	Clay & Sand
25	B.A.10	[20]	B.Abbas, Iran	P, St.	1200	20.5	6500*	Clay & Sand
26	B.A.11	[20]	B.Abbas, Iran	P, St.	1200	20.5	6150*	Clay & Sand
27	B.A.12	[20]	B.Abbas, Iran	P, St.	1200	20.5	6230*	Clay & Sand
28	B.A.13	[20]	B.Abbas, Iran	P, St.	1200	20.5	6650*	Clay & Sand
29	B.A.14	[20]	B.Abbas, Iran	P, St.	1200	20.5	6550*	Clay & Sand
30	B.A.15	[20]	B.Abbas, Iran	P, St.	1200	21.5	7150*	Clay & Sand
31	B.A.16	[20]	B.Abbas, Iran	P, St.	1200	21.5	7255*	Clay & Sand
32	B.A.17	[20]	B.Abbas, Iran	P, St.	1200	21.5	7100*	Clay & Sand
33	B.A.18	[20]	B.Abbas, Iran	P, St.	1200	21.5	7300*	Clay & Sand
34	B.A.19	[20]	B.Abbas, Iran	P, St.	1200	21.5	7705*	Clay & Sand
35	B.A.20	[20]	B.Abbas, Iran	P, St.	1200	22.5	8400*	Clay & Sand
36	B.A.21	[20]	B.Abbas, Iran	P, St.	1200	22.5	8305*	Clay & Sand
37	B.A.22	[20]	B.Abbas, Iran	P, St.	1200	22.5	8350*	Clay & Sand
38	B.A.23	[20]	B.Abbas, Iran	P, St.	1200	22.5	8500*	Clay & Sand
39	B.A.24	[20]	B.Abbas, Iran	P, St.	1200	22.5	8105*	Clay & Sand
40	B.A.25	[20]	B.Abbas, Iran	P, St.	1200	22.5	8050*	Clay & Sand
41	B.A.26	[20]	B.Abbas, Iran	P, St.	1200	24	9205*	Clay & Sand
42	B.A.27	[20]	B.Abbas, Iran	P, St.	1200	24	9350*	Clay & Sand
43	B.A.28	[20]	B.Abbas, Iran	P, St.	1200	24	9405*	Clay & Sand
44	B.A.29	[20]	B.Abbas, Iran	P, St.	1200	24	9150*	Clay & Sand
45	B.A.30	[20]	B.Abbas, Iran	P, St.	1200	25	1025*	Clay & Sand
46	B.A.31	[20]	B.Abbas, Iran	P, St.	1200	25	10505*	Clay & Sand
47	Fhwasf	[1,11]	S.F, USA	P, St.	273	9.1	490	Sand
48	Khoz 1	[11]	Ahwaz, Iran	Sq, Conc.	350	30	1400	Clay & Sily sand
49	Khoz 2	[11]	Ahwaz, Iran	Sq, Conc.	500	30	2025	Clay & Sily sand
50	Khoz 3	[11]	Ahwaz, Iran	Sq, Conc.	350	30	1050	Clay & Sily sand
51	Kp1	[1,11]	Kal, Belgium	HP, St.	368	14	3500	Sand

Table 3. (Continued).

52	L & D12	[1,11]	L&D, USA	HP, St.	360	16.5	1170	Sand
53	L &D13A	[1,11]	L & D, USA	HP, St.	360	16.5	2900	Sand
54	L & D16	[1,11]	L & D, USA	HP, St.	360	16.2	3600	Sand
55	L & D31	[1,11]	L & D, USA	P, St.	300	14.2	1310	Sand
56	L & D315	[1,11]	L & D, USA	HP, St.	360	11.3	817	Sand
57	L & D316	[1,11]	L & D, USA	HP, St.	360	11.3	870	Sand
58	L & D32	[1,11]	L & D, USA	P, St.	300	11	560	Sand
59	L & D35	[1,11]	L & D, USA	P, St.	350	12.2	360	Sand
60	Rasht 2	[11]	Rasht, Iran	Sq, Conc.	300	30	1600	Clay & Sand
P=Pipe, Sq=Square, HP=H-Section, Conc=Concrete, St=Steel, b=Diameter, D=Embedment Length, R_{ult}=Total capacity, *:Dynamic testing								

The piles were generally driven in sandy soils, and for some cases there were thin layers of loam and clay along the pile shaft, although all of them have entered into a granular soil layer in a depth at least ten times more than their diameter. The cumulative thickness of clay layers did not exceed 10% of the pile length for a few cases. In Fig. 4, two typical pile driving diagrams within the data base are shown.

