CPT-Based Design Method for Axial Capacities of Drilled Shafts and Auger Cast-in-Place Piles

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Abstract: The paper presents observations from a newly compiled database of static load tests comprising 68 instrumented drilled shafts and auger cast-in-place piles in sands, sand mixtures, silt mixtures, and clays at 37 sites around the world. The measured unit shaft friction and base resistance of the database piles are compared with values calculated using well known methods that correlate capacity directly to the cone penetration test (CPT) end resistance. It is shown that the updated Laboratoire Central des Ponts et Chaussées (LCPC) method in 2012 is the best performing of existing CPT-based methods. A new CPT approach is proposed that, similar to other recently published approaches and experimental studies, involves the soil behavior type index (I_c) determined in CPTs in the formulation. This approach can be expected to lead to more reliable estimates of pile capacity as it provides an improved fit to the new database while also being consistent with the trends implicit in the LCPC method, which is based on a larger and independent data set. **DOI: 10.1061/(ASCE)GT.1943-5606.0002542.**

Author keywords: Pile and piling; Drilled shaft; Bored pile; Auger cast-in-place pile (ACIP); Continuous flight auger (CFA) pile; Clay; Silt; Sand.

Introduction

The assessment of pile capacity using correlations with the cone penetration test (CPT) end resistance continues to increase in popularity due to the ongoing growth of the CPT worldwide and to its ability to eliminate user subjectivity and facilitate automation of pile capacity calculations directly from measured data. The shift to design using CPT data has already occurred for driven piles in sand, as evidenced by the phasing out in the ISO recommendations of the traditional earth pressure approach [i.e., ISO 19901-4 (ISO 2021)] in favor of CPT-based methods that have been shown to have much higher reliability (Nadim et al. 2020; Phoon and Retief 2016; Schneider et al. 2008). The most popular CPT-based approaches for the design of drilled shafts (bored piles) and auger cast-in-place piles (ACIPs) [also known as continuous flight auger (CFA) piles] involve direct application of empirical factors (α_s or β_c and $\alpha_{b0.1}$) to the CPT (corrected) end resistance (q_t) for estimation of unit shaft friction (q_s) and end bearing stress at a displacement of 10% of the pile diameter ($q_{b0.1}$), that is

$$q_s = q_t/\beta_c = \alpha_s q_t \tag{1}$$

$$q_{b0.1} = \alpha_{b0.1} q_t \tag{2}$$

LCPC Methods

One of the first comprehensive direct CPT-based methods, which is known as the Laboratoire Central des Ponts et Chaussées (LCPC)-1982 method, was proposed by Bustamante and Gianeselli (1982) and has been used extensively in Europe for many years. Briaud and Tucker (1988), Robertson et al. (1988), and O'Neill et al. (1999) indicated that this method provided better predictions than other empirical correlations.

A larger database comprising 174 full scale static load tests (most of which are unpublished) was used to update LCPC-1982 and is referred to as LCPC-2012 (Frank 2017). The unit shaft friction and end bearing resistance for this method are determined using the following expressions:

$$q_s = \alpha_{\text{pile-soil}}\, q_{\text{soil}} \tag{3}$$

$$\text{where } q_{\text{soil}} = (a q_c + \mathrm{b})(1 - e^{-c q_c}) \tag{4}$$

$$q_{b0.1} = \alpha_{b0.1} q_c \tag{5}$$

where $\alpha_{\text{pile-soil}}$ is an empirical coefficient, which is related to the pile construction method and soil type through the a, b, and c parameters. Eq. (4) requires units of MPa to be used for q_c (and this leads to calculation of q_{soil} and hence q_s) in MPa. The total cone end resistance (q_t), which requires correction to the q_c value for excess pore pressures at the cone shoulder (u_2), is not employed.

Eslami and Fellenius (1997)

Eslami and Fellenius (1997) compiled a database of 102 full-scale pile load tests although it is noted that the majority of tests were conducted on driven piles. They proposed the following CPT-based correlations, where the unit shaft friction and end bearing resistance are related to what is referred to as the effective cone resistance (q_E), which is defined as the difference between q_t and u_2:

$$q_s = q_E/\beta_{cE} = (q_t - u_0)/\beta_{cE} \tag{6}$$

$$q_b = q_E \tag{7}$$

Recommended β_{cE} values for both displacement and nondisplacement piles are 250 in sand, 100 in sand-silt mixtures, 40 in silt, and 20 in clay; see Fig. 1.

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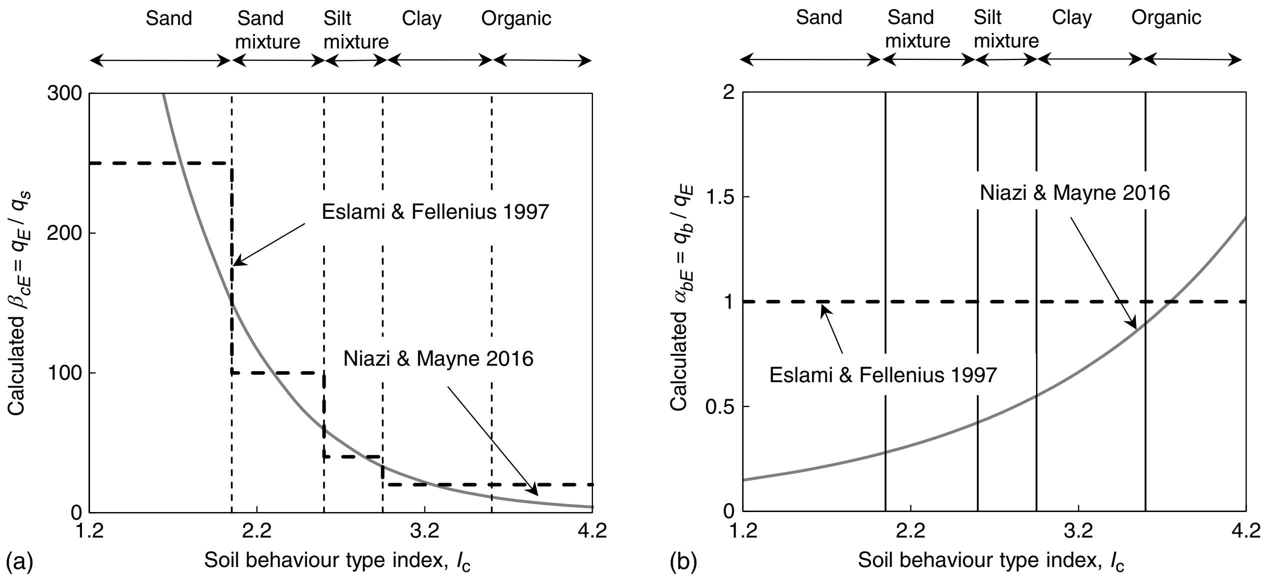

Fig. 1. Proposed correlations for shaft and base capacities with the effective cone resistance.

Niazi and Mayne (2016)

The correlations of Niazi and Mayne (2016) are also written in terms of the effective cone resistance (q_E) but the constant β_{cE} varies with the soil behavior type index, I_c, where

$$I_c = [(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2]^{0.5} \tag{8}$$

where Q_{tn} = stress normalized q_t value; and F_r = normalized friction ratio (Robertson 2009).

The following relationships are proposed based on 47 pile tests on drilled shafts (bored piles). These are written as functions of I_c and hence capture the full grading spectrum. The resulting variation of β_{cE} is shown in Fig. 1 where it is also compared with the stepped nature of the Eslami and Fellenius formulation

$$q_s = q_E/\beta_{cE} = (q_t - u_2)/\phi_{tc} 10^{(0.732Ic-3.605)} \tag{9}$$

$$q_b = \alpha_{bE} q_E = 10^{(0.325Ic-1.218)}(q_t - u_2) \tag{10}$$

where ϕ_{tc} varies with loading direction and is 0.85 and 1.11 for piles undergoing tension and compression loading, respectively.

The wide range of methods employed for drilled shafts and doubts related to their reliability prompted the investigation reported in this paper, which first involved compilation of a new and independent database of static load tests performed on drilled shafts and ACIPs in sands, sand mixtures, silt mixtures, and clays. The existing direct CPT-based methods, described previously, are used to examine and quantify the level of uncertainty associated with each method. A new approach is then proposed and shown to provide an improved fit to the database and to be consistent with the trends implicit in LCPC-2012, which is based on a larger and independent data set.

Database of Static Pile Load Tests

The database compiled comprises results from static load tests on a total of 68 instrumented nondisplacement piles at 37 sites with soil conditions involving clays, silt mixtures, sand mixtures, and sands; CPT data were available close to the location of each test pile. This database only considered maintained load tests, and the quoted displacements were obtained when the creep rate was less than 0.004 mm/min. The instrumented data from the pile tests are distilled into 81 reliable measurements of shaft friction in well-defined soil layers at these 37 sites. A total of 34 of the pile load tests were performed in sand, 22 were in silt-sand mixtures, and 12 were in clay. The database piles have lengths and diameters, as plotted in Fig. 2, and a mean diameter (D), length (L), and slenderness ratio (L/D) of about 0.6, 12, and 20 m, respectively. All details concerning the soil properties and load test data in addition to the CPT profiles at each site are provided by Doan (2019).

CPT results including corrected end resistance (q_t), sleeve friction (f_s), and pore pressure (u_2) closest to each test pile were digitized and recorded. In cases where reliable pore pressure data were not available, the value of q_t was assumed equal to $1.15q_c$ in lightly overconsolidated clays when the normalized cone resistance (Q_{tn}) was less than 6 and equal to q_c at higher Q_{tn} values; this approximation is justified by Lehane et al. (2017). Mean normalized cone

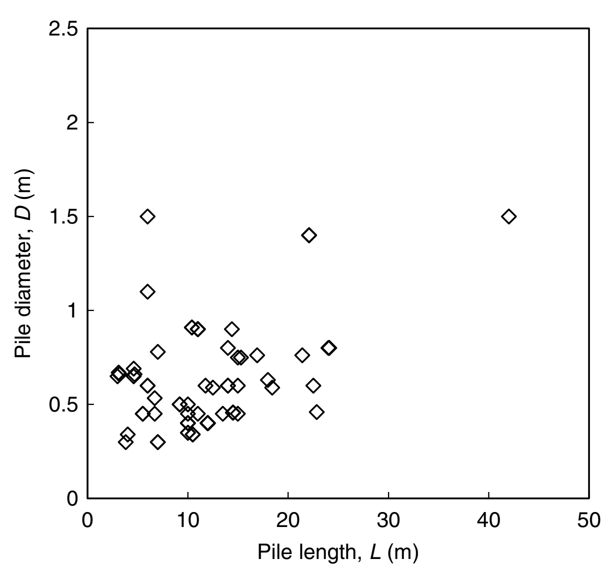

Fig. 2. Pile dimensions in the database compiled for this study.

