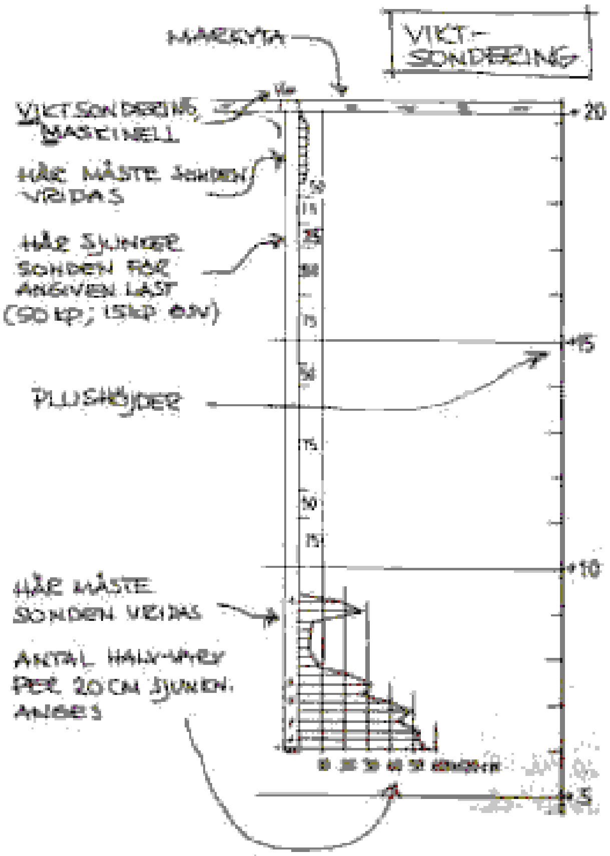

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Pile capacity by direct CPT and CPTu methods applied to 102 case histories

Abolfazl Eslami and Bengt H. Fellenius

Abstract: Six methods to determine axial pile capacity directly from cone penetration test (CPT) data are presented, discussed, and compared. Five of the methods are CPT methods that apply total stress and a filtered arithmetic average of cone resistance. One is a recently developed method, CPTu, that considers pore-water pressure and applies an unfiltered geometric average of cone resistance. To determine unit shaft resistance, the new method uses a new soil profiling chart based on CPTu data. The six methods are applied to 102 case histories combining CPTu data and capacities obtained in static loading tests in compression and tension. The pile capacities range from 80 to 8000 kN. The soil profiles range from soft to stiff clay, medium to dense sand, and mixtures of clay, silt, and sand. The pile embedment lengths range from 5 to 67 m and the pile diameters range from 200 to 900 mm. The new CPTu method for determining pile capacity demonstrates better agreement with the capacity determined in a static loading test and less scatter than by CPT methods.

Key words: cone penetration test, pile capacity, toe resistance, shaft resistance, soil classification.

Résumé : Six méthodes utilisées pour déterminer la capacité axiale d'un pieu à partir des résultats de l'essai de pénétration au cône sont présentées, discutées et comparées. Cinq de ces méthodes sont des méthodes CPT (essai de pénétration au cône) qui s'expriment en contraintes totales et par une moyenne arithmétique filtrée de la résistance en pointe. La dernière méthode, plus récente, est basée sur le CPTu (piézocône) et prend en compte la pression interstitielle et une moyenne géométrique non filtrée de la résistance en pointe. Dans cette méthode, la résistance du fût par unité de surface est déterminée par un nouvel abaque qui profile le sol en fonction des résultats CPTu. Les six méthodes ont été appliquées à 102 cas en combinant les résultats CPTu avec les capacités obtenues lors d'essais de chargement statique en compression et en tension. Les capacités des pieux vont de 80 kN à 8000 kN. Parmi les sols on trouve des argiles molles à raides, des sables moyennement lâches à denses et des mélanges d'argile, de silt et de sable. Les longueurs d'enfouissement des pieux varient de 5 à 67 m et leur diamètre de 200 à 900 mm. La nouvelle méthode CPTu pour déterminer la capacité d'un pieu est plus proche des résultats des essais statiques et est moins dispersée que les méthodes CPT.

Mots clés : essai de pénétration au cône, piézocône, capacité d'un pieu, résistance du fût, classification des sols.
[Traduit par la rédaction]

Introduction

The geotechnical engineering practice has developed several methods and approaches to estimate axial pile capacity. The methods by necessity include simplifying assumptions and (or) empirical approaches regarding soil stratigraphy and load transfer. Therefore, the design often becomes somewhat of a guessing game and a rather subjective exercise. The work presented in this paper aims toward ameliorating the situation in the area of static analysis of load transfer, basing the approach on in situ testing using the cone penetrometer, specifically the piezocone, CPTu.

The cone penetration test (CPT) is simple, fast, and relatively economical, supplies continuous records with depth, and allows a variety of sensors to be incorporated with the penetrometer. The advantage of using CPT data for pile design, as opposed to basing the analysis on a theoretical model, is that dependency on "undisturbed" sampling and subsequent conventional laboratory testing are avoided. Moreover, it is not necessary to furnish intermediate parameters, such as earth pressure and bearing capacity coefficients, K_s and N_q. Because of similarities between the cone penetrometer and a pile, the penetrometer can be considered as a model pile. In fact, estimation of pile capacity from CPT data was one of its first applications.

Case histories from full-scale tests are compiled and analyzed by means of five direct CPT methods for pile capacity estimation employed in current North American practice and the authors' recently developed direct CPTu method.

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Case records database and soil profiling from CPTu data

A database of case histories from the results of 102 full-scale pile loading tests is compiled with information on soil type and results of CPT soundings performed close to the pile locations. The cases were obtained from 36 sources reporting data from 40 sites in 13 countries. Table 1 summarizes the main case record data as to reference, pile characteristics, pile loading test results, and soil profiles.

The majority of the case records are from the United States. The soils at the sites consist of sediments of clay (soft clay, stiff clay, silty clay, sandy clay), silt (clayey silt, sandy silt), and sand (clayey sand, silty sand, gravelly sand). About 80%

Table 1. Case record summary.

No.	Case	Reference	Site location	Pile shape and material[a]	Pile diameter, b (mm)	Embedment length, D (m)	Total capacity, R_{ult} (kN)	Soil profile
Group I								
1	UBC3	Campanella et al. 1989	B.C., Canada	P, S	324	16.8	630	Soft clay, sand
2	UBC5	Campanella et al. 1989	B.C., Canada	P, S	324	31.1	1100	Soft clay, sand, silt
3	NWUP	Finno 1989	Ill., U.S.A.	P, S	450	15.2	1020	Sand, clay
4	FHWASF	O'Neil 1988	Calif., U.S.A.	P, S	273	9.1	490	Sand
5	BGHD1	Altaee et al. 1992*a*, 1992*b*	Iraq	Sq, C	285	11.0	1000	Uniform sand
6	BGHD2	Altaee et al. 1992*a*, 1992*b*	Iraq	Sq, C	285	15.0	1600	Uniform sand
7	POLA1	CH2M Hill 1987	Calif., U.S.A.	Oct, C	610	25.8	5455	Silt, sand
8	POLA2TOE	Urkkada 1995	Calif., U.S.A.	Oct, C	610	32.5	3650	Silt, sand
9	TWNTP4	Yen et al. 1989	Taiwan	P, S	609	34.3	4330	Sand, clay, sand
10	TWNTP5	Yen et al. 1989	Taiwan	P, S	609	34.3	2500	Sand, clay, sand
11	TWNTP6	Yen et al. 1989	Taiwan	P, S	609	34.3	4460	Sand, clay, sand
12	L&D314	Briaud et al. 1989	Ill., U.S.A.	HP, S	360	12.0	1170	Sand
13	L&D35	Briaud et al. 1989	Ill., U.S.A.	P, S	350	12.2	630	Sand
14	L&D316	Briaud et al. 1989	Ill., U.S.A.	HP, S	360	11.2	870	Sand
15	L&D32	Briaud et al. 1989	Ill., U.S.A.	P, S	300	11.0	500	Sand
16	L&D38	Briaud et al. 1989	Ill., U.S.A.	P, S	400	11.1	945	Sand
17	L&D315	Briaud et al. 1989	Ill., U.S.A.	HP, S	360	11.3	817	Sand
18	A&N2	Haustorfer and Plesiotis 1988	Australia	Sq, C	450	13.7	4250	Sand
19	N&SB144	Nottingham 1975	Fla., U.S.A.	P, S	270	22.5	765	Sand
20	QBSA	Konrad and Roy 1987	Que., Canada	P, S	220	7.5	83	Sensitive clay
21	UHUC1	O'Neil 1981	Tex., U.S.A.	P, S	273	13.2	780	Clay, sandy clay
22	UHUT1	O'Neil 1981	Tex., U.S.A.	P, S	273	13.2	485	Clay, sandy clay
23	UHUC11	O'Neil 1981	Tex., U.S.A.	P, S	273	13.2	800	Clay, sandy clay
24	UHUT11	O'Neil 1981	Tex., U.S.A.	P, S	273	13.2	520	Clay, sandy clay
Group II								
25	UBC2	Campanella et al. 1989	B.C., Canada	P, S	324	13.8	290	Soft clay, sand
26	UBCA	Campanella et al. 1989	B.C., Canada	P, S	915	67.0	7500	Soft clay, sand, silt
27	NWUH	Finno 1989	Pa., U.S.A.	HP, S	450	15.2	1010	Sand, clay
28	JPNOT1	Matsumoto et al. 1995	Japan	P, S	800	8.2	4700	Sand clay (soft rock)
29	LSUA1	Tumay and Fahkroo 1981	Calif., U.S.A.	Sq, C	350	9.5	900	Sand, clay
30	LSUN11	Tumay and Fahkroo 1981	Calif., U.S.A.	Sq, C	450	36.5	2950	Silty clay, silty sand
31	LSUN15	Tumay and Fahkroo 1981	Calif., U.S.A.	P, S	400	37.5	2800	Silty clay, silty sand
32	LSUN28	Tumay and Fahkroo 1981	Calif., U.S.A.	Tr, C	500	30.5	2160	Clay, silty sand, clay
33	LSUN215	Tumay and Fahkroo 1981	Calif., U.S.A.	P, S	350	31.1	1710	Clay, silty sand, clay
34	LSUR30	Tumay and Fahkroo 1981	Calif., U.S.A.	Sq, C	750	19.8	2610	Fill, sandy clay
35	LTN741	Reese et al. 1988	Tex., U.S.A.	Rd, C	810	24.1	7830	Stiff clay, sand

Table 1 (*continued*)