The SPT tests were carried out close to pile locations, and properly represent the geotechnical characteristics of the surrounding soil (Fig. 5).

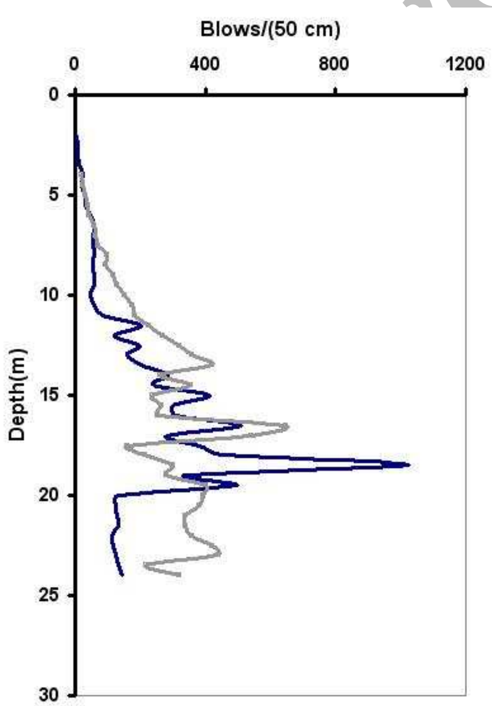

Fig. 4. Two typical pile driving diagrams within data base [20]

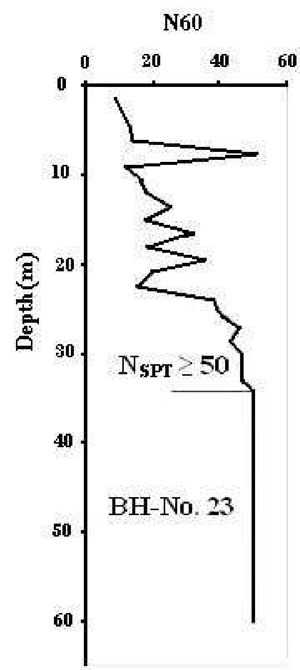

Fig. 5. A typical SPT test diagram within data base [21]

The embedment lengths of piles vary from 8.5 to 34.2 m; the pile diameters from 246 to 1424 mm, while the pile capacities range from 520 to 6450 kN. The piles are mainly made of steel, and most have a round cross section. The majority of accomplished loading tests on piles were compressive, although in 11 cases pullout tests have been performed.

The loading procedure in all pile load tests was Slow Maintained Load, SML, for which the load-movement diagram is illustrated in Fig 6. In all static pile load tests the minimum time span between the end of pile driving and the start of the pile load test was one month. Since the standard penetration tests are ranked as large strain tests, the recorded load in complete plunging was considered as the ultimate bearing capacity of the pile in those cases. In a few cases that complete plunging did not occur, the 80% criterion of Brinch Hansen was considered for ultimate capacity determination.

It is also important to note that in the case of dynamic tests, the time span between the end of construction and when the restrike test is taken is about one month; and using a strong hammer, the rate of pile penetrations per drop during the test exceeds minimum quake. Therefore, the bearing capacity of piles was fully mobilized, as presented for a typical case (Fig. 7).