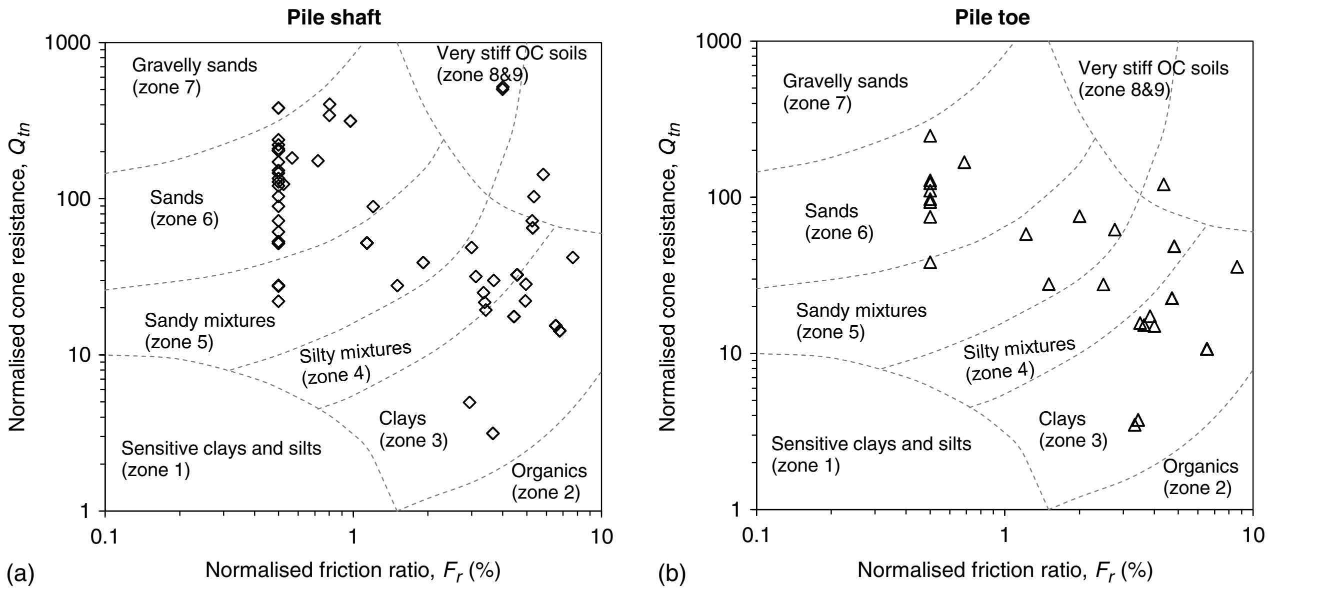

Fig. 3. Soil behavior types along the pile shafts and at the bases of the database piles.

resistances (Q_{tn}) and friction ratios (F_r) corresponding to the measurements of ultimate pile shaft friction (q_s) and ultimate end bearing ($q_{b0.1}$) are presented in Figs. 3(a and b). These results were used to evaluate the soil behavior type index (I_c) proposed by Robertson (2009). Silt-sand mixtures are defined as deposits with I_c values between 2.05 and 2.60 while deposits with I_c values between 2.60 and 2.95, above 2.95, and below 2.05 are referred to as silts, clays, and sands, respectively. The database only considered shaft friction and back capacity measurements in layers with a near-constant I_c value. The q_t value at the pile base was taken as the average value in a zone extending $1.5D$ (pile diameter) above and below the pile tip [e.g., as employed by Bustamante and Gianeselli (1982) and others]. The normalized cone resistance in partly saturated soils (discussed later) was evaluated assuming the vertical effective stress (σ'_v) was equal to the vertical total stress (σ_v).

Table 1 provides a summary of the database piles and includes information related to the reference for the case history, the loading direction (tension or compression), the soil description, the pile boring method (e.g., ACIP, dry boring, boring under bentonite/polymer/casing), the length of the soil shaft over which the shaft friction is recorded (L_s), the average value of σ'_{v0} over this segment length (σ'_{v0}), and the I_c index corresponding to the shaft friction measurement as well as that relating to the pile base. The listed values of σ'_{v0} are total vertical stresses in partly saturated soils.

Table 2 presents the ultimate shaft friction (q_{sm}) measured over a segment of the pile shaft of length L_s. The soil type did not vary over the selected segment lengths. The measured ultimate end bearing (q_{bm}), defined as the base stress at which the pile head load displacement curve reached a clear plateau (usually in clays) or the stress at a pile head displacement (δ_h) of $0.1D$. These measurements are also compared with calculated values of ultimate shaft friction (q_{sc}) and end bearing (q_{bc}) obtained using the formulations proposed by the LCPC-1982 and LCPC-2012 methods, Eslami and Fellenius (1997), and Niazi and Mayne (2016); see Eqs. (3)–(10).

Performance of Existing Methods

The ability of the four CPT-based methods, described previously, to predict ultimate shaft friction (q_s) and base capacity ($q_{b0.1}$) of the database piles is examined in Table 2 and in Figs. 4–6. A number of general observations may be made:

1. The q_s values in materials with CPT end resistances in excess of about 2 MPa are overestimated significantly by LCPC-1982; see Fig. 4.
2. Measured β_{cE} values fall below those recommended by Eslami and Fellenius (1997) and Niazi and Mayne (2016) in the lower I_c range ($I_c < 2.4$), indicating these methods underpredict shaft friction in sands and silty sands; see Fig. 5.
3. Base capacities tend to be overpredicted by all methods. The Eslami and Fellenius (1997) approach overpredicts significantly in all soil types while that of LCPC-1982 overestimates for $q_{c,\text{base}}$ in excess of about 5 MPa. The Niazi and Mayne (2016) base capacity predictions are reasonable in sands but overestimate $q_{b0.1}$ values in silt and clay soils ($I_c > 2.2$); see Fig. 5.
4. As illustrated in Fig. 6, the ratio of calculated to measured capacities for the methods are close to unity and therefore the significant under and overpredictions of the methods when considering specific soil types are masked. However, the coefficients of variation (COV) of calculated to measured ratios, q_{sc}/q_{sm} and q_{bc}/q_{bm}, are large and, in general, indicate a higher level of uncertainty compared with a mean COV value of 0.4 expected for drilled shafts (Phoon and Retief 2016).

The mean (μ) and COV for ratios of calculated to measured shaft frictions are summarized in Table 3. The COVs for q_{sc}/q_{sm} obtained for the LCPC-1982, Eslami and Fellenius (1997), and Niazi and Mayne (2016) methods have an average of about 0.5. This is higher than reported by the associated references for these methods and suggests that, if the ratios of q_{sc}/q_{sm} are normally distributed, there is a probability as high as $1/40$ that the calculated shaft friction of a pile is more than two times the actual capacity (noting 2 is a typical factor of safety). The LCPC-2012 method has the best statistics with a COV for q_{sm}/q_{sc} of 0.33 and, for this method, the probability that the calculated shaft friction of a pile is more than two times the actual capacity is less than $1/750$ (again assuming a normal distribution).

The COV for the ratios of calculated to measured base capacities (q_{bc}/q_{bm}) are a little lower than the corresponding ratios for shaft friction although the mean values (μ) are generally considerably