No.	Case	Reference	Site location	Pile shape and material[a]	Pile diameter, b (mm)	Embedment length, D (m)	Total capacity, R_{ult} (kN)	Soil profile
36	LTN484	Tucker 1986	Calif., U.S.A.	Rd, C	450	7.6	750	Silty sand, clay, sand
37	LTN742	Reese et al. 1988	Tex., U.S.A.	Rd, C	810	24.1	5850	Stiff clay, sand
38	NETH2	Viergever 1982	The Netherlands	Sq, C	256	9.3	700	Fill, clay, silty sand
39	MILANO	Gambini 1985	Italy	P, S	330	10.0	625	Clay, silty sand, clay
40	OKLACO	Neveles and Donald 1994	Okla., U.S.A.	Rd, C	660	18.2	3600	Sand, silty clay (shale)
41	L&D31	Briaud et al. 1989	Ill., U.S.A.	P, S	300	14.2	1310	Sand
42	SEATW	Horvitz et al. 1981	Wash., U.S.A.	Rd, C	350	15.8	900	Sand
43	GIT1	Mayne 1993	Ga., U.S.A.	Rd, C	750	16.8	4500	Fill, silty sand
44	KP1	Weber 1987	Belgium	HP, S	400	14.0	3500	Soft soil, dense sand
45	MP1	Weber 1987	France	HP, S	400	14.0	2125	Soft clay, stiff clay
46	KALO14A	Van Impe et al. 1988	Belgium	Rd, C	600	12.0	5500	Peat, clay, sand
47	KALO14B	Van Impe et al. 1988	Belgium	Rd, C	600	12.0	6100	Peat, clay, sand
48	A&N3	Haustorfer and Pleslotis 1988	Australia	Sq, C	355	10.2	1300	Silt, sand, dense sand
49	USPB1	Albiero et al. 1995	Brazil	Rd, C	350	9.4	645	Clay and silt, silty sand
50	USPB2	Albiero et al. 1995	Brazil	Rd, C	400	9.4	725	Clay and silt, silty sand
51	N&SWPB1	Nottingham 1975	Fla., U.S.A.	Sq, C	450	8.0	1140	Silty sand
52	N&SWPB2	Nottingham 1975	Fla., U.S.A.	Sq, C	450	11.3	830	Silty sand
53	N&SB143	Nottingham 1975	Fla., U.S.A.	P, S	270	22.5	1620	Sand, dense sand
54	N&SB1348	Nottingham 1975	Fla., U.S.A.	Sq, C	450	14.9	1720	Sand, dense sand
55	PRS	Urkkada 1996	Puerto Rico	P, S	300	28.4	1240	Peat, sand, soft clay
56	PRL	Urkkada 1996	Puerto Rico	P, S	300	31.4	1890	Peat, sand, clay
57	UFL53	Avasarala et al. 1994	Fla., U.S.A.	Sq, C	350	20.4	1260	Sand, silt
58	MUMB	Hunt 1993	Wis.,U.S.A.	P, S	273	12.0	1686	Fill, till
59	SPB	Decourt and Niayama 1994	Brazil	Rd, C	500	8.7	3000	Silty sand
60	YOG1	Milovoc and Stevanovic 1982	Yugoslavia	Rd	520	12.0	430	Clay
61	A&N1	Haustorfer and Plesiotis 1988	Australia	Sq, C	450	14.0	3850	Dense sand, limestone
62	PNTRA5	Almeida et al. 1996	U.K.	P, S	219	25.0	190	Stiff clay, silt, clay
63	PNTRA6	Almeida et al. 1996	U.K.	P, S	219	32.5	460	Stiff clay, silt, clay
64	CVVDNC	Aimeida et al. 1996	U.K.	P, S	305	10.0	400	Stiff clay, till, silty sand
65	CWDND	Aimeida et al. 1996	U.K.	P, S	305	10.0	404	Stiff clay, till, silty sand
66	CWDNE	Almeida et al. 1996	U.K.	P, S	305	10.0	380	Stiff clay, till, silty sand
67	CWDNF	Almeida et al. 1996	U.K.	P, S	305	10.0	390	Stiff clay, till, silty sand

Table 1 (*continued*)

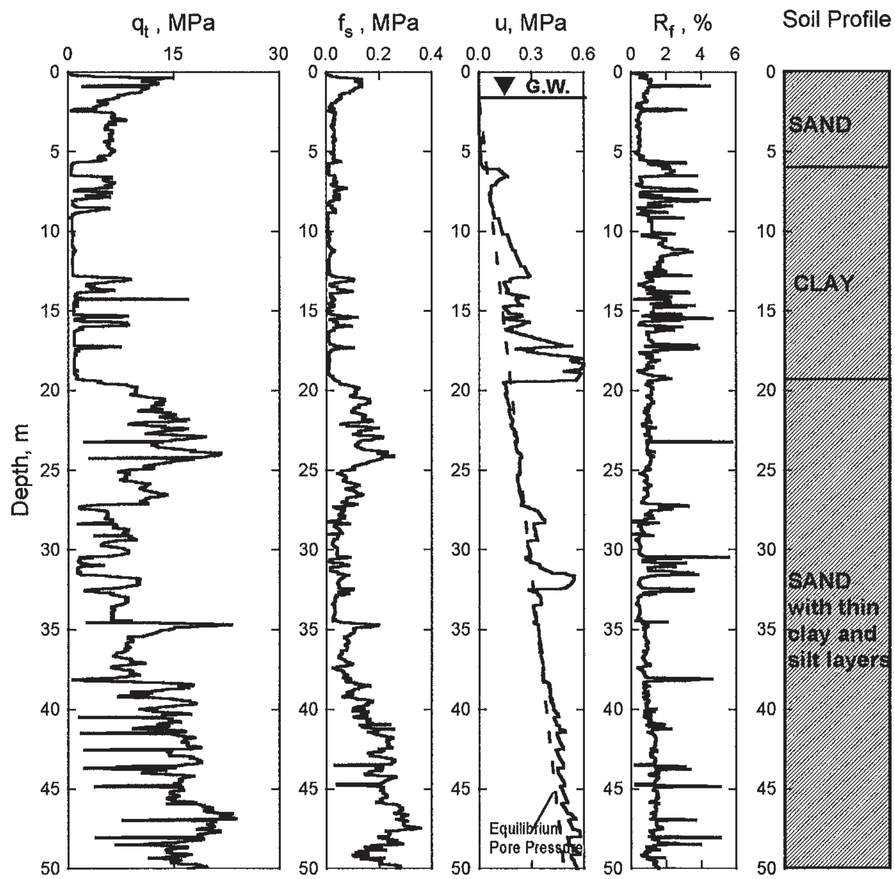
No.	Case	Reference	Site location	Pile shape and material[a]	Pile diameter, b (mm)	Embedment length, D (m)	Total capacity, R_{ult} (kN)	Soil profile
68	CWDNG	Almeida et al. 1996	U.K.	P, S	203	10.0	311	Stiff clay, till, silty sand
69	CWDNH	Almeida et al. 1996	U.K.	P, S	203	10.0	350	Stiff clay, till, silty sand
70	CWDNI	Almeida et al. 1996	U.K.	P, S	203	10.0	290	Stiff clay, till, silty sand
71	CWDNJ	Almeida et al. 1996	U.K.	P, S	203	10.0	280	Stiff clay, till, silty sand
72	CWDNK	Almeida et al. 1996	U.K.	P, S	203	10.0	350	Stiff clay, till, silty sand
73	ONSYA1	Almeida et al. 1996	Norway	P, S	219	15.0	105	Soft clay
74	ONSYB1	Almeida et al. 1996	Norway	P, S	812	15.0	444	Soft clay
75	LSTDA7	Almeida et al. 1996	Norway	P, S	219	15.0	78	Soft clay
76	LSTDA8	Almeida et al. 1996	Norway	P, S	219	22.5	86	Soft clay
77	LSTDB2	Almeida et al. 1996	Norway	P, S	812	15.0	374	Soft clay
Group III								
78	JPNOT2	Matsumoto et al. 1995	Japan	P, S	800	8.2	3190	Stiff clay (soft rock)
79	JPNOT3	Matsumoto et al. 1995	Japan	P, S	800	8.2	3250	Stiff clay (soft rock)
80	LSUB12	Tumay and Fahkroo 1981	Calif., U.S.A.	Rd, C	900	37.8	3960	Silt, silty clay, sand, silt
81	LSUN216	Tumay and Fahkroo 1981	Calif., U.S.A.	P, S	400	41.8	1890	Clay, silty sand, clay
82	LSUH1	Tumay and Fahkroo 1981	Calif., U.S.A.	Sq, C	450	29.0	1935	Clay, sand, clay
83	LSUR24	Tumay and Fahkroo 1981	Calif., U.S.A.	Sq, C	600	19.8	2025	Fill, sandy clay
84	UFL22	Avasarala et al. 1994	Fla., U.S.A.	Sq, C	350	16.0	1350	Sand
85	UFL52	Avasarala et al. 1994	Fla., U.S.A.	Sq, C	500	11.0	2070	Sand
88	OKLAST	Neveles and Donald 1994	Okla., U.S.A.	P, S	610	18.2	3850	Sand, silty clay (shale)
87	ALABA	Laier 1994	Ala., U.S.A.	HP, S	310	36.3	2130	Sand, silty clay, sand
88	L&D13A	Briaud et al. 1989	Ill., U.S.A.	HP, S	360	16.S	2900	Sand
89	L&D16	Briaud et al. 1989	Ill., U.S.A.	HP, S	360	16.2	3600	Sand
90	L&D34	Briaud et al. 1989	Ill., U.S.A.	P, S	360	14.4	1300	Sand
91	L&D37	Briaud et al. 1989	Ill., U.S.A.	P, S	400	14.6	1800	Sand
92	L&D12	Briaud et al. 1989	Ill., U.S.A.	HP, S	360	16.5	1170	Sand
93	L&D21	Briaud et al. 1989	Ill., U.S.A.	HP, S	360	16.8	1260	Sand
94	YOG2	Milovic and Stevanovic 1982	Yugoslavia	Rd	520	14.5	700	Clay
95	LTN930	Richmann and Speer 1989	Calif., U.S.A.	Rd, C	400	6.5	1385	Silty sand, sand
96	LTN938	Richmann and Speer 1989	Calif., U.S.A.	Rd, C	400	9.3	1370	Silt, gravelly sand
97	PTSER	Appendino 1981	Italy	Rd, C	508	35.8	5500	Sandy slit, dense sand
98	PT361	Appendino 1981	Italy	Rd, C	508	42.0	6000	Silty sand, clay, sand

Table 1 (*concluded*)

No.	Case	Reference	Site location	Pile shape and material[a]	Pile diameter, b (mm)	Embedment length, D (m)	Total capacity, R_{ult} (kN)	Soil profile
99	N&SJC1	Nottingham 1975	Fla., U.S.A.	Sq, C	450	9.2	1845	Sand, clay
100	N&SBI215	Nottingham 1975	Fla., U.S.A.	Sq, C	250	21.3	810	Sand, silty sand
101	N&SBI316	Nottingham 1975	Fla., U.S.A.	Sq, C	350	15.9	1485	Fill, silty sand
102	N&SBI42	Nottingham 1975	Fla., U.S.A.	P, S	270	15.2	660	Fill, sand, dense sand

[a] P, pipe; Sq, square; Oct, octagonal; HP, H section; Rd, round; Tr, triangular; C, concrete; S, steel.

Fig. 1. Typical piezocone profile (data from Yen et al. 1989). R_f, friction ratio; u, pore pressure.



of the CPT cases included in the data are obtained by electrical cone and 20% by mechanical cone. All cases from silt and clay soils, about half the total, include pore-pressure measurements. Most of the CPT measurements are at a vertical spacing of 300 mm or smaller. A typical CPTu profile taken from one of the records is shown in Fig. 1.