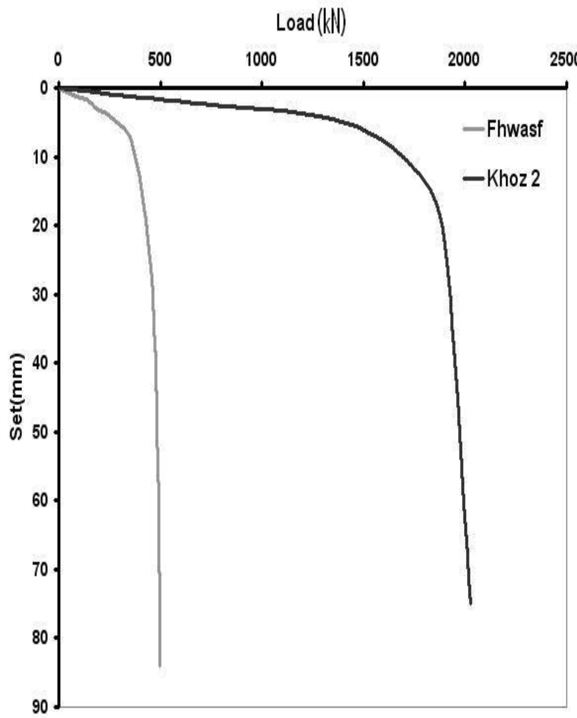

Fig. 6. Two typical load test diagrams within data base [1,11]

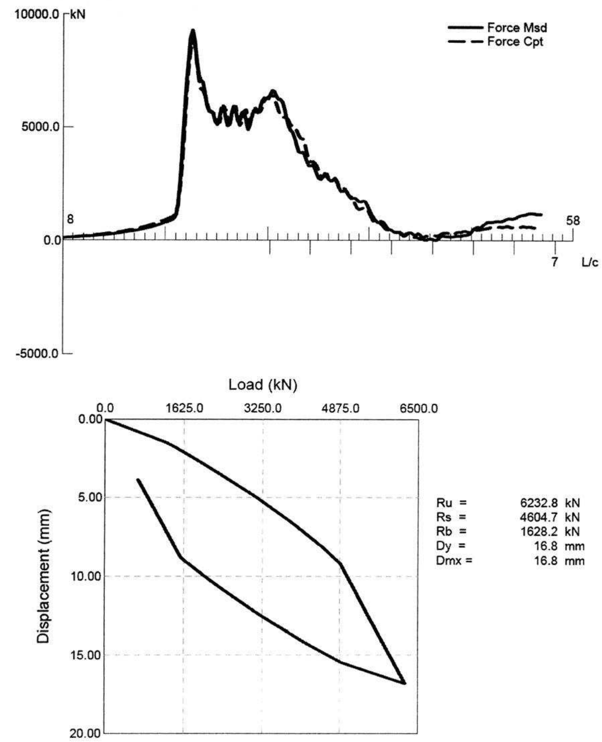

Fig. 7. The CAPWAP analysis result of case No. 27 , B.A.12 [20]

6. VALIDATIONS

Statistical and probability approaches were engaged to verify the SPT predictive methods. Cumulative probability and Log-Normal methods have been considered to compare different approaches of pile capacity determination. According to the cumulative probability approach, the ratio of the predicted value (Q_p) to the measured value (Q_m) has been drawn versus cumulative probability [1, 22]. For a series of numerals, Q_p/Q_m has been set ascending and indexed with 1 to n. Then for each of the relative amounts, the cumulative probability factor has been calculated as follows:

$$P\ (\%) = \frac{i}{n+1} 100 \tag{9}$$

Where P is the cumulative probability factor, *i* is the index of the considered case, and *n* is the number of total cases. To determine the convergence or deviation tendency of the output of prediction, the following criteria have been referred:

- The value of Qp/Qm at the cumulative probability of 50% is a measurement of the tendency to overestimate or underestimate the pile capacity. The closer to a ratio of unity, the better the agreement. To estimate the average error, the following equation can be used:

$$E_{ave} = \left(\frac{Q_p}{Q_m}\right)_{\%50} - 1 \tag{10}$$

- The slope of the line through the data points is a measurement of the dispersion or standard deviation. The flatter the line, the better the general agreement.