Table 1. Database of pile load tests

Number	References	Loading direction	Boring methods	Test site	Soil description	D (m)	L (m)	L_s (m)	Pile shaft σ'_{vo} (kPa)	Pile shaft q_t (kPa)	Pile shaft F_r (%)	Pile shaft I_c (or I_c^*)	Pile toe σ'_{vo} (kPa)	Pile toe q_t (kPa)	Pile toe F_r (%)	Pile toe I_c
1	Carvalho and Albuquerque (2013)	T	Dry	São Paulo, Brazil	Sediment	0.35	10.0	3.5	57	1,070	4.4	2.9	—	—	—	—
	Carvalho and Albuquerque (2013)	T	Dry	São Paulo, Brazil	Residual clay	0.35	10.0	9.0	155	2,360	6.8	3.1	—	—	—	—
2	Carvalho and Albuquerque (2013)	T	Dry	São Paulo, Brazil	Sediment	0.40	10.0	3.5	57	1,070	4.4	2.9	—	—	—	—
	Carvalho and Albuquerque (2013)	T	Dry	São Paulo, Brazil	Residual clay	0.40	10.0	9.0	155	2,360	6.8	3.1	—	—	—	—
3	Carvalho and Albuquerque (2013)	T	Dry	São Paulo, Brazil	Sediment	0.40	10.0	3.5	57	1,070	4.4	2.9	—	—	—	—
	Carvalho and Albuquerque (2013)	T	Dry	São Paulo, Brazil	Residual clay	0.50	10.0	9.0	155	2,360	6.8	3.1	—	—	—	—
4	Albuquerque et al. (2011)	C	Dry	São Paulo, Brazil	Residual/colluvial clay	0.40	12.0	2.5	34	2,500	1.1	2.2	—	—	—	—
	Albuquerque et al. (2011)	C	Dry	São Paulo, Brazil	Residual clay	0.40	12.0	8.5	122	2,000	6.6	3.1	177	2,060	6.5	3.2
5	Albuquerque et al. (2011)	C	Dry	São Paulo, Brazil	Residual/colluvial clay	0.40	12.0	2.5	34	2,500	1.1	2.2	—	—	—	—
	Albuquerque et al. (2011)	C	Dry	São Paulo, Brazil	Residual clay	0.40	12.0	8.5	122	2,000	6.6	3.1	177	2,060	6.5	3.2
6	Albuquerque et al. (2011)	C	Dry	São Paulo, Brazil	Residual/colluvial clay	0.40	12.0	2.5	34	2,500	1.1	2.2	—	—	—	—
	Albuquerque et al. (2011)	C	Dry	São Paulo, Brazil	Residual clay	0.40	12.0	8.5	122	2,000	6.6	3.1	177	2,060	6.5	3.2
7	Albuquerque et al. (2011)	C	CFA	São Paulo, Brazil	Residual/colluvial clay	0.40	12.0	2.5	34	2,500	1.1	2.2	—	—	—	—
	Albuquerque et al. (2011)	C	CFA	São Paulo, Brazil	Residual clay	0.40	12.0	8.5	122	2,000	6.6	3.1	177	2,060	6.5	3.2
8	Albuquerque et al. (2011)	C	CFA	São Paulo, Brazil	Residual/colluvial clay	0.40	12.0	2.5	34	2,500	1.1	2.2	—	—	—	—
	Albuquerque et al. (2011)	C	CFA	São Paulo, Brazil	Residual clay	0.40	12.0	8.5	122	2,000	6.6	3.1	177	2,060	6.5	3.2
9	Albuquerque et al. (2011)	C	CFA	São Paulo, Brazil	Residual/colluvial clay	0.40	12.0	2.5	34	2,500	1.1	2.2	—	—	—	—
	Albuquerque et al. (2011)	C	CFA	São Paulo, Brazil	Residual clay	0.40	12.0	8.5	122	2,000	6.6	3.1	177	2,060	6.5	3.2
10	da Fonseca and Santos (2003)	C	Casing	Porto, Portugal	Residual silt	0.60	6.0	3.0	54	3,899	5.3	2.6	108	5,298	4.8	2.6
11	da Fonseca and Santos (2003)	C	CFA	Porto, Portugal	Residual silt	0.60	6.0	3.0	54	3,899	5.3	2.6	108	5,298	4.8	2.6
12	Zein and Ayoub (2012)	C	Dry	Khartoum, Sudan	Clayey sand and silty clay	0.20	3.5	1.8	32	4,200	5.6	2.4	63	4,200	5.7	2.6
13	Mayne and Harris (1993)	C	Dry	Georgia, US	Residual silt and sand	0.76	16.9	8.5	137	5,205	1.9	2.4	262	7,320	2.5	2.6
14	Mayne and Harris (1993)	C	Dry	Georgia, US	Residual silt and sand	0.76	21.4	8.5	137	5,205	1.9	2.4	283	31,990	0.7	1.6
15	Park et al. (2011)	C	CFA	Kansas, US	Sand, silt, clay	0.46	22.9	11.4	206	27,200	0.6	1.6	411	27,200	2.0	2.2
16	Brown (2002)	C	Casing	Alabama, US	Residual clayey-silt	0.90	11.0	5.5	100	3,364	4.6	2.7	141	3,382	4.7	2.8
17	Brown (2002)	C	Bentonite	Alabama, US	Residual clayey-silt	0.90	11.0	5.5	100	3,364	4.6	2.7	141	3,382	4.7	2.8
18	Brown (2002)	C	Dry polymer	Alabama, US	Residual clayey-silt	0.90	11.0	5.5	100	3,364	4.6	2.7	141	3,382	4.7	2.8
19	Brown (2002)	C	Liquid polymer	Alabama, US	Residual clayey-silt	0.90	11.0	5.5	100	3,364	4.6	2.7	141	3,382	4.7	2.8
20	Brown (2002)	C	CFA	Alabama, US	Residual clayey-silt	0.45	11.0	5.5	100	3,364	4.6	2.7	141	3,382	4.7	2.8
21	Briaud et al. (2000)	C	Dry	Texas, US	Pleistocene stiff clay	0.91	10.4	5.2	92	4,000	7.7	2.8	162	6,009	8.6	2.9
22	Briaud et al. (2000)	C	Bentonite	Texas, US	Pleistocene sand	0.91	10.4	5.2	90	8,400	1.2	2.0	159	8,406	1.2	2.1
23	Brown et al. (2006)	C	Dry	Grimsby, UK	Stiff to firm silty clay	0.60	11.8	5.9	59	2,120	3.1	2.6	118	2,020	3.6	2.9
24	Elbanna et al. (2007)	T	Dry	Alberta, Canada	Silt and clayey silt	1.40	22.1	12.3	158	4,100	3.4	2.7	—	—	—	—
	Elbanna et al. (2007)	T	Dry	Alberta, Canada	Silt and clayey silt	1.40	22.1	14.8	178	4,100	3.4	2.8	—	—	—	—
	Elbanna et al. (2007)	T	Dry	Alberta, Canada	Silt and clayey silt	1.40	22.1	17.0	196	4,100	3.4	2.8	237	4,100	3.5	2.9
25	Finno et al. (1989)	C	Bentonite	Illinois, US	Sand, silty sand	0.46	14.5	3.7	29	18,007	1.0	1.6	—	—	—	—
	Finno et al. (1989)	C	Bentonite	Illinois, US	Soft firm clay	0.46	14.5	10.9	131	650	3.7	3.5	116	660	3.4	3.4
26	Finno et al. (1989)	C	Casing	Illinois, US	Sand, silty sand	0.46	14.5	3.7	29	18,007	1.0	1.6	—	—	—	—
	Finno et al. (1989)	C	Casing	Illinois, US	Soft firm clay	0.46	14.5	10.9	131	650	3.7	3.5	116	660	3.4	3.4
27	Iskander et al. (2003)	C	Dry	Massachusetts, US	Varved clay	0.9	14.4	11.5	92	656	2.9	3.2	115	656	3.3	3.4
28	O'Neill and Reese (1970)	C	Dry	Texas, US	Beaumont clay	0.78	7.0	3.5	70	2,240	3.7	2.7	120	2,220	3.8	2.9
29	Doan and Lehane (2019)	C	Dry	Western Australia, Australia	Silt mixtures, high OCR	0.30	3.8	1.9	48	8,140	5.8	2.4	87	10,890	4.4	2.3
30	Durham (2006)	C	CFA	Western Australia, Australia	Sand and silt mixtures	0.30	7.0	3.5	38	10,914	0.7	1.6	—	—	—	—
31	Durham (2006)	C	CFA	Western Australia, Australia	Sand and silt mixtures	0.30	7.0	6.0	58	1,886	1.5	2.5	66	1,886	1.5	2.5
32	Lehane (2009)	C	CFA	Western Australia, Australia	Sand	0.23	4.0	2.0	36	9,000	0.5	1.6	—	—	—	—
33	Lehane (2009)	C	CFA	Western Australia, Australia	Sand	0.34	4.0	2.0	36	9,000	0.5	1.6	—	—	—	—
34	Lehane (2009)	C	CFA	Western Australia, Australia	Sand	0.34	10.5	5.3	62	4,007	0.5	2.0	—	—	—	—

Table 1. (*Continued.*)

Number	References	Loading direction	Boring methods	Test site	Soil description	D (m)	L (m)	L_s (m)	Pile shaft σ'_{vo} (kPa)	Pile shaft q_t (kPa)	Pile shaft F_r (%)	Pile shaft I_c (or I_c^*)	Pile toe σ'_{vo} (kPa)	Pile toe q_t (kPa)	Pile toe F_r (%)	Pile toe I_c
35	Lehane (2009)	C	CFA	Western Australia, Australia	Sand	0.34	10.5	5.3	62	4,000	0.5	2.0	—	—	—	—
36	Lehane (2009)	C	CFA	Western Australia, Australia	Sand	0.45	5.5	2.8	50	15,000	0.5	1.5	—	—	—	—
37	Lehane (2009)	C	CFA	Western Australia, Australia	Sand	0.45	5.5	2.8	50	15,000	0.5	1.5	—	—	—	—
38	Lehane (2009)	C	CFA	Western Australia, Australia	Sand	0.45	6.7	3.4	60	6,800	0.5	1.8	—	—	—	—
39	Pine (2016)	C	CFA	Western Australia, Australia	Sand	0.60	15.0	7.5	135	16,000	0.5	1.6	—	—	—	—
40	Tucker (1986)	T	—	California, US	Silty sand	0.53	6.7	3.4	60	9,600	0.5	1.7	—	—	—	—
41	Franke and Garbrecht (1977)	C	Bentonite	Germany	Silty sand	1.10	6.0	3.0	54	17,000	0.5	1.5	—	—	—	—
42	Franke and Garbrecht (1977)	C	Bentonite	Germany	Silty sand	1.50	6.0	3.0	54	16,000	0.5	1.5	—	—	—	—
43	Konstantinidis et al. (1987)	T	—	California, US	Silty sand and gravel	0.69	4.6	2.3	41	28,000	0.80	1.4	—	—	—	—
44	Konstantinidis et al. (1987)	T	—	California, US	Silty sand and gravel	0.67	3.1	1.6	28	20,000	0.80	1.5	—	—	—	—
45	Konstantinidis et al. (1987)	T	—	Nevada, US	Silty clay	0.65	4.6	2.3	39	28,000	4.00	2.0	—	—	—	—
46	Konstantinidis et al. (1987)	T	—	Nevada, US	Silty clay	0.65	3.0	1.5	26	22,500	4.00	2.0	—	—	—	—
47	Konstantinidis et al. (1987)	T	—	Utah, US	Silty clay	0.66	3.2	1.6	27	2,500	5.3	2.5	—	—	—	—
48	Konstantinidis et al. (1987)	T	—	Utah, US	Silty clay	0.66	4.7	2.4	40	5,000	5.3	2.4	—	—	—	—
49	Kruizinga (1975)	C	Bentonite	Bachy, Netherlands	Sand	0.63	18.0	9.0	72	11,018	0.5	1.6	144	11,018	0.5	1.6
50	Kruizinga (1975)	C	Bentonite	Soleton, Netherlands	Sand	0.59	12.5	6.3	50	11,013	0.5	1.6	100	25,025	0.5	1.4
51	Kruizinga (1975)	C	Bentonite	Soleton, Netherlands	Sand	0.59	18.4	9.2	74	15,018	0.5	1.5	147	2,537	4.0	2.9
52	Caputo and Viggiani (1988)	C	Bentonite	Naples, Italy	Silty sand	1.50	42.0	21.0	157	9,998	0.5	1.9	304	10,082	0.5	2.1
53	Mandolini et al. (2002)	C	CFA	Naples, Italy	Silty sand	0.80	24.0	12.0	94	2,835	0.5	2.2	178	15,865	0.5	1.7
54	Mandolini et al. (2002)	C	CFA	Naples, Italy	Silty sand	0.60	22.5	11.3	89	2,748	0.5	2.2	168	10,796	0.5	1.8
55	Mandolini et al. (2002)	C	CFA	Naples, Italy	Silty sand	0.80	24.1	12.1	94	2,314	0.5	2.3	179	17,516	0.5	1.7
56	Cadogan and Gavin (2006)	C	Dry	Dublin, Ireland	Overconsolidated sand	0.10	2.0	1.0	20	17,000	2.0	1.8	40	17,000	2.0	1.8
57	Cadogan et al. (2010)	C	Dry	Dublin, Ireland	Overconsolidated sand	0.10	2.0	1.0	20	15,000	2.0	1.8	40	15,000	2.0	1.9
58	Cadogan et al. (2010)	C	Dry	Dublin, Ireland	Overconsolidated sand	0.20	3.0	1.5	30	15,000	2.0	1.8	60	15,765	2.0	1.9
59	Gavin et al. (2009)	C	CFA	Killarney, Ireland	Loose and dense sand	0.80	14.0	7.5	60	4,626	0.5	1.9	112	10,628	0.5	1.7
60	Gavin et al. (2009)	C	CFA	Killarney, Ireland	Loose and dense sand	0.45	15.0	9.0	72	4,344	0.5	2.0	120	10,630	0.5	1.8
61	Current study	C	Dry	Western Australia, Australia	Silt mixtures, high OCR	0.75	15.0	7.5	150	7,073	3.0	2.5	—	—	—	—
62	Current study	C	CFA	South Australia, Australia	Silt	0.75	15.3	6.0	108	3,170	5.0	2.8	—	3,170	—	—
63	Current study	C	CFA	Western Australia, Australia	Sand	0.60	14.0	7.0	91	11,637	0.5	1.7	—	—	—	—
64	Current study	C	CFA	Western Australia, Australia	Sand	0.60	14.0	7.0	91	9,957	0.5	1.7	—	—	—	—
65	Current study	C	CFA	Western Australia, Australia	Sand	0.50	9.2	4.6	83	35,660	0.5	1.3	—	—	—	—
66	Current study	C	CFA	Western Australia, Australia	Sand	0.50	9.2	4.6	83	35,660	0.5	1.3	—	—	—	—
67	Current study	C	CFA	Western Australia, Australia	Sand	0.45	10.0	5.0	40	16,400	0.5	1.4	—	—	—	—
68	Current study	C	CFA	South Australia, Australia	Silty clay/clays	0.45	13.5	7.0	126	2,913	4.9	2.9	243	13,721	2.8	2.4