Most of the piles have a square or round cross section and the pile materials are steel and concrete. All but 10 of the piles were installed by driving. The pile embedment lengths range from 5 to 67 m, the pile diameters from 200 to 900 mm, and the pile capacities from 80 to 8000 kN. The cases have been separated into three groups as follows.

Group I (cases 1–24 in Table 1) includes 14 compression static loading tests, where the toe and shaft resistances were determined separately, and 10 tension tests. Thus, the database includes 14 cases of known toe resistance and 24 cases of known shaft resistance.

Group II (cases 25–77 in Table 1) includes 34 compression static loading tests for which no separation of shaft and toe resistances is reported, and 19 tension tests, where the cone data do not include records of sleeve friction.

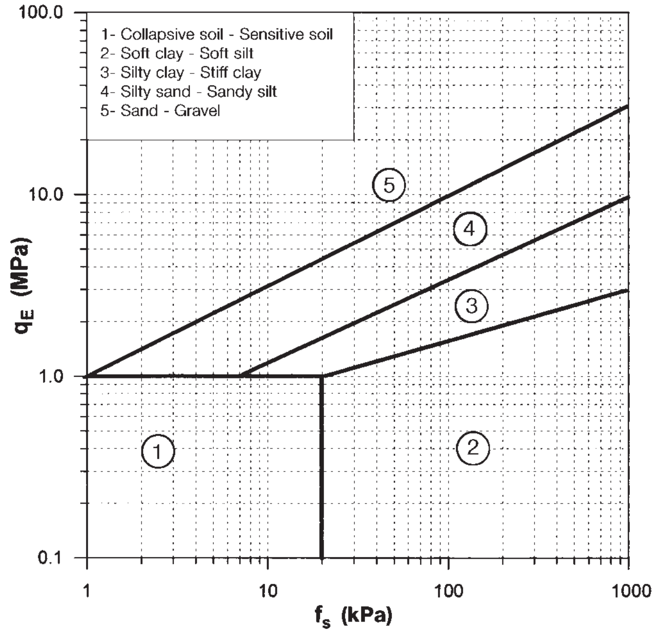
Group III (cases 78–102 in Table 1) includes cases where the ultimate resistance was not indisputably reached in the static loading test. The maximum load is, therefore, considered a lower-bound capacity in these cases. Some of the producers of the data designated the maximum test load to be the capacity of the pile, which may actually be the case for some of the tests.

A primary purpose of the CPT is to identify the soil layer boundaries and determine soil type in terms of the grain size, i.e., soil profiling. Begemann (1953, 1963, 1965) pioneered soil profiling based on mechanical cone data and pointed out that the soil type can be related to the CPT friction ratio (ratio of sleeve friction to cone resistance). The Begemann soil profiling chart presents cone resistance against sleeve friction (mechanical cone data only). Later investigations (Campanella and Robertson 1988) have shown the need for correcting the cone resistance for the pore pressure generated at the cone shoulder. The advent of the piezocone enabled soil profiling that includes cone resistance, sleeve friction, and pore pressure measurements.

Robertson et al. (1986), Robertson (1990), and Campanella et al. (1989) proposed soil profiling charts based on piezocone data by plotting the cone resistance versus friction ratio. This manner of plotting means that a variable is plotted versus its own inverse value in the charts, which violates the fundamental rule that dependent and independent variables must be rigorously separated and distorts the data. The authors prefer to use a profiling diagram similar to that presented by Begemann (1965), with two differences: First, the cone resistance, q_c, is corrected for the pore pressure acting on the shoulder. (The corrected resistance is denoted q_t.) Second, an "effective" cone resistance, q_E, is used instead of the cone resistance, q_c ($q_E = q_t - u_2$, where u_2 is the pore pressure measured behind the cone point). The diagram uses a log–log plot to magnify the relations in soft and loose soil as opposed to the linear plot used by Begemann (1965).

The database contains a large amount of cone test data, which have been plotted in a cone resistance (q_E) versus sleeve friction (f_s) diagram. The data points were found to segregate on five main soil categories: collapsive–sensitive soil, soft clay – soft silt, silty clay – stiff clay, silty sand, and sand and gravel, as delineated in Fig. 2. The boundaries shown in the diagram were obtained by enveloping approximately 90% of all points of each main soil category. A detailed presentation of the profiling method will be presented in a separate paper.

Fig. 2. New soil profiling chart.



Pile capacity from CPT data

Two main approaches for application of cone data to pile design have evolved: indirect and direct methods. Indirect CPT methods employ soil parameters, such as the friction angle and undrained shear strength estimated from the cone data as evaluated from bearing capacity and (or) cavity expansion theories, which introduce significant uncertainties. The indirect methods disregard horizontal stress, include strip-footing bearing capacity theory, and neglect soil compressibility and strain softening. The authors consider the indirect methods less suitable for use in engineering practice and will not refer to them further.

Direct CPT methods more or less equate the measured cone resistance to the pile unit resistance. As detailed below, some of the methods use the cone sleeve friction in determining unit shaft resistance. Others proportion the shaft resistance to the cone resistance. Several methods modify the resistance values to the difference in diameter between the pile and the cone. As opposed to the indirect methods, mean effective stress, soil compressibility, and rigidity affect the pile and the cone in a similar manner, which eliminates the need to supplement the field data with laboratory testing and to calculate intermediate values, such as the earth pressure coefficient, K_s, and the bearing capacity coefficient, N_q.

To relate the cone resistance to the pile unit toe resistance, current CPT methods determine the arithmetic average of the CPT data over an "influence zone." Often, the test data include a small amount of randomly distributed extreme values, "peaks and troughs," that may be representative for the response of the cone to the local soil characteristics, but not for a pile having a much larger diameter. Keeping the extreme values could result in an average that is not representative of the pile resistance at the site. Therefore, before averaging, it is common practice to manually filter and smooth the data, either by applying a "minimum path" rule (Schmertmann 1978) or, more subjectively, by simply removing the peaks and troughs from the records.

Current CPT direct methods

The following direct methods, currently used in North American practice, are considered: (*i*) Schmertmann and Nottingham, (*ii*) DeRuiter and Beringen (commonly called the European method), (*iii*) Bustamante and Gianselli (commonly called the French method), (*iv*) Meyerhof, and (*v*) Tumay and Fakhroo.

The *Schmertmann and Nottingham* method is based on a summary of the work on model and full-scale piles presented by Nottingham (1975) and Schmertmann (1978). The unit toe resistance, r_t, is taken as equal to the average of the cone resistance over an influence zone extending from 6*b* to 8*b* above the pile toe, where *b* is the pile diameter, and 0.7*b* to 4*b* below the pile toe (see eq. [1]). The average is determined after first filtering the q_c data to "minimum-path" values. Details on the filtering and minimum-path rules are given by Schmertmann (1978). An upper limit of 15 MPa is imposed for the unit toe resistance.

$$[1] \qquad r_t = C_{OCR}\, q_{ca}$$

where r_t is the pile unit toe resistance; C_{OCR} is the correlation coefficient governed by the overconsolidation ratio, OCR, of the soil; and q_{ca} is the arithmetic average of q_c in an influence zone.

The extent of the influence zone depends on the trend of the q_c values and follows recommendations by Begemann

(1963), who based the zone extent on an assumed logarithmic spiral failure pattern for the pile toe.

The pile unit shaft resistance, r_s, may be determined from the sleeve friction as expressed by [2]:

$$[2] \qquad r_s = K f_s$$

where K is a dimensionless coefficient. The K coefficient depends on pile shape and material, cone type, and embedment ratio. In sand, the K coefficient ranges from 0.8 to 2.0, and in clay it ranges from 0.2 to 1.25. Within a depth of the first eight pile diameters below the ground surface, the unit shaft resistance is linearly interpolated from zero at the ground surface to the value given by [2].

Alternatively, in sand, but not in clay, the shaft resistance may be determined from the cone resistance:

$$[3] \qquad r_s = C q_c$$

where C is a dimensionless coefficient, which is a function of the pile type and ranges from 0.8 to 1.8%. An upper limit of 120 kPa is imposed on the unit shaft resistance, r_s, whether determined by [2] or [3]. For uplift capacity (tension resistance), the shaft resistance is reduced to 70% of that determined by [2] or [3].

The *European method* (DeRuiter and Beringen 1979) is based on experience gained from offshore construction in the North Sea. For unit toe resistance of a pile in sand, the method is the same as the Schmertmann and Nottingham method. In clay, the unit toe resistance is determined from total stress analysis according to conventional bearing capacity theory as indicated in [4] and [5]:

$$[4] \qquad r_t = N_c S_u$$

$$[5] \qquad S_u = \frac{q_c}{N_k}$$

where N_c is the conventional bearing capacity factor; S_u is the undrained shear strength; and N_K is a dimensionless coefficient, ranging from 15 to 20, reflecting local experience. An upper limit of 15 MPa is imposed for the unit toe resistance. Schmertmann (1978) also states, but without providing details, that the toe resistance value is governed by the overconsolidation ratio, OCR, of the soil.

The unit shaft resistance in sand is determined by either [1] with $K = 1$ or [2] with $C = 0.3\%$. In clay, the unit shaft resistance may also be determined from the undrained shear strength, S_u, as given in [6]:

$$[6] \qquad r_s = \alpha S_u$$

where α is the adhesion factor equal to 1.0 and 0.5 for normally consolidated and overconsolidated clays, respectively.

An upper limit of 120 kPa is imposed on the unit shaft resistance. For tension capacity, the shaft resistance is reduced to 75% of the shaft resistance in compression.

The *French method* (Bustamante and Gianeselli 1982) is based on experimental work by Laboratoire Central des Ponts et Chausees (LCPC). The sleeve friction, f_s, is neglected and the unit toe and unit shaft resistances are both determined from the average cone resistance, q_c. Bustamante and Gianeselli (1982) provide detailed filtering rules for selecting the average cone resistance. The unit toe resistance, r_t, is estimated to range from 40 to 55% of the average value of q_c over a zone of 1.5b above and 1.5b below the pile toe (b is pile diameter).

The unit shaft resistance is determined from [3] with the C coefficient ranging from 0.5 to 3.0%, as governed by the magnitude of the cone resistance, type of soil, and type of pile. Upper limits of the unit shaft resistance are imposed, ranging from 15 to 120 kPa depending on soil type, pile type, and pile installation method.

The *Meyerhof method* (Meyerhof 1956, 1976, 1983) is based on theoretical and experimental studies of deep foundations in sand. The unit toe resistance in sand is given by [7], and the influence of scale effect of piles and shallow penetration in dense sand strata is considered by applying two modification factors, C_1 and C_2, to the q_c average. The unit toe resistance of a bored pile is reduced to 30% of that determined from [7]:

$$[7] \qquad r_t = q_{ca} C_1 C_2$$

where q_{ca} is the arithmetic average of q_c in a zone ranging from 4b above to 1b below the pile toe; $C_1 = [(b + 0.5)/2b]^n$ is a modification factor for scale effect when $b > 0.5$ m, otherwise $C_1 = 1$; $C_2 = D_b/10b$ is a modification for penetration into dense strata when $D_b < 10b$, otherwise $C_2 = 1$; n is an exponent equal to 1 for loose sand, 2 for medium dense sand, and 3 for dense sand; and D_b is the embedment (in m) of the pile in dense sand strata.