The result of cumulative probability analysis is shown in Fig. 8. The error estimation of the proposed method and five other methods are summarized in Table 4.

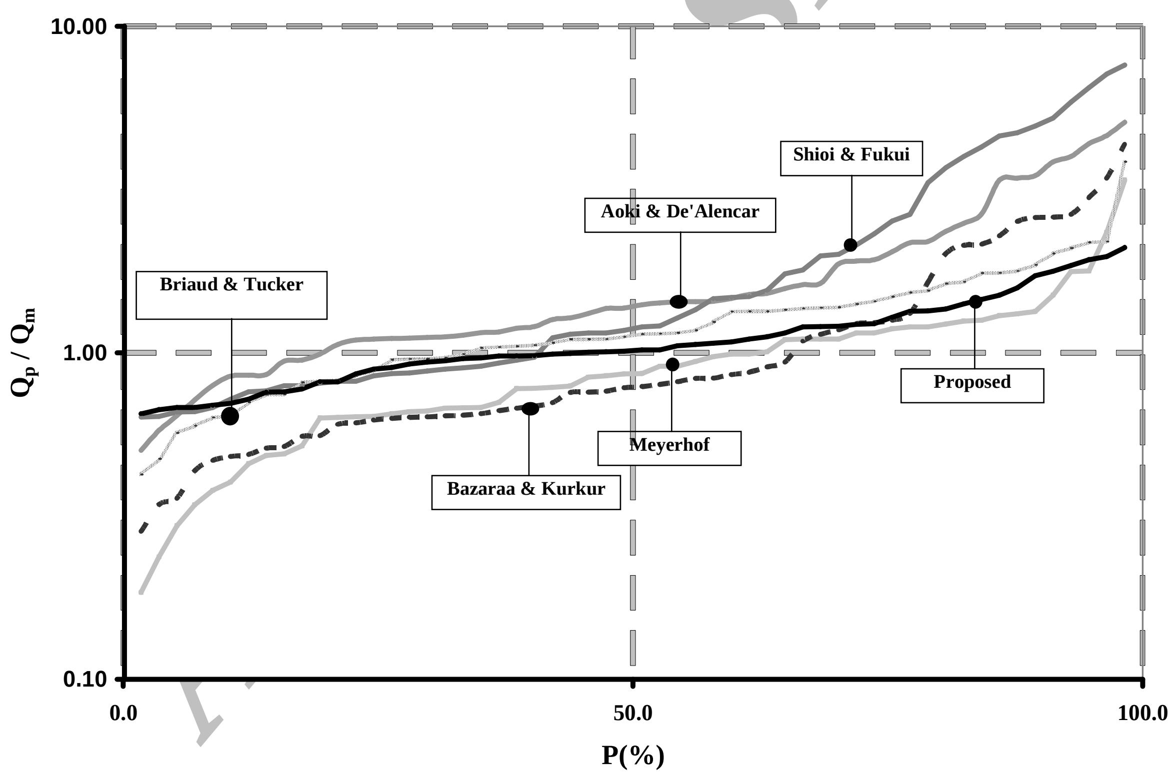

Fig. 8. Cumulative probability diagram for different methods of pile capacity determination

The results of the comparison showed that the proposed method predicted closer values for bearing capacity to the actual values of cases among other methods. The error of the proposed method is about 8%, while this is 18% for Briaud & Tucker [23], 17% for the Meyerhof [8] methods, and 43% for the Aoki & De’Alencar [6] method.

The low scatter of prediction is another advantage of the proposed method in comparison with the other SPT current methods. Based on this analysis, the Aoki & De’Alencar [6] prediction is highly overestimating. This overestimation can be due to its failure criterion. Among the five SPT methods, the predictions by Meyerhof [8] and Bazaraa & Kurkur [10] are conservative. This is due to ignoring the plug effect in pipe piles that are categorized as low displacement piles in this method. In the Bazaraa & Kurkur [10] method, the error is related to the energy ratio of SPT blows and the type of failure criterion to assess the pile bearing capacity being ignored. Ignoring the energy ratio can also be a source of error in the Briaud & Tucker [9] method.