Note: T = tension; and C = compression.

Table 2. Prediction for database piles

Number	References	Test site	Measured		LCPC-82		LCPC-2012		Eslami and Fellenius (1997)		Niazi and Mayne (2016)	
			q_{sm} (kPa)	q_{bm} (kPa)	q_{sc}/q_{sm}	q_{bc}/q_{bm}	q_{sc}/q_{sm}	q_{bc}/q_{bm}	q_{sc}/q_{sm}	q_{bc}/q_{bm}	q_{sc}/q_{sm}	q_{bc}/q_{bm}
1	Carvalho and Albuquerque (2013)	São Paulo, Brazil	26	—	0.7	—	0.8	—	1.0	—	1.2	—
	Carvalho and Albuquerque (2013)	São Paulo, Brazil	57	—	1.0	—	0.9	—	2.1	—	1.6	—
2	Carvalho and Albuquerque (2013)	São Paulo, Brazil	22	—	0.8	—	0.9	—	1.2	—	1.3	—
	Carvalho and Albuquerque (2013)	São Paulo, Brazil	66	—	0.9	—	0.8	—	1.8	—	1.4	—
3	Carvalho and Albuquerque (2013)	São Paulo, Brazil	21	—	0.9	—	1.0	—	1.3	—	1.5	—
	Carvalho and Albuquerque (2013)	São Paulo, Brazil	62	—	0.9	—	0.9	—	1.9	—	1.5	—
4	Albuquerque et al. (2011)	São Paulo, Brazil	39	—	1.1	—	1.1	—	0.6	—	0.7	—
	Albuquerque et al. (2011)	São Paulo, Brazil	44	—	1.1	—	1.1	—	2.3	—	2.2	—
5	Albuquerque et al. (2011)	São Paulo, Brazil	21	—	3.0	—	2.0	—	1.2	—	1.3	—
	Albuquerque et al. (2011)	São Paulo, Brazil	54	—	0.9	—	0.9	—	1.9	—	1.8	—
6	Albuquerque et al. (2011)	São Paulo, Brazil	35	—	1.2	—	1.2	—	0.7	—	0.8	—
	Albuquerque et al. (2011)	São Paulo, Brazil	46	—	1.1	—	1.1	—	2.2	—	2.1	—
7	Albuquerque et al. (2011)	São Paulo, Brazil	80	—	0.5	—	0.5	—	0.3	—	0.3	—
	Albuquerque et al. (2011)	São Paulo, Brazil	47	760	1.1	0.9	1.1	1.2	2.1	2.3	2.0	1.5
8	Albuquerque et al. (2011)	São Paulo, Brazil	80	—	0.5	—	0.5	—	0.3	—	0.3	—
	Albuquerque et al. (2011)	São Paulo, Brazil	53	530	0.9	1.4	1.0	1.7	1.9	3.3	1.8	2.2
9	Albuquerque et al. (2011)	São Paulo, Brazil	69	—	0.6	—	0.6	—	0.4	—	0.4	—
	Albuquerque et al. (2011)	São Paulo, Brazil	36	—	1.4	—	1.4	—	2.8	—	2.7	—
10	da Fonseca and Santos (2003)	Porto, Portugal	62	1,149	1.1	2.1	0.9	1.4	0.6	4.6	1.3	2.0
11	da Fonseca and Santos (2003)	Porto, Portugal	59	1,538	1.1	1.5	1.0	1.0	0.7	3.5	1.3	1.5
12	Zein and Ayoub (2012)	Khartoum, Sudan	72	796	1.0	2.1	0.8	1.6	0.6	5.3	1.0	2.2
13	Mayne and Harris (1993)	Georgia, US	80	1,792	0.6	1.8	0.8	1.2	0.7	4.1	1.0	1.7
14	Mayne and Harris (1993)	Georgia, US	73	7,000	0.7	1.4	0.9	1.4	0.7	4.6	1.1	0.9
15	Park et al. (2011)	Kansas, US	95	6,017	1.9	1.4	1.2	1.1	1.1	4.5	1.1	1.4
16	Brown (2002)	Alabama, US	55	1,500	1.0	1.0	0.9	0.7	1.6	2.3	1.7	1.2
17	Brown (2002)	Alabama, US	36	1,000	1.6	1.5	1.4	1.0	2.4	3.5	2.6	1.8
18	Brown (2002)	Alabama, US	59	1,150	1.0	1.3	0.9	0.9	1.5	3.0	1.6	1.5
19	Brown (2002)	Alabama, US	72	1,500	0.8	1.0	0.7	0.7	1.2	2.3	1.3	1.2
20	Brown (2002)	Alabama, US	45	—	1.2	—	1.2	—	1.9	—	2.1	—
21	Briaud et al. (2000)	Texas, US	72	1,598	0.9	1.5	0.8	1.1	1.4	3.7	1.7	2.0
22	Briaud et al. (2000)	Texas, US	109	1,065	0.8	3.2	0.6	2.4	0.3	7.9	0.6	2.4
23	Brown et al. (2006)	Grimsby, UK	71	776	0.5	0.9	0.5	1.0	0.7	2.5	0.6	1.3
24	Elbanna et al. (2007)	Alberta, Canada	150	—	0.5	—	0.4	—	0.6	—	0.5	—
	Elbanna et al. (2007)	Alberta, Canada	106	—	0.6	—	0.6	—	0.9	—	0.8	—
	Elbanna et al. (2007)	Alberta, Canada	77	1,600	0.9	1.0	0.8	0.8	1.2	2.3	1.1	1.2
25	Finno et al. (1989)	Illinois, US	147	—	0.8	—	0.7	—	0.5	—	0.5	—
	Finno et al. (1989)	Illinois, US	24	402	0.9	0.7	0.8	0.7	0.3	0.3	0.6	0.2
26	Finno et al. (1989)	Illinois, US	130	—	0.9	—	0.7	—	0.6	—	0.5	—
	Finno et al. (1989)	Illinois, US	28	—	0.8	—	0.7	—	0.3	—	0.5	—
27	Iskander et al. (2003)	Massachusetts, US	18	306	1.2	0.9	1.1	0.9	0.4	0.5	0.6	0.4
28	O'Neill and Reese (1970)	Texas, US	48	967	0.8	0.9	0.8	0.7	1.1	2.2	1.1	1.1
29	Doan and Lehane (2019)	Western Australia, Australia	148	3,071	0.6	1.4	0.6	1.1	0.6	3.6	0.8	1.2
30	Durham (2006)	Western Australia, Australia	73	—	2.5	—	1.1	—	0.6	—	0.6	—
31	Durham (2006)	Western Australia, Australia	27	472	1.2	—	1.3	—	0.7	—	1.2	—
32	Lehane (2009)	Western Australia, Australia	96	—	0.9	—	0.8	—	0.4	—	0.4	—
33	Lehane (2009)	Western Australia, Australia	63	—	1.4	—	1.2	—	0.6	—	0.6	—
34	Lehane (2009)	Western Australia, Australia	27	—	2.5	—	1.6	—	0.6	—	1.1	—
35	Lehane (2009)	Western Australia, Australia	25	—	2.7	—	1.7	—	0.6	—	1.2	—
36	Lehane (2009)	Western Australia, Australia	80	—	1.2	—	1.2	—	0.7	—	0.6	—
37	Lehane (2009)	Western Australia, Australia	80	—	1.2	—	1.2	—	0.7	—	0.6	—
38	Lehane (2009)	Western Australia, Australia	50	—	1.4	—	1.2	—	0.5	—	0.7	—
39	Pine (2016)	Western Australia, Australia	91	—	1.2	—	1.1	—	0.7	—	0.7	—
40	Tucker (1986)	California, US	52	—	1.9	—	1.4	—	0.7	—	0.7	—
41	Franke and Garbrecht (1977)	Germany	65	—	1.7	—	1.4	—	1.0	—	0.8	—
42	Franke and Garbrecht (1977)	Germany	108	—	1.0	—	0.9	—	0.6	—	0.5	—
43	Konstantinidis et al. (1987)	California, US	80	—	2.3	—	1.4	—	1.4	—	0.8	—
44	Konstantinidis et al. (1987)	California, US	61	—	2.2	—	1.6	—	1.3	—	0.8	—
45	Konstantinidis et al. (1987)	Nevada, US	149	—	1.3	—	0.8	—	0.8	—	1.1	—
46	Konstantinidis et al. (1987)	Nevada, US	147	—	1.0	—	0.7	—	0.6	—	0.9	—
47	Konstantinidis et al. (1987)	Utah, US	66	—	0.4	—	0.6	—	0.4	—	0.6	—
48	Konstantinidis et al. (1987)	Utah, US	72	—	0.7	—	0.9	—	0.7	—	0.9	—
49	Kruizinga (1975)	Bachy, Netherlands	75	2,500	1.5	1.8	1.0	0.9	0.6	4.4	0.6	0.9
50	Kruizinga (1975)	Soleton, Netherlands	80	4,400	1.4	1.7	1.0	1.1	0.5	5.7	0.5	1.0

Table 2. (*Continued*.)