The unit shaft resistance is determined from either [1] with $K = 1$, or [2] with $C = 0.5\%$. For bored piles, reduction factors of 70 and 50%, respectively, are applied to the calculated values of shaft resistance.

The *Tumay and Fakhroo method* (Tumay and Fakhroo 1981) is based on an experimental study in clay soils in Louisiana. The unit toe resistance is determined the same way as in the Schmertmann and Nottingham method. The unit shaft resistance is determined according to [2] with the K coefficient determined according to [8], where the coefficient is no longer dimensionless:

$$[8] \qquad K = 0.5 + 9.5\, e^{-0.09 f_s}$$

where the sleeve friction f_s is measured in kPa.

For a sleeve friction ranging from 10 to 50 kPa, [8] results in a K coefficient ranging from about 4.5 to 0.6. An upper limit of 60 kPa is applied to the unit shaft resistance, r_s.

Comments on the current methods

When using either of the five current direct CPT methods, difficulties arise as follows:

(1) The CPT methods were developed more than a decade ago, therefore their calibration has not made use of the more accurate measurements achievable with modern cone penetrometers.

(2) Although the recommendations are specified to soil type (“clay” or “sand;” very cursorily characterized), the quoted methods do not include a means for identifying the soil type from CPT data. Instead, the soil profile governing the coefficients relies on information from conventional boring and sampling, and laboratory testing, which may not be fully relevant to the CPT data.

(3) All five CPT methods specify that extreme values be eliminated from the data, that is, they require the measurements to be filtered. The filtering may cause the operator to unwittingly bias the results if, in removing the extremes, values that

are representative of the pile–soil load transfer are also removed.

(4) The CPT methods were developed before the advent of the piezocone and, therefore, neglect the pore pressure acting on the cone shoulder (Campanella and Robertson 1988). The subsequent error in the cone stress value is smaller in sand, larger in clay.

(5) The CPT methods employ total stress values, whereas effective stress governs the long-term behavior of piles.

(6) The current CPT methods are locally developed, that is, they are based on limited types of piles and soils and may not be relevant outside the local area.

(7) The upper limit of 15 MPa, which is imposed on the unit toe resistance in the Schmertmann and Nottingham, European, and Tumay and Fakhroo methods, is not reasonable in very dense sands where values of pile unit toe resistance higher than 15 MPa frequently occur. Except for the Meyerhof method, all of the CPT methods also impose an upper limit to the unit shaft resistance, which cannot be justified because values of pile unit shaft resistance higher than the recommended limits occur frequently.

(8) All five CPT methods involve a judgment in selecting the coefficient to apply to the average cone resistance used in determining the unit toe resistance.

(9) In the Schmertmann and Nottingham method and the European method, the overconsolidation ratio OCR is used to relate q_c to r_t. However, although the OCR is often known in clay, it is not easily determined for sand.

(10) In the European method, considerable uncertainty results when converting cone data to undrained shear strength, S_u, and then using this S_u value to estimate the pile toe capacity. The undrained shear strength is not a unique parameter and depends significantly on the type of test used, strain rate, etc. Furthermore, drained soil characteristics also govern long-term pile capacity in cohesive soils. The use of undrained strength characteristics for long-term capacity is therefore not justified.

(11) In the French method, the 1.5b length of the influence zone below the pile toe is too short. (The influence zone is the zone above and below the pile toe over which the cone resistance is averaged.) For example, Meyerhof (1956, 1976) indicated that the length of the influence zone below the pile toe may extend to 10b. Altaee et al. (1992*a*, 1992*b*) reported a case where the depth was found to be 5b. The length of the zone below the pile toe is particularly important if a weaker layer exists near the pile toe, because the ability of the soil to resist the pile toe load is reduced due to the development of horizontal tension in overlying dense soil.

(12) The French method makes no use of sleeve friction, which disregards an important component of the CPT results and soil characterization.

Obviously, the current methods leave something to be desired with regard to the estimation of pile capacity from cone penetrometer data. The advent of the piezocone has provided the means for an improved method, and the authors have developed a new method based on CPTu measurements. The development of the method necessitated a review of the approach toward the filtering of the cone data to account for values that are not representative for determining the cone resistance average. It has also been necessary to discuss the length of the influence zone over which the representative q_c value is to be determined.

Cone resistance average

Natural soil deposits, particularly sands, produce cone resistance profiles with many peaks and troughs. The cone resistance variations reflect the variations of soil characteristics and strengths. Therefore, when determining pile toe resistance, which is a function of the soil conditions in a zone above and below the pile toe, an average must be determined that is representative for the zone. It is important to note that the pile diameter controls the extent of rupture surface below and above the pile toe. Therefore, the value must be a function of the pile diameter.

Filtering the cone data is necessary, because were a mean produced from the unfiltered data, occasional, unrepresentative high and low values would have a disproportionate influence. The filtering approach included in the CPT methods was developed when the CPT data were obtained in hard-copy diagrams only and it includes a considerable bias. However, subjective filtering is now not necessary, because current tests produce results which are easily averaged by direct computer processing.

Two types of averaging can be considered: arithmetic and geometric. The arithmetic average of the cone resistance is

$$[9] \qquad q_{ca} = \frac{q_{c1} + q_{c2} + q_{c3} + \ldots + q_{cn}}{n}$$

where q_{ca} is the arithmetic average of values ranging from q_{c1} to q_{cn}. Without first downgrading the influence of random peaks and troughs, the arithmetic average is only useful where the CPT values are uniform, i.e., in very homogeneous soils. Filtering is therefore necessary in most cases. If done manually, a bias is easily introduced and the results will often differ between different persons. A filtering effect can be achieved directly, however, by calculating the geometric average of the q_c values, which is defined as

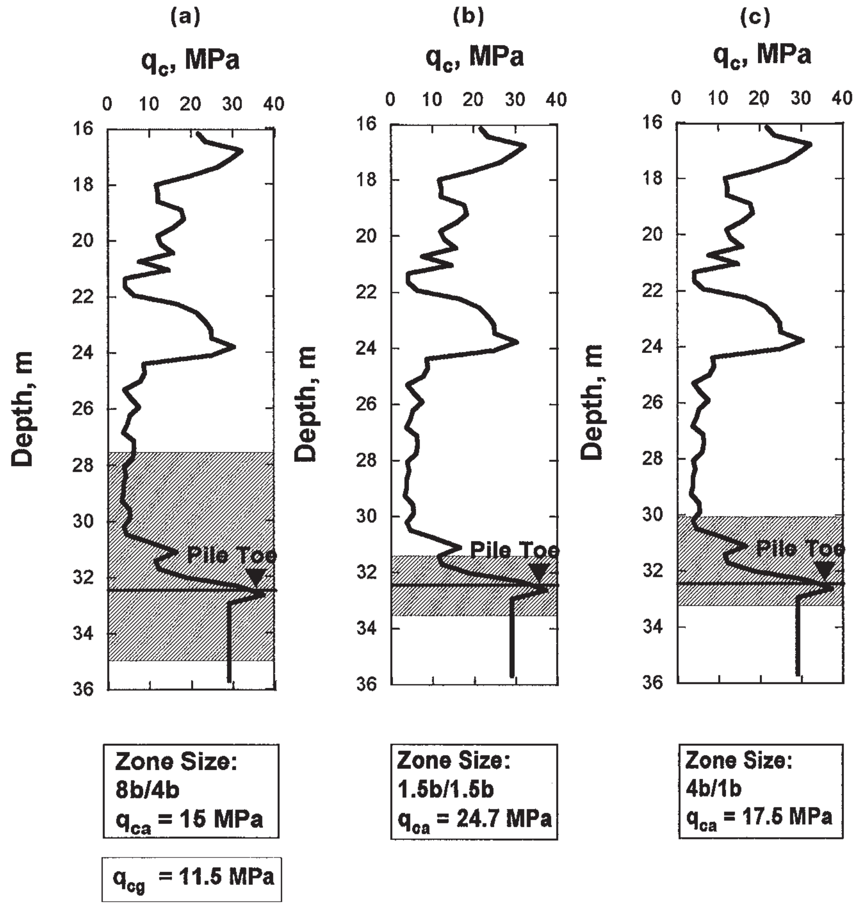
$$[10] \qquad q_{cg} = (q_{c1}\, q_{c2}\, q_{c3} \ldots q_{cn})^{1/n}$$

where q_{cg} is the geometric average of values ranging from q_{c1} to q_{cn}.

The bias in the arithmetic mean as opposed to the geometric average arises from the influence of the absolute magnitude instead of ratios of variations (Kennedy and Neville 1986). Consider the following series of 12 values: 5, 5, 2, 5, 25, 5, 6, 1, 6, 6, 30, and 6. Eight of the values are either 5 or 6, the remaining four are 1, 2, 25, and 30. Clearly, therefore, the dominant values lie between 5 and 6. The arithmetic and geometric averages are 8.5 and 5.7, respectively, and the geometric average is closer to the dominant values, as opposed to the arithmetic average. Thus, by taking the geometric average of q_c values in a zone at the vicinity of the pile toe, a filtered representative value is obtained that is unaffected by bias and, therefore, repeatable.

Figure 3 presents a case history from a CPT profile and pile with the toe located at a depth of 32 m. The soil is sand. Figures 3*a*, 3*b*, and 3*c* show three zones of influence: the Schmertmann–Nottingham and European methods, the French method, and the Meyerhof method, respectively. The arithmetic averages are given below each diagram. The arithmetic q_c average for a zone determined by the Schmertmann and Nottingham and European methods (applying the minimum path rule) is 15.0 MPa. When applying the French method (with manual elimination of peaks and troughs) and the Meyerhof method, the arithmetic means are 24.7 and

Fig. 3. Comparison of q_c average for different direct CPT methods.



17.5 MPa, respectively. The geometric q_c average over the 8*b*/4*b* zone is 11.5 MPa without using a minimum path or special filtering of the data. Of course, Fig. 3 serves more to demonstrate the difference between the direct CPT methods than to prove that the geometric average of 11.5 MPa is more representative for the dominant range of cone resistance than the filtered arithmetic average.

Failure (influence) zone

To estimate the unit toe resistance of a pile (large-scale pile) from the average cone point resistance (a small-scale pile), it is necessary to identify the failure zone around the pile toe. That is, the problem is to define the length of the influence zone for averaging the cone resistance. However, no specific evidence exists to support whether the failure is local punching or general shear failure. Experimental studies by Meyerhof (1956, 1976) indicate, as mentioned, that a pile must penetrate a distance of about 10 pile diameters into a bearing soil layer to fully mobilize the ultimate unit toe resistance of that layer. DeBeer (1963) found that a similar depth was necessary for developing resistance for penetrometers in sand. Experimental and numerical work by Altaee et al. (1992*a*, 1992*b*) on two concrete piles in uniform, medium dense sand suggest that the zone influencing the toe resistance extends from 5*b* above to 5*b* below the pile toe.