Table 4. Relative error for five current methods and the proposed methods

No.	Method	Error (%)	Description
1	Proposed	8	Over estimate
2	Briaud & Tucker	18	Over estimate
3	Meyerhof	-17	Under estimate
4	Shioi & Fukui	26	Over estimate
5	Bazaraa & Kurkur	-23	Under estimate
6	Aoki & De'Alencar	41	Over estimate

The log normal distribution can be employed to evaluate the performance of the pile capacity prediction method [23]. The log normal distribution is acceptable to represent the ratio of Q_p/Q_m; however, it is not symmetric around the mean, which means that the Log Normal distribution does not give an equal weight for under prediction and over prediction. In order to use Log Normal distribution, the mean (μ_{ln}) and standard deviation (σ_{ln}) are evaluated for the natural logarithm of Q_p/Q_m as follows:

$$\mu_{\ln}\left(\frac{Q_p}{Q_m}\right)=\frac{1}{n}\sum_{i=1}^{n}\ln\left(\frac{Q_p}{Q_m}\right) \tag{11}$$

$$\sigma_{\ln}\left(\frac{Q_p}{Q_m}\right)=\sqrt{\frac{1}{n-1}\sum_{i=1}^{n}\left(\ln\left(\frac{Q_p}{Q_m}\right)_i-\mu_{\ln}\right)^2} \tag{12}$$

The ratio Q_p/Q_m and the natural logarithm of the ratio $\ln(Q_p/Q_m)$ for each pile were calculated. Then, the mean (μ_{ln}) and standard deviation (σ_{ln}), and the coefficient of variation (COV) of $\ln(Q_p/Q_m)$ for each method were determined.

The Log normal distribution is defined as the distribution with the following density:

$$\mathrm{f}(\mathrm{x})=\frac{1}{\sqrt{2\pi}\sigma_{\ln}\mathrm{x}}\mathrm{Exp}\left(-\frac{1}{2}\left(\frac{\ln(\mathrm{x})-\mu_{\ln}}{\sigma_{\ln}}\right)^2\right) \tag{13}$$

where $x=(Q_p/Q_m)$, μ_{ln} is the mean of $\ln(Q_p/Q_m)$, and σ_{ln} is the standard deviation of $\ln(Q_p/Q_m)$. The Log Normal distribution was used to evaluate the different methods based on their prediction accuracy and precision. Fig. 9 shows the Log Normal distribution for different methods considered in this paper, which confirms the results of cumulative probability analysis.

Based on the Log Normal distribution analysis, the probability that predictions fall within a ±25% accuracy level in these methods can be estimated as follows:

$$\mathrm{P}(\%)=100\int_{0.75}^{1.25}\mathrm{f}(\mathrm{x})\mathrm{dx} \tag{14}$$

The results of this analysis are presented in Table 5. These results indicate that the proposed method has a better precision than others in predicting the pile bearing capacity.