Number	References	Test site	Measured		LCPC-82		LCPC-2012		Eslami and Fellenius (1997)		Niazi and Mayne (2016)	
			q_{sm} (kPa)	q_{bm} (kPa)	q_{sc}/q_{sm}	q_{bc}/q_{bm}	q_{sc}/q_{sm}	q_{bc}/q_{bm}	q_{sc}/q_{sm}	q_{bc}/q_{bm}	q_{sc}/q_{sm}	q_{bc}/q_{bm}
51	Kruizinga (1975)	Soleton, Netherlands	106	1,000	0.9	0.9	0.8	1.0	0.6	2.4	0.5	1.3
52	Caputo and Viggiani (1988)	Naples, Italy	63	1,415	1.6	2.9	1.2	2.1	0.6	6.8	1.0	2.0
53	Mandolini et al. (2002)	Naples, Italy	45	2,800	1.0	1.7	1.1	1.4	0.6	5.6	0.7	1.2
54	Mandolini et al. (2002)	Naples, Italy	60	3,000	0.8	1.4	0.8	0.9	0.4	3.5	0.5	0.8
55	Mandolini et al. (2002)	Naples, Italy	55	2,900	0.7	1.8	0.7	1.5	0.4	6.0	0.5	1.2
56	Cadogan and Gavin (2006)	Dublin, Ireland	143	4,500	0.8	1.1	0.7	0.8	0.5	3.8	0.6	0.9
57	Cadogan et al. (2010)	Dublin, Ireland	120	3,500	0.8	1.3	0.7	0.9	0.5	4.3	0.7	1.0
58	Cadogan et al. (2010)	Dublin, Ireland	120	4,090	0.8	1.2	0.7	0.8	0.5	3.9	0.7	1.0
59	Gavin et al. (2009)	Killarney, Ireland	35	2,400	2.2	1.8	1.4	1.1	0.5	4.4	0.9	1.0
60	Gavin et al. (2009)	Killarney, Ireland	36	2,280	2.0	1.9	1.3	1.2	0.5	4.6	0.9	1.0
61	Current study	Western Australia, Australia	90	—	0.8	—	0.9	—	0.8	—	1.4	—
62	Current study	South Australia, Australia	97	—	0.5	—	0.5	—	0.8	—	1.0	—
63	Current study	Western Australia, Australia	53	—	2.2	—	1.6	—	0.9	—	1.0	—
64	Current study	Western Australia, Australia	59	—	1.7	—	1.3	—	0.7	—	0.8	—
65	Current study	Western Australia, Australia	204	—	1.2	—	0.6	—	0.7	—	0.4	—
66	Current study	Western Australia, Australia	188	—	1.3	—	0.7	—	0.8	—	0.5	—
67	Current study	Western Australia, Australia	82	—	1.3	—	1.2	—	0.8	—	0.6	—
68	Current study	South Australia, Australia	97	2,400	0.5	1.7	0.5	1.7	0.8	5.7	1.0	2.0

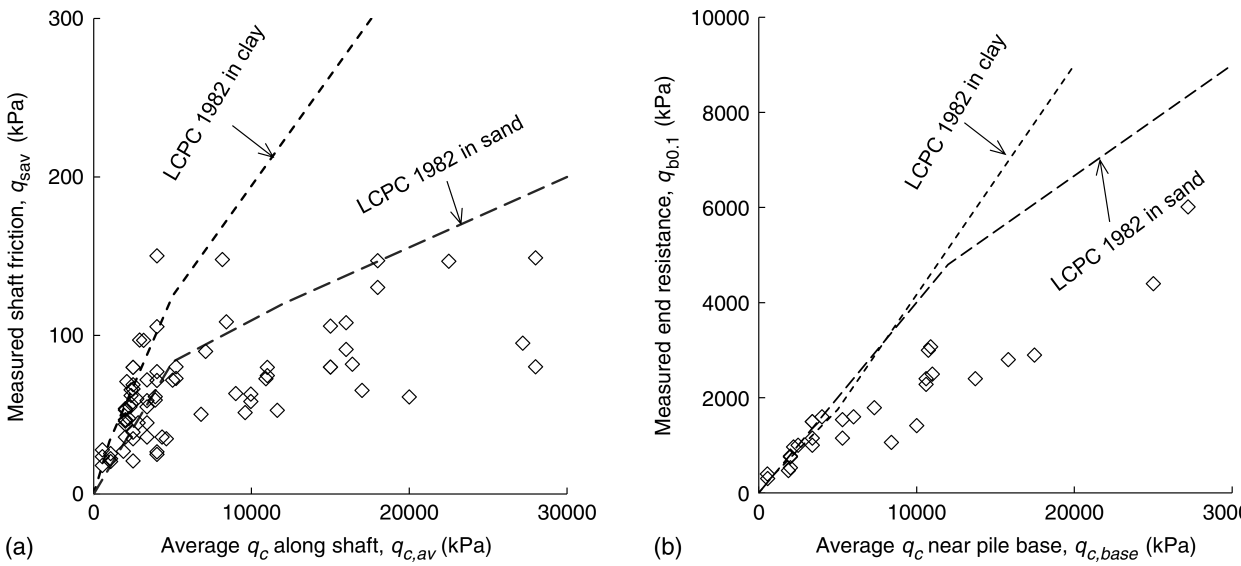

Fig. 4. Comparison of measured shaft friction and base resistance with LCPC-1982 method.

larger than unity and hence nonconservative. The base capacity statistics for LCPC-2012 are the best of the four methods considered.

Considerations for Formulation Development

Doan and Lehane (2020) examined the basis for a correlation between shaft friction (q_s) of buried piles and the CPT q_t value. This study concludes that while q_t reflects the in situ horizontal effective stress and hence provides this basis, the q_t value also depends significantly on the frictional strength and compressibility of the soil surrounding the cone as well as on the degree of drainage during cone insertion. Doan and Lehane (2020) proposed use of the I_c index to reflect these additional parameters and provided evidence in support of a correlation between q_s, q_t, and I_c. Such a correlation is examined in the following sections while also allowing for the potential nonlinear relationship between q_s and q_t, such as indicated in Fig. 4. Other factors that require consideration are first described.

Pore Pressure Measurement (u_2)

Correlations between shaft friction and the effective cone resistance (q_E), such as those adopted in Eqs. (6)–(10), require accurate measurements of the cone pore pressure (u_2), which can often be greater than 50% of q_t in lightly overconsolidated clay. However, the measurement of u_2 is often unreliable as pointed out by Lunne et al. (1997), Mayne (2007), and others, who show that the improper saturation of the porous filter element or desaturation during the cone penetration in dilatant soils results in sluggish

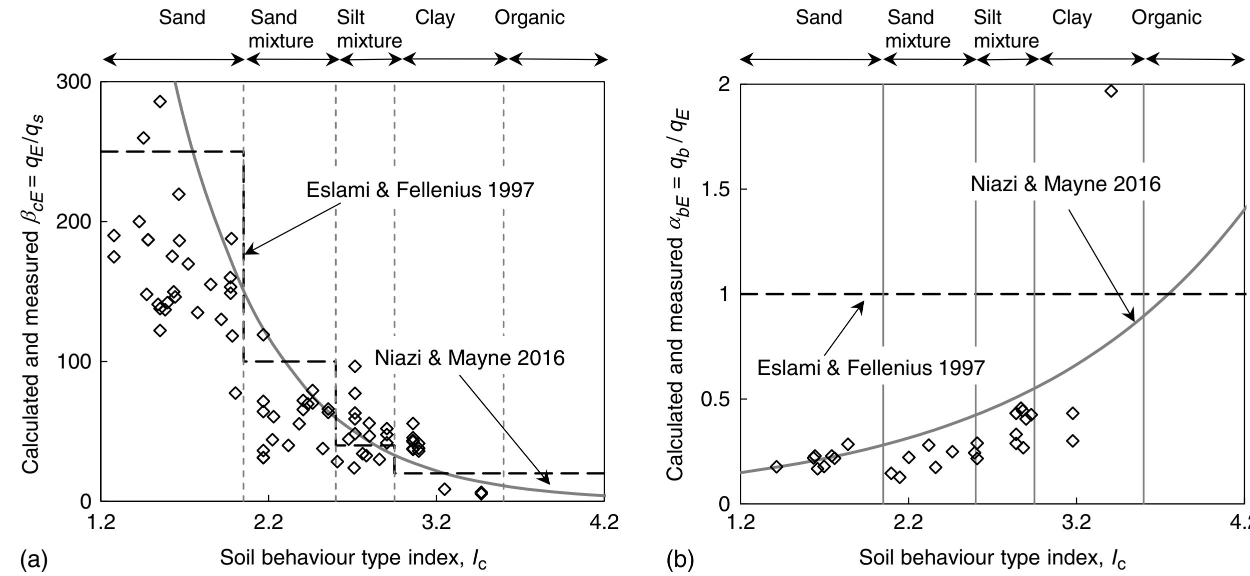

Fig. 5. Relationship between the shaft and base resistances of the database piles with the effective cone resistance.