Meyerhof (1951) and DeBeer (1963) suggested that in homogeneous soil, a logarithmic spiral can be assumed to define the failure zone, as shown in Fig. 4. The failure pattern is general shear failure type, in which the rupture surfaces extend to the body of the pile or penetrometer at some distance above the pile toe or cone point. The rupture surface is determined by the expression for the radius of a logarithmic spiral as

$$[11] \quad r = r_0\, e^{\theta \tan\phi}$$

where r is the radius of the logarithmic spiral; r_0 is the radius of the logarithmic spiral for $\theta = 0$ (assumed equal to the penetrometer diameter); θ is the angle between a radius and r_0, as shown in Fig. 4*a*; and ϕ is the angle between the radius and the normal at that point on the spiral (assumed equal to the friction angle of the soil).

The height of the failure zone above the pile toe, r_c, is determined by substituting π for θ in [11] and assuming $r_0 = b$:

$$[12] \quad r_c = b\, e^{\pi \tan\phi}$$

where r_c is the height of the failure zone above the pile toe. The value of r_c, in units of penetrometer diameter, can be realized as a criterion to evaluate the depth of penetration of the pile to fully mobilize the ultimate toe resistance of the bearing layer.

The deepest point of the rupture surface below the pile toe is determined by maximizing the projection of the radius of the logarithmic spiral on the vertical axis as follows:

$$[13] \quad z = b\, e^{\theta \tan\phi} \cos\theta$$

Fig. 4. (*a*) Principle of a logarithmic spiral rupture surface around cone point and pile toe. *r*, radius of logarithmic spiral; r_0, radius of logarithmic spiral for $\theta = 0°$; r_c, height of the failure zone above the pile toe. (*b*) Rupture surface for different ϕ angles.

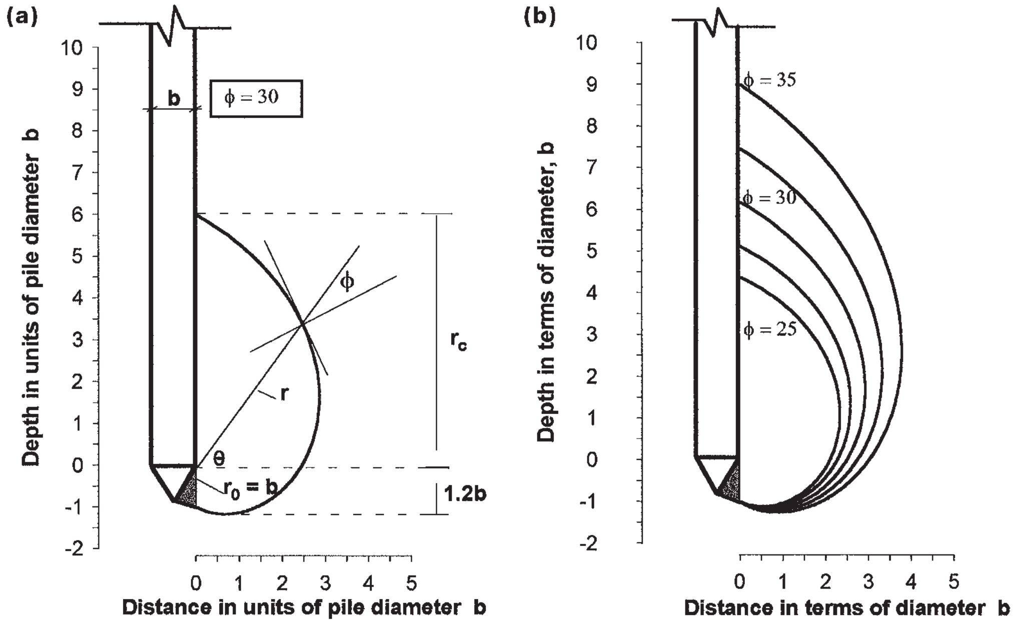


Fig. 5. Comparison of pile unit toe resistance for different influence zones. (*a*) Homogeneous soil. (*b*) Nonhomogeneous soil.

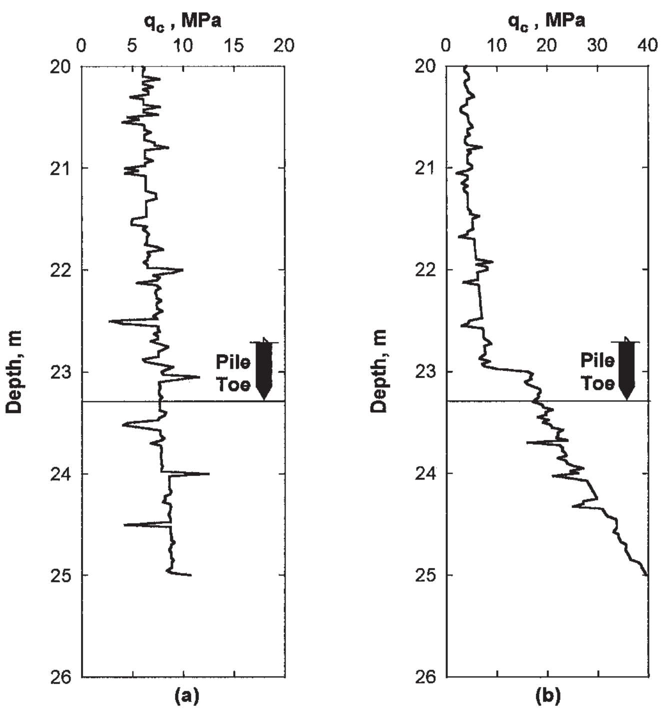


Table 2. Geometric average of cone resistance q_c for different size zones.

Zone		q_c average (MPa)	
Height above	Depth below	Homogeneous	Nonhomogeneous
2*b*	2*b*	7.66	16.13
4*b*	2*b*	7.43	11.79
6*b*	2*b*	7.09	9.06
8*b*	2*b*	6.93	8.06
2*b*	4*b*	7.89	19.92
4*b*	4*b*	7.65	14.89
6*b*	4*b*	7.38	11.97
8*b*	4*b*	7.15	10.11

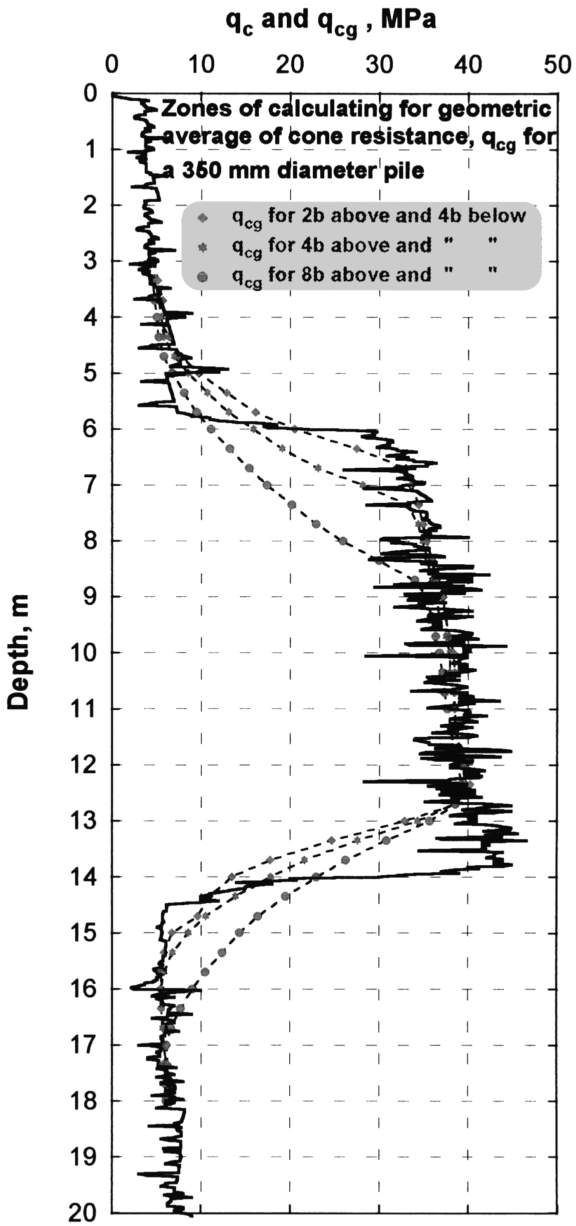
[14] $$\frac{dz}{d\theta} = 0 \rightarrow \theta = \phi$$

Figure 4*b* presents different rupture surfaces, which were developed from [11] by substituting ϕ angles from 25 to 35°. The failure zone includes a height above the pile toe ranging from 4*b* to 9*b*, a depth below the pile toe ranging from 1.1*b* to 1.5*b*, and a horizontal extent ranging from 2*b* to 4*b* (see Fig. 4*b*). For example, for $\phi = 30°$ and $\theta = 180°$, the value of r_c is about 6*b* and the total height of the rupture surface is about 7.5*b*.

The cone resistance needs to be averaged to a representative value within the failure (influence) zone, and the logarithmic spiral is but one way to determine the extent of the zone. To illustrate the importance of the length of the influence zone in a nonhomogeneous soil as opposed to a homogeneous soil, Fig. 5*a* presents CPT data from homogeneous soil and Fig. 5*b* shows the records for a two-layered soil with a distinct increase of cone resistance in the lower layer (nonhomogeneous soil). A 350 mm diameter pile (10 times the standard cone diameter) is assumed to have been installed a small distance into the lower layer.

The geometric average has been calculated for zones of different lengths above and below the pile toe. The distance above is either 2*b*, 4*b*, 6*b*, or 8*b* and the distance below is 2*b* or 4*b*. As presented in Table 2, the geometric q_c values in the homogeneous soil range from 6.9 to 7.9 MPa, a variation of about ±5%, that is, the average is essentially insensitive to the

Fig. 6. Comparison of cone resistance and calculated geometric average (accounting for a 350 mm pile diameter) for a sliding position above, in, and below a dense layer sandwiched between loose layers.



length of the zone. In a homogeneous soil, therefore, the length is not critical for the averaging of the cone resistance. In contrast, the average in the nonhomogeneous soil ranges from 8.1 to 19.9 MPa, a variation of about ±40%. That is, for a pile in nonhomogeneous soil, the extent of the assumed zone of influence is of significant importance.

The height or length to choose for the averaging is particularly important when the pile toe is near the vicinity of the boundary between two layers, as demonstrated in Fig. 6, which presents a CPT profile for a layer with high cone resistance sandwiched between two layers with low cone resistances. A pile of 350 mm diameter is again used to illustrate the effect of three different heights of the failure zone above the pile toe on the q_c average values. The three heights are 2*b*, 4*b*, and 8*b*. All three zones have been assigned a 4*b* depth below the pile toe. The figure illustrates the geometric averages over the three zones for a "sliding" location of the pile toe.

Because the extent of the influence zone for determining the average cone resistance to apply to pile unit toe resistance is not critical in homogeneous soil, diverse theoretical analyses used for determining the extent of the zone may provide different size zones, but the averages in these zones will be quite similar. By contrast, in nonhomogeneous soil, the average will depend on the height, but theoretical analysis does not apply.

Current North American practice is to apply an extent of the zone according to the Schmertmann and Nottingham method, i.e., maximum depth below the pile toe is 4*b*. Although in theory the expansion of a rupture surface in a homogeneous soil below the pile toe does not exceed about 1.5*b*, it is necessary to take into account the cone resistance in a zone somewhat deeper than 1.5*b* for two reasons. First, the greater weight should be given to the q_c values in the soil below the pile toe to correspond to that the failure path is not along a straight vertical line but follows a curved surface which is longer than 1.5*b*. Second, the calculations must consider that the pile toe resistance reduces due to tension that develops when a weak soil exists some short distance below the pile toe, i.e., the punching effect. In view of this, the 4*b* depth below the pile toe appears reasonable and it would be wrong to change the practice.