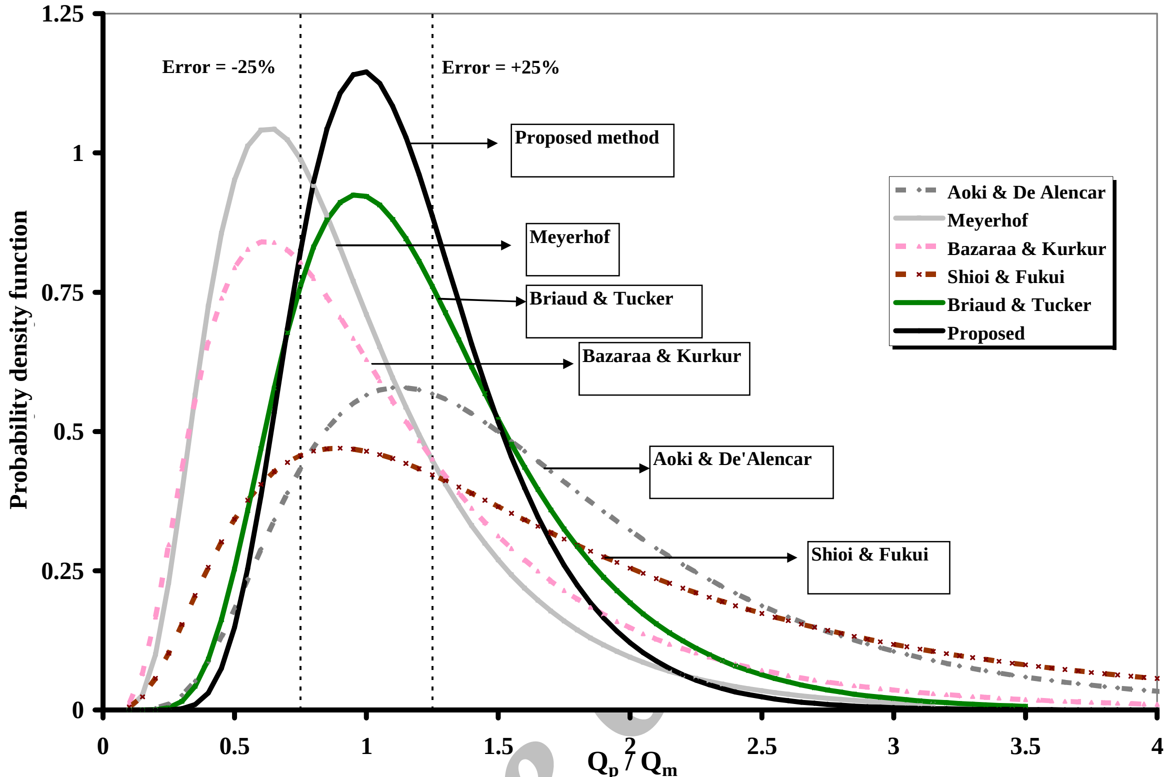

Fig. 9. Log normal distribution diagram for different methods of pile capacity determination

Table 5. The probability of estimating within ±25% for five and proposed methods

No.	Method	Probability of estimating within ±25 % error (%)
1	Aoki & De'Alencar	27
2	Meyerhof	36
3	Bazaraa & Kurkur	30
4	Shioi & Fukui	20
5	Briaud & Tucker	42
6	Proposed Method	63

7. CONCLUSION

Determining the bearing capacity of piles is an interesting subject in geotechnical engineering. The complex nature of the embedment ground of piles and lack of suitable analytical models for predicting the pile bearing capacity are the main reasons for the geotechnical engineer's tendency to peruse further research on this subject.

Among different common methods, pile load testing and dynamic tests with a pile driving analyzer and a signal matching process can represent reasonable results, but such tests are expensive, time-consuming, and the costs are often difficult to justify for ordinary or small projects. Direct bearing capacity predicting methods for piles are developed based on in-situ testing data, especially SPT and CPT, having applications that have shown an increase in recent years. SPT test is the most frequent in-situ test in geotechnical practice because of its simplicity, easy performance, short time, and low cost.

Current methods to estimate the pile capacity based on SPT involve some shortcomings. To overcome these deficiencies a new approach has been developed by considering failure zone extension, data processing and plunging failure mechanism.

Using (qc/pa)/N60 ratios suggested by Robertson et al., and converting the Eslami & Fellenius CPTu based method to SPT format has shown that the new method has an acceptable precision in estimating the bearing capacity of piles.

Besides, by using a data base that consisted of 6o pile case histories including 43 full scale static load tests, 17 dynamic tests, and SPT data at a minimum distance from the pile location, the predictive methods were compared and verified.

A comparison of the current methods has been performed by error investigation with cumulative probability and Log-Normal approaches. The results of the comparison demonstrate that the error of the new method is within an acceptable range, and the variance is low in contrast with other methods. The Meyerhof and Briaud & Tucker methods also predict the capacity with reasonable accuracy.
Therefore, a new approach based on SPT N-value or SPT-CPT equivalent parameters can be considered in geotechnical practice.

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