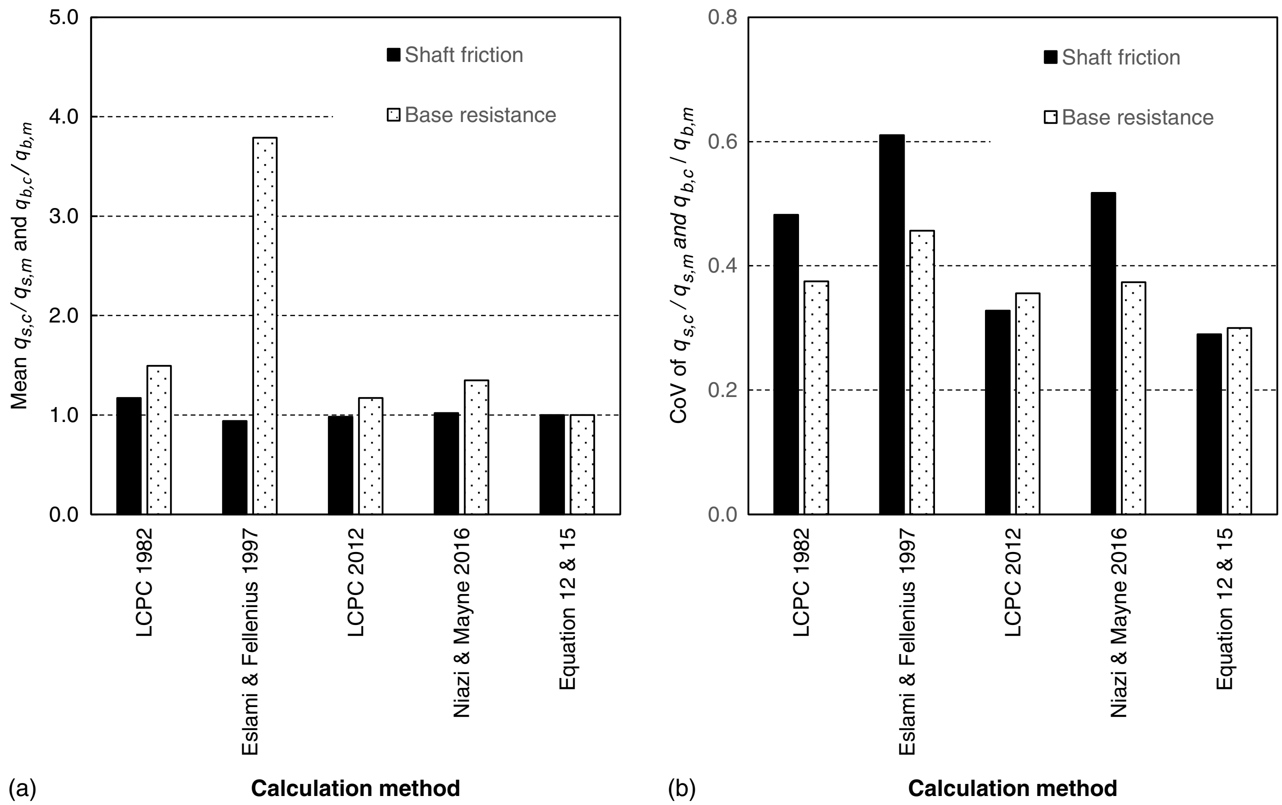

Fig. 6. Performance of different calculation methods.

response times. These effects can be very significant in soils with low cone end resistances, and consequently q_E is not considered a suitable parameter to employ in correlations.

Effects of Dilation and Vertical Effective Stress

Centrifuge tests reported by Foray et al. (1998) and Lehane et al. (2005) demonstrated the importance of dilation on the shaft capacity in sands of rough piles with various diameters and stress levels. The radial expansion or dilation that arises due to shearing induced at the interface of a pile leads to an increase in lateral stress ($\Delta\sigma'_{rd}$) and hence in the friction that can develop because of the constraint to the dilation provided by the surrounding sand mass. The cavity expansion stiffness controls the level of constraint and equals $4G/D$, where G is sand's operational (nonlinear) shear stiffness and D is the pile diameter. As a consequence, the relative

Table 3. Calculated to measured shaft frictions and base resistances

Method	Ratios of calculated to measured capacities			
	Shaft friction (q_{sc}/q_{sm})		Base resistance (q_{bc}/q_{bm})	
	Mean (μ)	COV ($=\sigma/\mu$)	Mean (μ)	COV ($=\sigma/\mu$)
LCPC-1982	1.17	0.48	1.49	0.38
LCPC-2012	0.98	0.33	1.17	0.36
Eslami and Fellenius (1997)	0.94	0.61	3.79	0.46
Niazi and Mayne (2016)	1.02	0.52	1.35	0.37
Eqs. (12) and (15)	1.00	0.29	1.00	0.30
Eqs. (13) and (15)	1.00	0.30	1.00	0.30

contribution of dilation to shaft capacity reduces as the diameter increases and also as the stress level increases.

The influence of dilation on the β_c values is verified in Figs. 7(a) and 8(a), which show that β_c in soils with $I_c < 2.05$ (i.e., sands) is greater (implying q_s is smaller) in piles with larger diameters and at higher stress levels. However, Figs. 7(b) and 8(b), which plot β_c in soils with $I_c > 2.05$ (i.e., silts and clays), show no dependence on diameter or stress level. These trends are consistent with the observations of Doan and Lehane (2020) from pile tests conducted in pressure chambers and direct shear tests that show that the dilation effects are very significant in clean sands but are negligible in clayey sands with a clay fraction exceeding about 4%.

It follows from the foregoing that a relationship between shaft friction and CPT end resistance in sands needs to allow for the

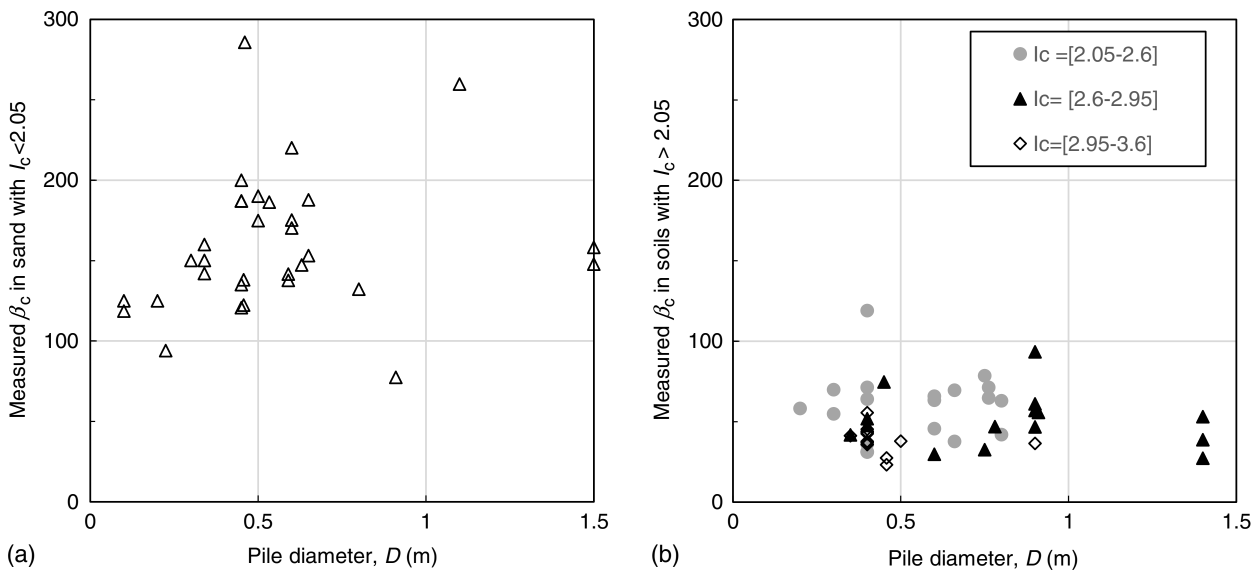

Fig. 7. Effect of pile diameter on the measured β_c values.

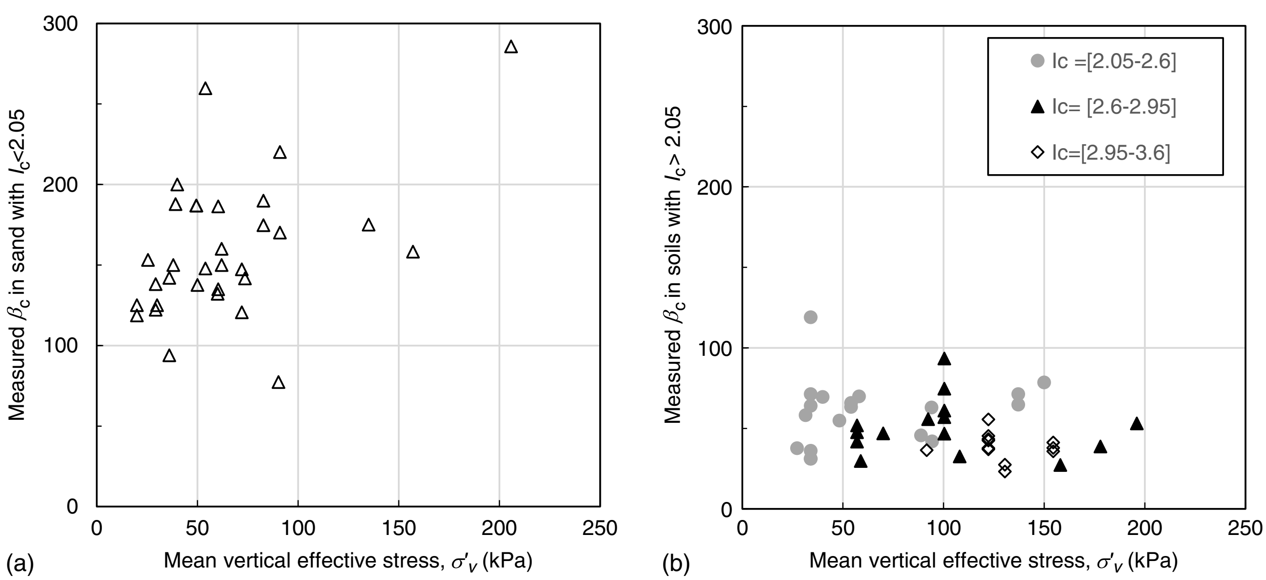

Fig. 8. Effects of stress level on the measured β_c values.

effects of pile diameter and stress level. However, given the relative shortage of high-quality load test data, a mean correlation is proposed in the following that is considered representative of typical drilled shafts. This was achieved by removing piles with diameters less than 300 mm and piles less than 3 m in length from the regression analysis. Lehane et al. (2020) show that, for driven piles, the contribution of dilation for piles with diameters between 300 mm and 1 m is typically about 10%. The same trends have also been exhibited in tension tests on drilled shafts presented by Turner and Kulhawy (1994).

Degree of Saturation (S_r)

Many onshore sites, including a number in the database compiled for this paper, are not fully saturated, i.e., $S_r < 1$. The CPT end resistance of partially saturated soils ($S_r < 1$) differs from the end resistance in saturated deposits due to variable levels of suction. Such differences may also be accentuated by a range of degrees of bonding and cementation. The I_c index cannot be determined when $S_r < 1$ because the vertical effective stress that is used to determine Q_{tn} is unknown. An alternative index, referred to in this paper as I_c^* is employed in the following when $S_r < 1$ and is evaluated using Eq. (8) assuming an ambient water pressure of zero with $\sigma_v' = \sigma_v$.

It has been shown by Niazi and Mayne (2016) that there is general tendency for the value of β_c to reduce with I_c. This tendency is examined in Fig. 9 by plotting the database β_c values for pile tests in both partially saturated and fully saturated soils against I_c but employing I_c^* when $S_r < 1$. It is seen that the trends of much of the data are very similar with most data falling on the best-fit regression line. Despite such relative insensitivity to the degree of saturation, the best-fit correlations presented subsequently do not include the data points obtained in the partially saturated deposits.