The current practice also applies the Schmertmann and Nottingham method, i.e., maximum distance above the pile toe is 8*b* when a pile is installed through a weak soil into a dense soil, which appears to be reasonable. Current practice does not include a rule for the case when a pile toe in a dense soil approaches a layer exhibiting a reduced cone resistance. For conditions of a pile to be installed through a dense soil into a weak soil, the authors prefer to use the height above the pile toe of 2*b* rather than 8*b* to avoid giving too much weight to the strength of the dense soil.

New direct CPTu method

A new direct CPTu method (Eslami and Fellenius 1995, 1996; Eslami 1996) has been developed based on the piezocone. In contrast to the five other methods, the data are unfiltered and no minimum path is used. Instead, the potential disproportionate influence of odd "peaks and troughs" is reduced by means of employing the geometric average of the cone point resistance as opposed to the arithmetic average used by the current CPT methods. Furthermore, the cone resistance is transferred to "effective" cone resistance, q_E, by subtracting the measured pore pressure, u_2, from the measured cone resistance, q_t. The *effective geometric average* determined this way times a correlation coefficient is equated to the pile unit toe resistance determined over an influence zone extending from 4*b* below the pile toe to a height of 8*b* above the pile toe when a pile is installed through a weak soil into a dense soil, and 2*b* above the pile toe when a pile is installed through a dense soil into a weak soil. The relation is given as

$$[15] \quad r_t = C_t \, q_{Eg}$$

where C_t is the toe correlation coefficient; and q_{Eg} is the geometric average of the cone point resistance over the influence zone after correction for pore pressure on shoulder and adjustment to

Table 3. Shaft correlation coefficient C_s.

Soil type	C_s Range (%)	Approximation (%)
Soft sensitive soils	7.37–8.64	8.0
Clay	4.62–5.56	5.0
Stiff clay and mixture of clay and silt	2.06–2.80	2.5
Mixture of silt and sand	0.87–1.34	1.0
Sand	0.34–0.60	0.4

effective stress. According to studies by DeBeer (1963), Kerisel (1964), Vesic (1964), and Salgado (1993), a one-to-one relationship exists between the cone resistance and the pile unit toe resistance in sand. That is, current literature indicates that the toe coefficient, C_t, is equal to unity.

Also the pile unit shaft resistance is correlated to the average effective cone point resistance with a modification according to soil type according to the approach detailed below. The shaft correlation coefficient (C_s) is determined from the soil profiling chart (Fig. 2), which uses both cone stress and sleeve friction. However, because the sleeve friction is a more variable measurement than the cone point resistance, the sleeve friction value is not applied directly.

[16] $r_s = C_s\, q_E$

where C_s is a function of soil type determined from the soil profiling chart; and q_E is the cone point resistance after correction for pore pressure on the cone shoulder and adjustment to effective stress.

Calibration of the proposed method

Calibration of the proposed CPTu method requires determination of the toe and shaft coefficients C_t and C_s, respectively. The 24 group I cases in the database where the toe and shaft resistances are known (14 and 24 cases, respectively) have provided the means for a calibration of the new method.

Calibration of the toe correlation coefficient, C_t

The measured unit toe resistance, r_t, and geometric average of effective cone resistance, q_{Eg}, at the vicinity of the pile toe were compiled with the 14 pile case histories with known pile toe resistance. For each case, the unit pile toe resistance was divided by the geometric average to determine the ratio r_t/q_{Eg}, which also is the toe correlation coefficient, C_t. The data resulted in an average ratio of 0.98 with a standard deviation of 0.09. That is, the toe correlation coefficient, C_t, can be taken as equal to unity, which is in agreement with the previously quoted technical literature.

The effective cone resistance in sensitive or soft clays could obviously be very small, resulting in an uncertain value of pile toe resistance. However, for a pile in cohesive soils, the major source of the pile capacity is shaft resistance not toe resistance. Therefore, an error in the estimation of the toe resistance in clay soils is not significant in practice.

Calibration of the shaft correlation coefficient, C_s

The values of pile unit shaft resistance determined in 10 tension static tests and measured separately in 14 compression static tests were used to correlate the CPTu data with the pile shaft resistance. No difference is considered for shaft resistance in tension or compression. The total shaft resistance was divided by the pile surface area to produce an average unit shaft resistance, r_s, and these values were compared with the average of the effective cone resistance. Table 3 presents the results of the ratios obtained for the shaft resistance and average effective cone resistance, r_s/q_{Eg}, as separated on the dominant soil type according to the five main soil categories of the soil profiling diagram. Notice that, although the short-term effective resistance recorded by the cone penetrometer must differ from the long-term pile resistance, the calibrated ratios include that difference, i.e., not quite an apple to apple comparison. Indeed, the calibration is approximate inasmuch as one relatively small set of data is matched against another small set of data. It is recognized that a compilation using a different series of test data could have resulted in slightly different ratios.

Fig. 7. Measured versus estimated pile capacity for group I (24 cases) by the CPTu method.

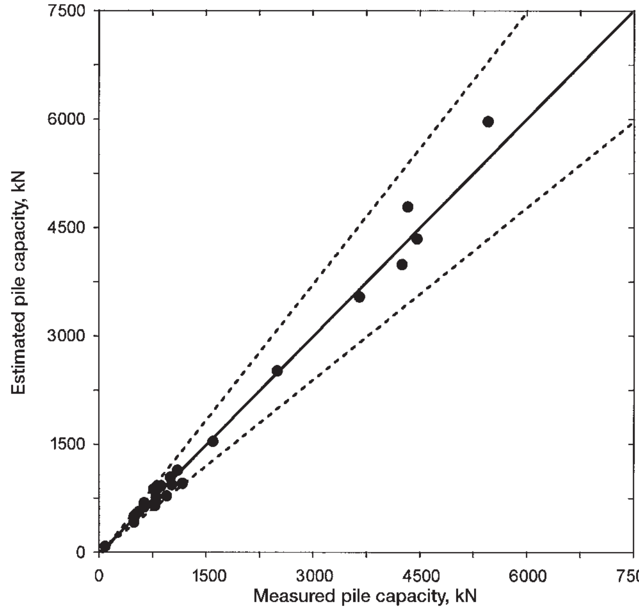


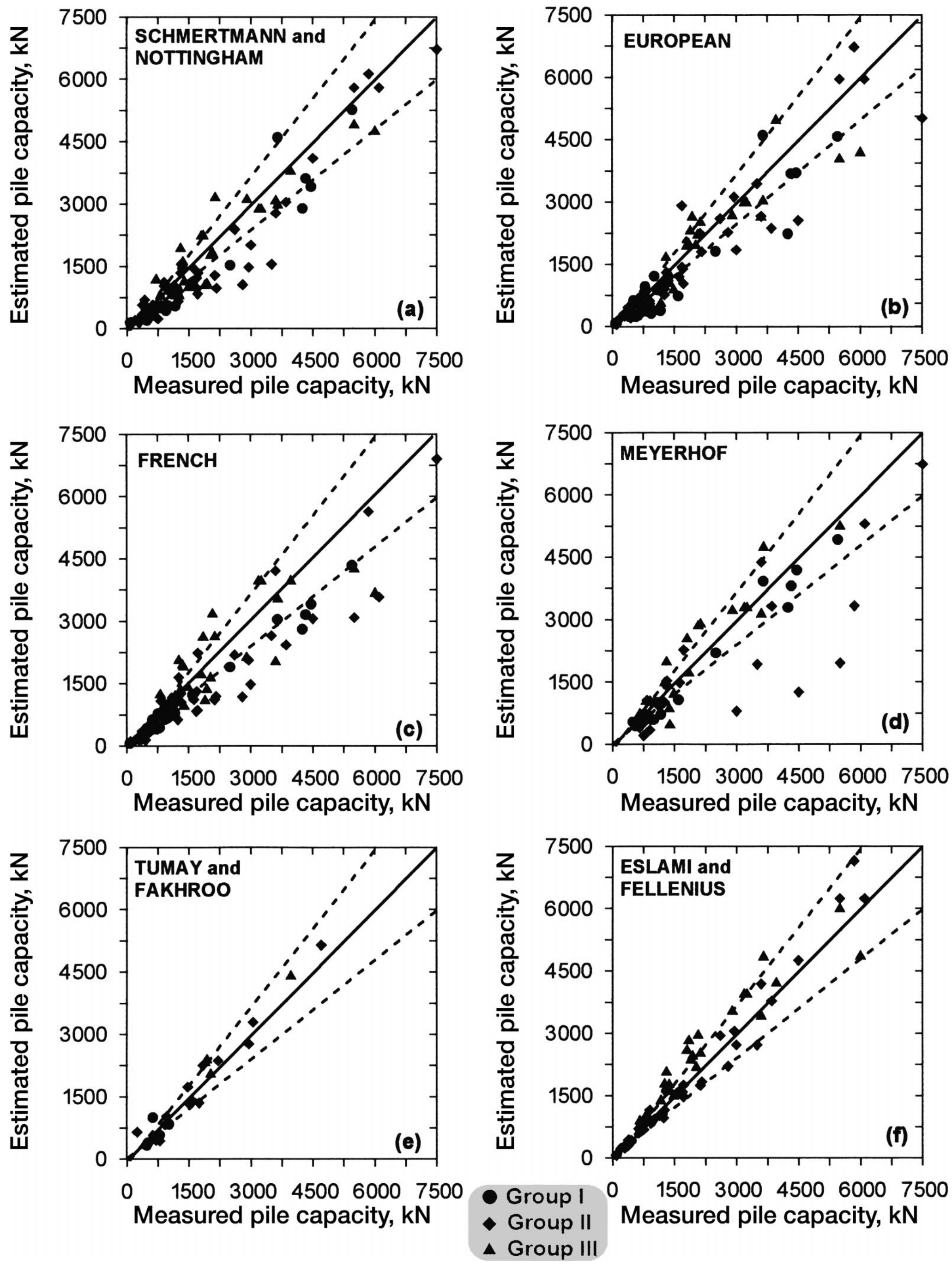
Table 3 also shows a judgment-selected approximated ratio of the correlation coefficient, C_s, whose values are the synthesis result of the calibration. Further research may show that the values should be adjusted.

Figure 7 illustrates the agreement between capacities (group I cases) determined by means of a toe correlation coefficient, C_t, equal to unity and a shaft correlation coefficient, C_s, as given in Table 3. The approximation of the coefficients results in an average absolute difference of only 7% and a standard deviation of 6%.

Validation of the CPTu method

The cone data were processed to determine the total capacity of the piles, and the results were compared with the capacities determined in the static loading tests. Where sleeve friction measurements were absent (some cases in group II), the soil profiling was accepted as given in the case records. For reference, the cone data were also processed by the five CPT methods

Fig. 8. Measured versus estimated pile capacity by different methods.



(the Tumay and Fakhroo method is pertinent to 25 cases and the Meyerhof method to 57 cases).

To compare the calculated and measured pile capacity for all case records, Figs. 8*a*–8*e* present the results of one of the current CPT methods, and Fig. 8*f* presents the proposed CPTu method. The diagrams use different symbols for each of the groups I–III case histories. The diagonal line in each diagram indicates perfect agreement between calculated and measured pile capacity. The broken line represents a deviation of ±20% from perfect agreement. The scatter is considerably smaller for the Eslami and Fellenius CPTu method. Note that for the latter, group I cases have been excluded, as the values are biased because they were used in the calibration of the method (the group I results are shown in Fig. 7).

The group III maximum test load is considered a lower-boundary capacity. This explains why many of the calculated values of pile capacity plot above the solid line. This is particularly evident for the new method, which displays 23 cases of the 25 cases of group III above the line.

Table 4 presents a numerical comparison of the methods in terms of the percent difference between the tested and calculated pile capacities, determined as the difference between the capacity calculated by the cone methods and the capacity found in the static loading test divided by the static loading test value. The percent difference is positive or negative, reflecting when the calculated value is larger or smaller, respectively, than the capacity found in the static loading test. Table 4 presents the arithmetic mean and standard deviation of the average percent difference for each of groups I–III. The average of the percent difference is less meaningful than the average of the absolute percent difference. Therefore, Table 4 also presents the mean and standard deviation of the average absolute percent difference for each of the groups.

For the 24 cases compiled in group I, all five CPT methods underestimate the pile capacity. The Schmertmann and Nottingham method, the European method, and the Tumay and Fakhroo method (applicable to five of the cases, only) show an average absolute percent difference of about 35%, whereas the French and the Meyerhof methods gives an average absolute percent difference of 20%.

For the 53 cases of group II (capacity well established from the static loading test), the average absolute percent differences found in the CPT methods range from 24 to 36%. In contrast, the Eslami and Fellenius CPTu method gives an average absolute percent difference of only 12%, which indicates a very good agreement between the calculated and measured capacities. More important, the agreement is consistent for all 53 cases, as evidenced by a standard deviation of 8%, as opposed to standard deviations ranging from 16 to 32% for the CPT methods.

Figure 9 illustrates the average absolute percent differences for group I and group II cases for the CPT methods. The group I results of the CPTu method have been excluded so that the comparison is not biased. Group III calculations are not included because of the uncertainty with the pile capacity values of the static loading tests.

Long and Shimel (1989) and Alsamman (1995) suggest that statistics will provide more insight when the values are plotted versus their cumulative average, called “cumulative probability.” For the current set of data, the ratio of calculated to tested pile capacities, Q_p/Q_m, is arranged in ascending order (numbered 1, 2, 3,...i,...n) and a cumulative probability, P, is determined for each capacity value as

$$[17] \quad P = \frac{i}{(n+1)}$$

where i is the number of the value considered in P. According to Long and Shimel (1989) and Alsamman (1995), to assess the bias and dispersion associated with a particular predictive method the following is useful:

(1) The value of Q_p/Q_m at P = 50% probability is a measure of the tendency to overestimate or underestimate the pile capacity. The closer the ratio is to unity, the better the agreement.
(2) Log-normally distributed data will plot on a straight line.
(3) The slope of the line through the data points is a measure of the dispersion or standard deviation. The flatter the line, the better general agreement.

Figure 10 shows the plot of Q_p/Q_m values versus cumulative probability for group I and group II cases. For the probability of 50%, the Q_p/Q_m value for the CPTu method is close to unity, whereas the ratios for the current methods are in the range of 70–80%, demonstrating a trend toward underestimation. The CPT methods exhibit a higher dispersion than the CPTu method, as indicated by the flatter slope of the line through the CPTu data. It is obvious that the results for the CPTu method are closer to log-normal distribution than those for the CPT methods.

Table 4. Percent differences in pile capacity estimation.

Method	Average		Group I (24 cases, capacity)	Group II (53 cases, capacity)	Group III (25 cases, maximum load)
Schmertmann and Noffingham	Strict				
		Mean	–29	5	–2
		SD	27	–13	29
	Absolute				
		Mean	36	3l	23
		SD	15	22	17
European	Strict				
		Mean	–26	–15	–6
		SD	32	25	23
	Absolute				
		Mean	34	24	20
		SD	23	17	13
French	Strict				
		Mean	–23	–24	5
		SD	13	26	33
	Absolute				
		Mean	23	32	26
		SD	13	16	19
Meyerhof	Strict				
		Mean	–17	–23	7
		SD	15	34	32
	Absolute				
		Mean	19	34	23
		SD	12	23	18
Tumay and Fakhroo	Strict				
		Mean	–12	–3	16
		SD	35	42	24
	Absolute				
		Mean	32	26	16
		SD	14	32	9
Eslami and Fellenius	Strict				
		Mean	–1	0	24
		SD	9	15	31
	Absolute				
		Mean	7	12	26
		SD	6	8	15

Conventional effective stress analysis

Many of the cases in the database include soil data and pore-water pressure information that make possible a back-calculation of the effective stress parameters, β and N_t, that correspond to the pile shaft and toe capacities. The relations for unit toe and shaft resistances determined according to effective stress analysis of pile load transfer (e.g., Fellenius 1996) are given as follows:

Fig. 9. Comparison of average absolute percent difference in pile capacity estimation for all methods.

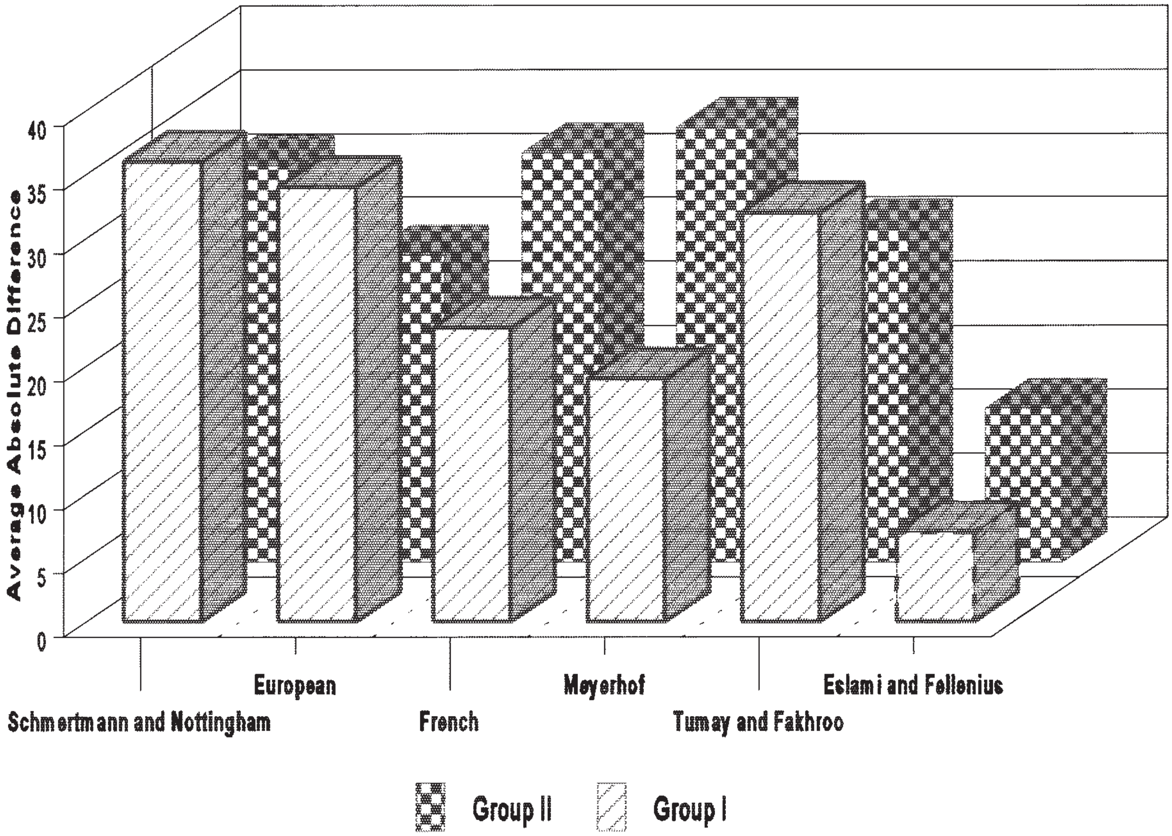


Fig. 10. Comparison of the methods estimation using the probability approach.

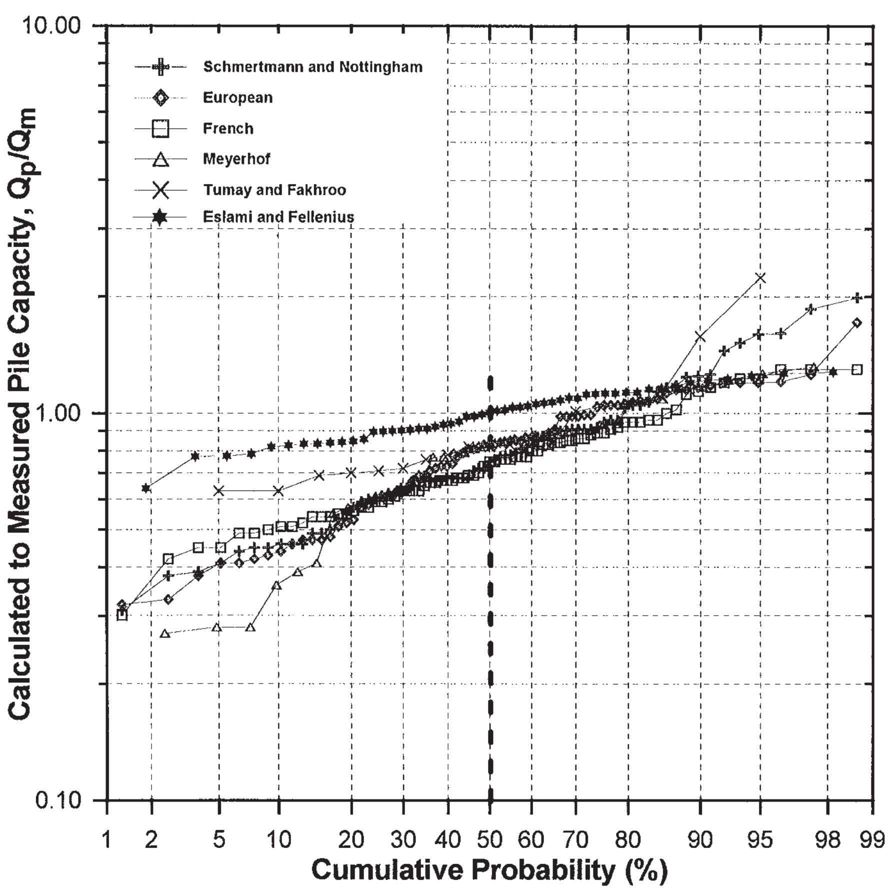


Table 5. Percent differences in pile capacity estimation.