Direction of Loading

There is a considerable body of literature indicating that the shaft friction developed on piles in sand under tension loading is about 20% less than that developed under compression loading

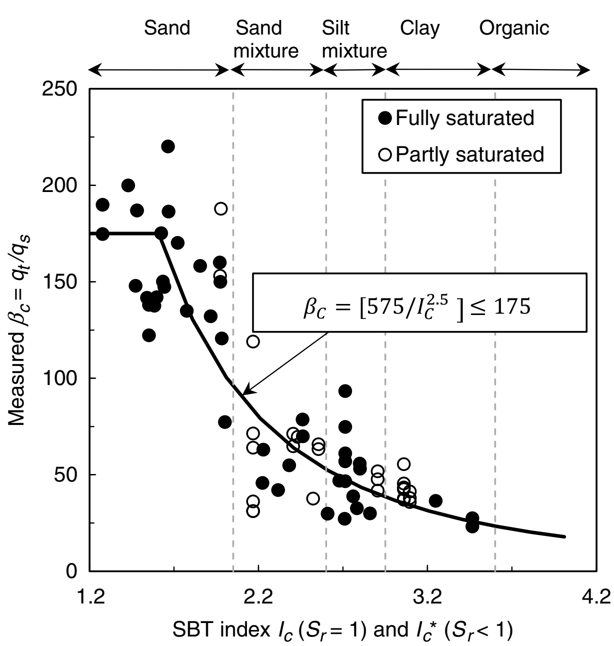

Fig. 9. Comparison of β_c values in partly and fully saturated soils.

(Allen 2005; Brown et al. 2007; De Nicola and Randolph 1993). However, reduced shaft friction under tension loading is not generally observed in silts and clays (Brown et al. 2018; O'Neill 2001). The correlations examined in the following section therefore employ a loading direction parameter, f_t/f_c, which is 1.0 and for all cases except for tension piles in sand when a value of 0.8 is adopted.

Best-Fit Formulations

Shaft Friction

Based on the foregoing, formulations with the following formats were investigated for the database of shaft friction measurements:

$$q_s/(f_t/f_c) = fn \cdot (q_t, I_c, u_2) \tag{11}$$

Statistical analyses indicated that the following expression provided a best overall fit to the database with the lowest coefficient of variation for the ratio of calculated to measured shaft friction

$$q_s = (f_t/f_c) 0.008\, p_a\, (I_c)^{1.6} (q_t/p_a)^{0.8} \quad p_a = 101 \text{ kPa COV} = 0.29 \tag{12}$$

Observations from the analyses are summarized in Figs. 10–12, and detailed as follows:

1. A second expression provided a comparable fit to the database to Eq. (12) and is the best-fit regression line when β_c is expressed as a unique function of I_c (Fig. 9)

$$q_s = (f_t/f_c) q_t/\beta_C \quad \text{where } \beta_C = [575/I_C^{2.5}] \leq 175 \, \text{COV} = 0.30 \tag{13}$$

Eq. (13) leads to β_c values that are comparable to those derived using a similar expression proposed by Doan and Lehane (2020) for buried piles in clayey sand. A maximum cut-off value of 175 is employed in Eq. (13) to provide a better fit to the shaft frictions measured in sands (Fig. 9).

2. Virtually identical mean and COV values were obtained using I_c and I_c^* in Eqs. (12) and (13) suggesting that the relationships are also applicable in partially saturated deposits.
3. The analyses indicated lower COVs when allowance was made for the lower shaft friction developed in tension compared with compression in sands, hence justifying the adoption of the f_t/f_c factor employed.
4. The best-fit relationship obtained using the effective cone resistance (q_E) was as follows:

$$q_s = (f_t/f_c) q_E/\beta_{CE} \quad \text{where } \beta_{CE} = [560/I_C^{2.5}] \, \text{COV} = 0.36 \tag{14}$$

5. Closer inspection of the effect of soil type indicated that COVs for q_{sc}/q_{sm} in the clays were lower than in the sands and silts; see Fig. 10. This observation is consistent with observations made by Alsamman (1995), Phoon and Kulhawy (2005), Reddy and Stuedlein (2017), and others.
6. An examination of the effect of the boring technique in Fig. 11 shows that, on average, the ratios of calculated-to-measured shaft friction of the database piles using Eq. (12) are approximately unity for all boring techniques; a similar trend is shown by Eq. (13). Greatest variability in ratios of calculated-to-measured shaft frictions arises for piles drilled under bentonite. Separate studies by Brown (2002) and Basu et al. (2010) also showed no clear systematic dependence on the boring method.

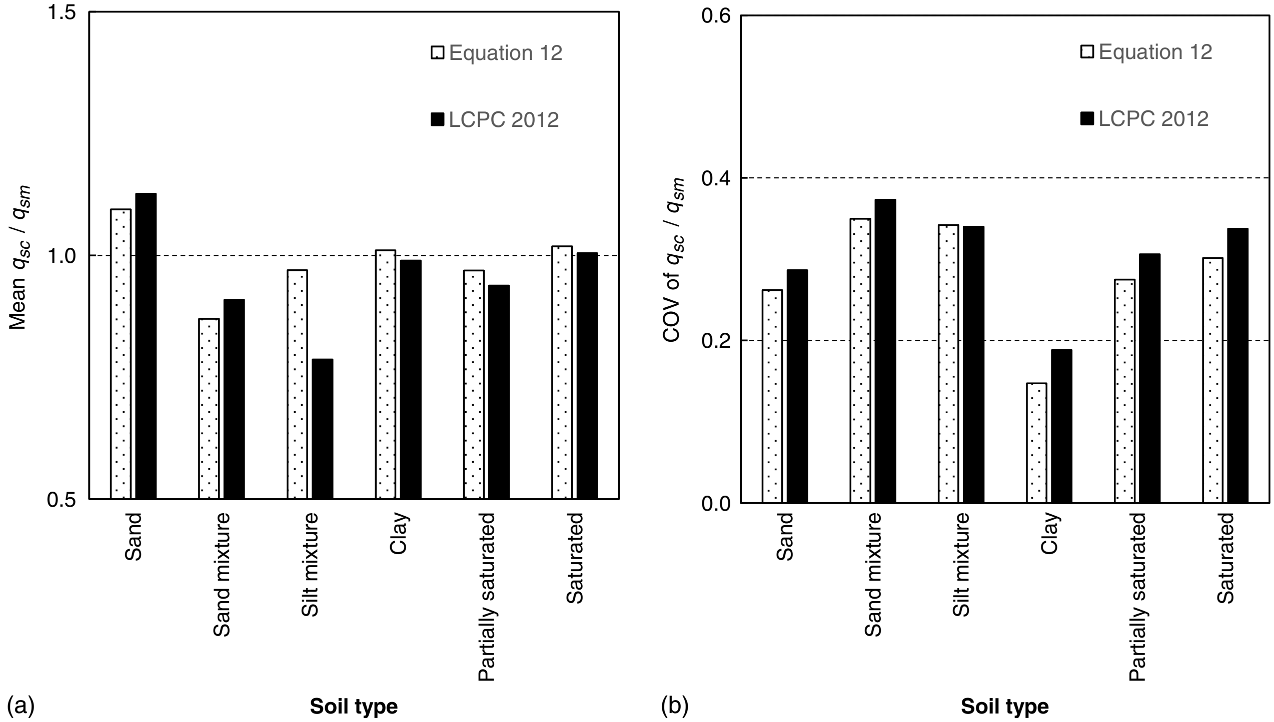

Fig. 10. Performance of Eq. (12) and LCPC-2012 method in different soil types.

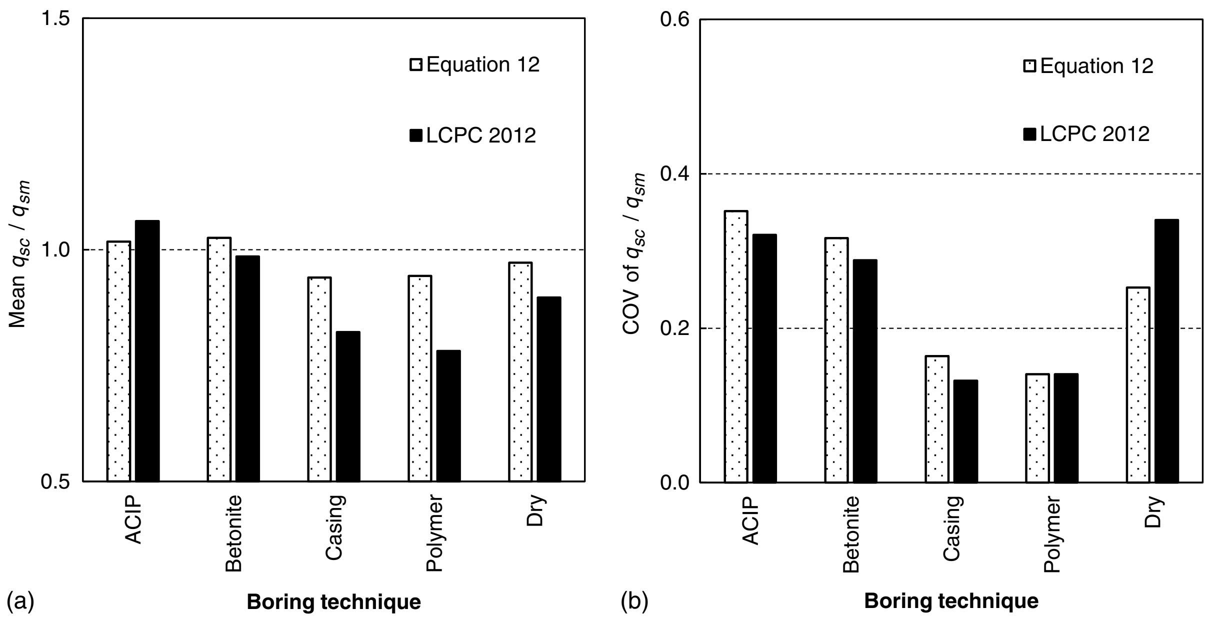

Fig. 11. Performance of Eq. (12) and LCPC-2012 method for piles with different boring techniques.

However, Brown (2002) did note that bores stabilized with polymer lead to greater friction than those supported by bentonite in Piedmont residual soil.

7. The relatively good predictive performance of Eq. (12) and the LCPC-2012 method arises because the latter method indirectly predicts a similar shaft friction, q_s, variation with I_c. This is demonstrated in Fig. 12, which plots the ratios of shaft frictions calculated using Eq. (12) to those calculated using the LCPC-2012 for a database of CPTs available to the authors from Australia (AU) and New Zealand (NZ). The deviation in the plotted ratio about unity is small, indicating good agreement of the two formulations.

Eq. (12) captures the strong influence of soil type via the I_c parameter and also the nonlinear dependence of q_s on q_t that has emerged empirically in this database study and that of LCPC-2012.