No.	Name	Soil profile	β (—)	N_t (—)	β/N_t (%)	N_t/β (—)
1	UBC3	Soft clay	0.25			
		Sand	0.43	34	1.3	80
2	UBC5	Soft clay	0.25			
		Sand	0.30			
		Silt	0.25	10	2.5	40
3	NWUP	Sand	0.70			
		Clay	0.25	2	10.0	10
4	FHWASF	Sand	0.35	75	0.5	210
5	BGHD1	Silty sand	0.40			
		Sand	0.65	31	2.1	50
6	BGHD2	Silty sand	0.50			
		Sand	0.70	35	2.0	50
7	POLA1	Silt	0.30			
		Sand	0.30	55	0.6	180
8	POLA2	Sand	0.30	40	—	—
9	TWNTP4	Silt	0.35			
		Clay	0.25			
		Sand	0.40	20	2.0	50
10	TWNTP5	Silt	0.30			
		Clay	0.20			
		Sand	0.35	*		
11	TWNTP6	Silt	0.30			
		Clay	0.20			
		Sand	0.40	22	1.8	55
12	L&D314	Sand	0.54	*		
13	L&D35	Sand	0.44	*		
14	L&D316	Sand	0.45	*		
15	L&D32	Sand	0.47	*		
16	L&D38	Sand	0.34	*		
17	L&D315	Sand	0.42	*		
18	A&N2	Loose sand	0.70			
		Dense sand	0.90			
		Limestone	1.00	60	1.7	60
19	N&SBI44	Sand	0.40			
20	QBSA	Soft clay	0.40	4	10.0	10
21	UHUC1	Stiff clay	0.50			
		Sand	0.60	27	2.2	45
22	UHUT1	Stiff clay	0.50			
		Sand	0.60	*		
23	UHUC11	Stiff clay	0.50			
		Sand	0.65	27	2.4	40
24	UHUT11	Stiff clay	0.50			
		Sand	0.55	*		
27	NWUH	Sand	0.70			
		Clay	0.25	2.3	10.0	10
49	USPB1	Sand	0.28	25	1.1	90
50	USPB2	Sand	0.31	20	1.6	65
59	SPB	Sand	0.85	60	1.4	70
62	PNTRA5	Clayey silt	0.15	*		
63	PNTRA6	Clay	0.27	*		
64	CWDNC	Stiff clay	0.40			
		Silty sand	0.40	*		
66	CWDNE	Stiff clay	0.38			
		Silty sand	0.38	*		
67	CWDNF	Stiff clay	0.39			
		Silty sand	0.39	*		
68	CWDNG	Stiff clay	0.46			
		Silty sand	0.46	*		
69	CWDNH	Stiff clay	0.51			
		Silty sand	0.51	*		
70	CWDNI	Stiff clay	0.43			
		Silty sand	0.43	*		
71	CWDNJ	Stiff clay	0.42			
		Silty sand	0.42	*		
72	CWDNK	Stiff clay	0.51			
		Silty sand	0.51	*		
73	ONSYA1	Soft clay	0.25	*		
74	ONSYB1	Soft clay	0.29	*		
75	LSTDA7	Soft clay	0.15	*		
76	LSTDA8	Soft clay	0.09	*		
77	LSTDB2	Soft clay	0.20	*		

Note : *, tension test.

$$[18] \quad r_s = \beta\, \sigma'_z$$

$$[19] \quad r_t = N_t \sigma'_{z=D}$$

where σ'_z is the effective overburden stress at depth z; β is the Bjerrum–Burland beta coefficient; N_t is the toe bearing capacity factor; $\sigma'_{z=D}$ is the effective vertical stress at pile toe level; and D is the pile embedment length.

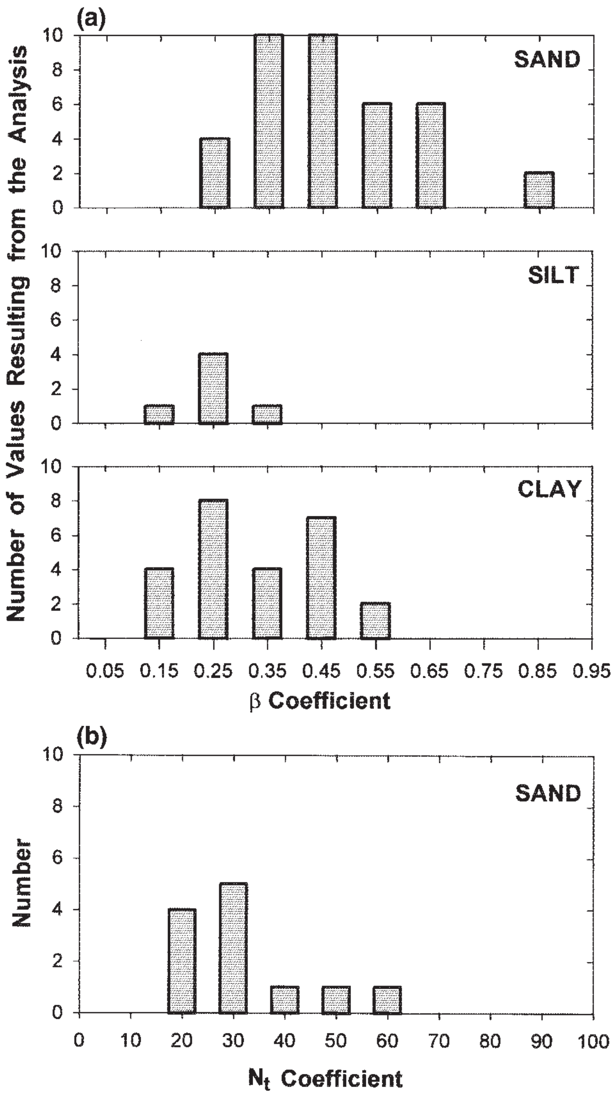
The cases are the group I cases and 19 of the group II cases (cases with no sleeve friction data were excluded). Table 5 summarizes the results of the calculations. Notice, that the β/N_t values shown in the second from the right column are not intended to provide a comparison to the friction ratio, but to the ratio between shaft and toe resistances. A histogram presentation of the β coefficient is shown in Fig. 11*a* separated on the main soil types, clay, silt, and sand. Figure 11*b* presents the results for the toe bearing coefficient, N_t, in sand.

Figure 11 shows, which is not surprising, that the average β coefficient is lowest in clay and largest in sand, with the values in silt in between. The ranges are wide and overlapping. They can of course be narrowed in a specific case with more soil information (from sampling and laboratory testing) coupled with local experience. The cone penetrometer, the piezocone in particular, would be quite useful in this process. However, as demonstrated in this study, using the piezocone cone directly and the CPTu method to determine the pile capacity is more reliable than using generally referenced β and N_t coefficients. This said, the effective stress analysis using β and N_t coefficients will never be disastrously off. Furthermore, it is not in conflict with the use of the cone penetrometer methods nor with any other method. In fact, whatever the method used to estimate the capacity of a pile, it is good practice to always check the results by comparing them with the results of an effective stress analysis.

Making the approximation that the q_{Eg} value used for the unit toe resistance, r_t, and the q_E values used for the unit shaft resistance, r_s, in the vicinity of the pile toe are equal, the shaft correlation coefficient, C_s, is equal to the ratio between the shaft and toe resistances (eq. [16] divided by eq. [15]; C_t is equal to unity). Thus

$$[20] \quad C_s = \frac{r_s}{r_t} = \frac{\beta}{N_t}$$

Fig. 11. Back-calculated effective stress coefficients. (*a*) β coefficient (*b*) N_t coefficient.



Fellenius (1989, 1995, 1996) proposed a ratio of β to N_t coefficients ranging from 1.2 to 8% in clays, 1.2 to 1.4% in silts, and 0.4 to 1% in sands, values which show good agreement with the ranges of C_s shown in Table 3, though this is less so with the ratios shown in Table 5.

Discussion

The CPTu method is particularly suitable in noncohesive soils. However, in dense soils, because of the limiting capacity of the cone penetrometer, piles are often taken deeper than the cone penetrometer. Site investigations to be used in pile design often require cone penetrometers with a 500 kN capacity. In cohesive soils, the pore-pressure measurements become crucial and, although the u_2 measurement has shown to provide reasonable correlations in this study, this may not be the governing measurement of pore-water pressure. The effect of the uncertainty is to a large extent included in the calibration and validation of the CPTu method. However, the number of cases used are limited and, doubtlessly, further correlation experience will result in adjustment of the coefficients, in particular, the shaft coefficient.

Furthermore, the cone test is a short-term test, and pile capacity is a long-term condition. Short-term versus long-term behavior involves soil set-up (capacity gain with time) phenomena, amongst others. For driven piles, further research should correlate measurements from dynamic testing of piles with CPTu data. It is conceivable that such studies will produce insights also in dynamic parameters to use when using CPTu measurements to estimate drivability of piles.

Conclusions

Five direct CPT methods and one direct CPTu method for determining pile capacity are presented and discussed. For the CPT methods, the main factors causing significant error in pile capacity estimation are that the methods (1) apply subjective smoothing of the CPT data and employ undrained shear strength, S_u; (2) impose upper limits on pile unit toe and shaft resistances; (3) apply broad correlation factors to separate tension and compression as well as pile material; and (4) disregard the development of excess pore pressures, dilatancy effects, and effective stresses. These disadvantages are avoided in the CPTu method.

The methods are applied to 102 case histories reporting tests on piles of different sizes, types, and lengths installed in different types of soil, where cone test data and results of static loading tests are available. The results of a comparison of the calculated pile capacities to the measured pile capacities are very favorable to the CPTu method, which shows better agreement with the capacity determined in a static loading test and less scatter than the CPT methods. The CPTu method is simple, easy to apply, and independent of all operator subjective influence (on the data, but not on the interpretation, which is where engineering judgment should be applied). Therefore, it is notably suitable for use in engineering practice.

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List of symbols

b pile diameter
C a dimensionless coefficient; a function of the pile type
C_1 modification factor for scale effect
C_2 $D_b/10b$; modification for penetration into dense strata
C_t toe correlation coefficient
C_s shaft correlation coefficient
D pile embedment length
D_b embedment of pile in dense sand strata
f_s cone sleeve friction
i the number of the value considered in cumulative probability P
K a dimensionless coefficient
K_s earth pressure coefficient
n total numbers of values
N_c conventional bearing capacity factor
N_K a dimensionless coefficient
N_q bearing capacity coefficient
N_t toe bearing capacity factor
P cumulative probability
q_c cone resistance (total; uncorrected for pore pressure on cone shoulder)
q_{ca} arithmetic average of q_c
q_{cg} geometric average of q_c
q_E q_c corrected for pore pressure on the cone shoulder and adjustment to "effective" stress
q_{Eg} geometric average of q_E after adjustment to effective stress
q_t q_c corrected for pore pressure on shoulder
Q_p calculated pipe capacity
Q_m tested pipe capacity
r radius of logarithmic spiral
r_c height of the failure zone above the pile toe
r_0 radius of logarithmic spiral for $\theta = 0$
r_s unit shaft resistance
r_t unit toe resistance
R_f friction ratio
R_{ult} total capacity
S_u undrained shear strength
u pore pressure
u_2 pore pressure measured behind the cone point
α adhesion factor
β Bjerrum–Burland beta coefficient
ϕ angle between the radius and the normal at that point on the spiral
σ_z' effective overburden stress at depth z
$\sigma_{z=D}'$ effective vertical stress at pile toe level
θ angle between a radius and r_0