The ability of Eq. (12) to predict skin friction distributions is examined by comparing measured and calculated shaft frictions

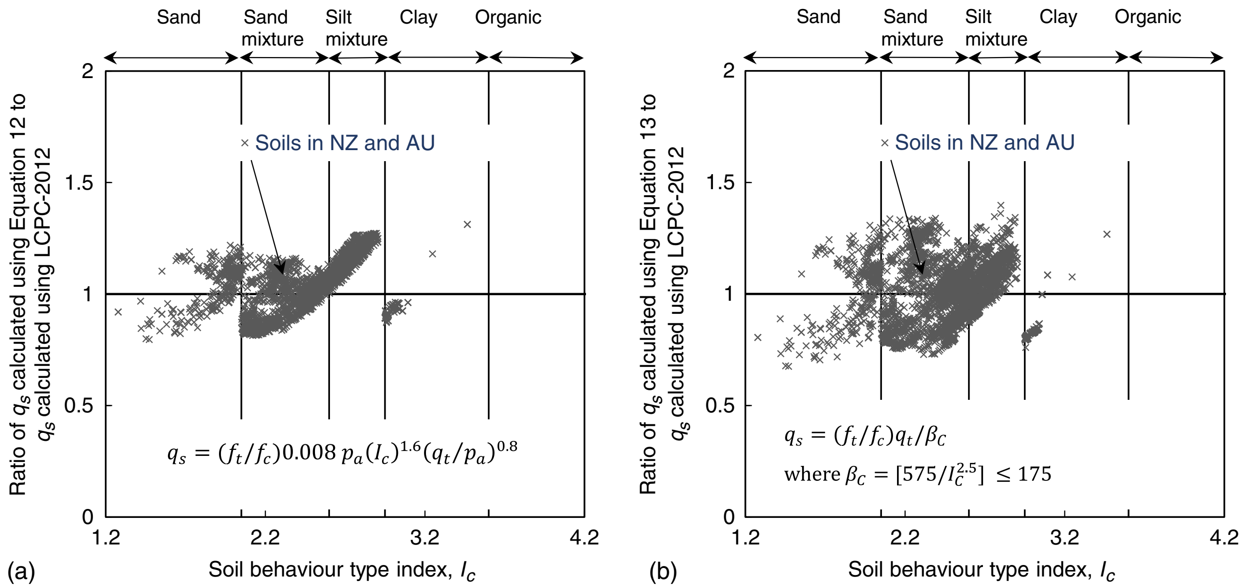

Fig. 12. Comparison between Eqs. (12) and (13) and the LCPC-2012 method using New Zealand and Australia soil data.

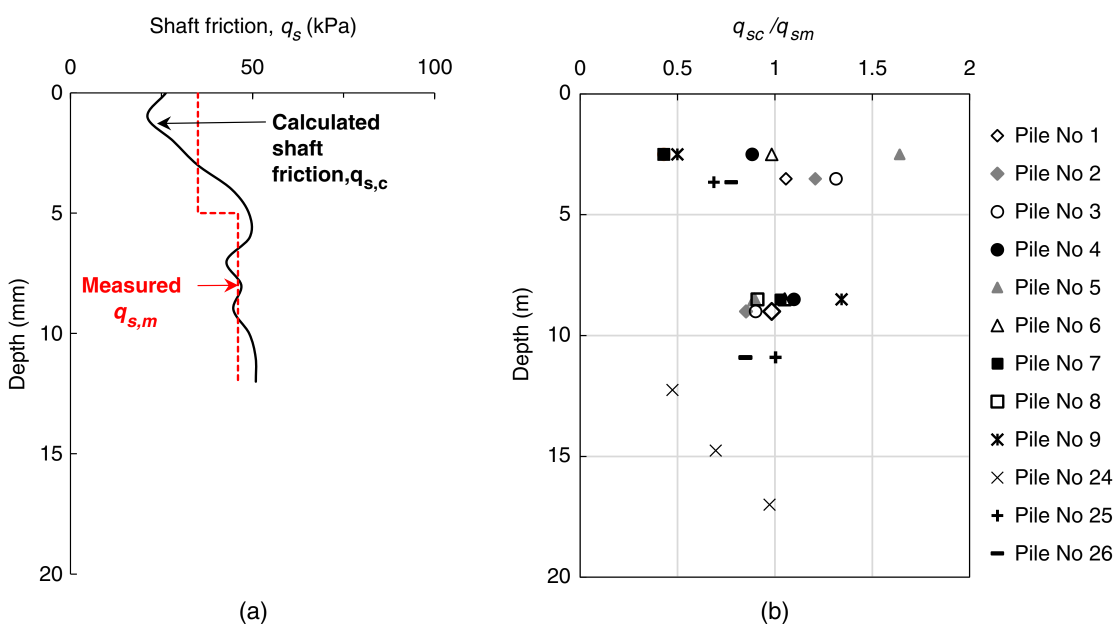

Fig. 13. (a) Comparison of measured shaft friction with friction calculated using Eq. (12) for Pile no. 6; and (b) variation with depth of ratios of calculated to measured localized shaft friction for selected pile tests where pile test numbers correspond with those in Table 1.

($q_{s,m}$ and $q_{s,c}$) for piles in the database comprising more than one segment length (L_s), i.e., where strain gauges permitted measurements over more than one section of the pile shafts. A typical comparison of calculated and measured local frictions is shown in Fig. 13(a) while Fig. 13(b) plots available localized $q_{s,c}/q_{s,m}$ ratios against depth. Fig. 13(b) indicates a spread in ratios in line with the overall COV of Eq. (12). However, importantly, it is seen that there is no systematic dependence of the ratios on the depth to a particular soil horizon, indicating the equation's suitability to predict localized shaft friction for drilled shafts. It can be shown that Eq. (13) gives closely comparable trends to those indicated for Eq. (12) in Fig. 13.

End Bearing

The pile base resistance at a displacement of 10% of the pile diameter ($q_{b0.1}$) also varies with I_c, as proposed by Niazi and Mayne (2016) and shown in Fig. 5. The best-fit simple correlation with I_c achieved for the database is as follows and compared with the database measurements in Fig. 14

$$q_{b0.1} = \alpha_{b0.1} q_t = 0.11 I_c q_t \quad 1.5 \leq I_c \leq 3.6 \quad \mathrm{COV} = 0.30 \quad (15)$$

The coefficient of variation of calculated to measured end bearing stresses is 0.3, which is an improvement on the four other empirical methods considered (Table 3). The expression is consistent

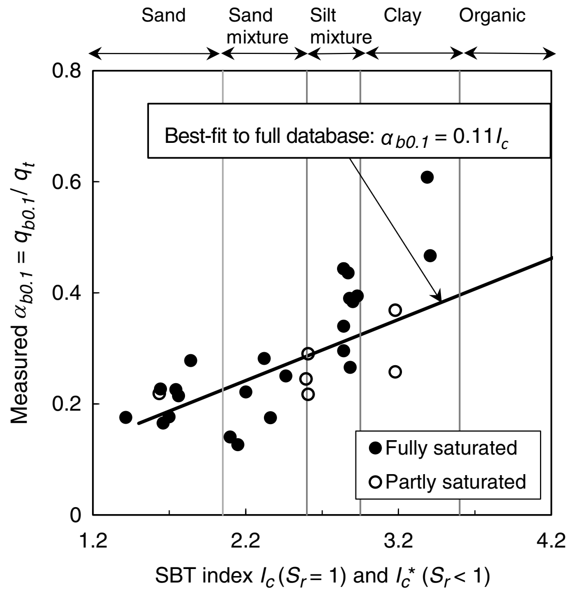

Fig. 14. Database values of $\alpha_{b0.01}$ plotted as a function of soil behavior types.

with analyses such as those of Lee and Salgado (1999) who showed $\alpha_{b0.1}$ $(= q_{b0.1}/q_t)$ reduces with increasing soil stiffness, hence reducing I_c value. Drainage effects also clearly affect the $\alpha_{b0.1}$ ratio, while q_t is likely to be almost fully undrained in silts and clays, the pile end bearing may be fully drained in a typical maintained load static test.

Conclusions

A new and independent database comprising 68 static load tests on largely instrumented drilled shafts and ACIPs is presented. The shaft friction and end bearing measurements are compared with capacities calculated using four popular existing CPT-based approaches. The LCPC-2012 method was shown to be the most reliable approach although, in general, the degree of method uncertainty for all methods was found to be poorer than reported values.

A new formulation for shaft friction (q_s) is proposed that incorporates a dependence on the soil behavior type index (I_c), as seen in experimental studies, and the nonlinear relationship with q_t adopted by the LCPC-2012 method. A direct formulation linking the base capacity and I_c is also proposed. These formulations are consistent with the LCPC-2012 method (which was derived from an independent and larger database) but can be expected to have improved reliability with a coefficient of variation for ratios of calculated-to-measured capacities of about 0.3 in sands, reducing to about 0.2 in clays. These COVs showed no systematic dependence on the boring method or the degree of saturation.

Data Availability Statement

All data and models used during the study appear in the published article.

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Notation

The following symbols are used in this paper:

D = pile diameter;
F_r = CPT normalized friction ratio;
f_s = CPT sleeve friction;
f_t/f_c = loading direction parameter;
G = operational (nonlinear) shear stiffness;
I_c = CPT soil behavior type index;
L = pile length;
L_s = length of a segment of the pile shaft;
Q_{tn} = CPT normalized end resistance;
q_{bc} = calculated end bearing resistance;
q_{bm} = measured end bearing resistance;
$q_{b0.1}$ = end bearing resistance at a displacement of 10% of the pile diameter;
q_c = cone tip resistance;
q_E = effective cone tip resistance;
q_s = unit shaft friction;
q_{sc} = calculated unit shaft friction;
q_{sm} = measured unit shaft friction;
q_t = corrected cone tip resistance;
S_r = degree of saturation;
u_2 = CPT pore pressure;
$\alpha_{b0.1}$ = ratio of end bearing resistance at a displacement of 10% of the pile diameter to cone resistance $q_{b0.1}/q_t$;
$\alpha_{\text{pile-soil}}$ = empirical coefficient;
α_s = ratio of unit shaft friction to cone resistance, q_s/q_t;
β_c = ratio of cone resistance to unit shaft friction, q_t/q_s;
β_{cE} = ratio of effective cone resistance to unit shaft friction, q_E/q_s;
$\Delta\sigma'_{rd}$ = increase in lateral stress;
δ_h = pile head displacement;
σ_v = vertical total stress;
σ'_v = vertical effective stress; and
ϕ_{tc} = coefficient of loading direction.

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