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

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Reference: Doan, L. V. and B. M. Lehane (2021). CPT-Based Design Method for Axial Capacities of Drilled Shafts and Auger Cast-in-Place Piles." Journal of Geotechnical and Geoenvironmental Engineering 147(8) https://doi.org/10.1061/(ASCE)GT.1943-5606.0002542

Summary:

To calculate axial capacities of drilled shafts (bored piles) and auger cast-in-place piles (continuous flight auger piles), the program uses the following expressions:

Shaft friction

$$q_s = (f_t/f_c)0.008\,(I_c)^{1.6}(q_t/p_a)^{0.8}p_a \;\; with \;\; p_a = 101\,kPa$$

End bearing

$$q_{b0.1} = 0.11\,I_c q_t$$

where q_s = unit shaft friction, f_t/f_c = loading direction parameter which is 1.0 for all cases except for tension piles in sand when a value of 0.8 is adopted, I_c = CPT soil behaviour type index, q_t = corrected cone tip resistance, $q_{b0.1}$ = end bearing resistance at a displacement of 10% of the pile diameter

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 (2012) method, which is based on a larger and independent data set.

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 in favour of CPT-based methods that have been shown to have much higher reliability (ISO-19901-4 2021, 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 ($q_{b0.1}$), i.e.:

$$q_s = q_t / \beta_c = \alpha_s q_t \qquad (1)$$

$$q_{b0.1} = \alpha_{b0.1} q_t \qquad (2)$$

LCPC methods

One of the first comprehensive direct CPT-based methods, which is known as the 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:

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

$$\text{where } q_{soil} = (aq_c + b)(1 - e^{-cq_c}) \tag{4}$$

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

where $\alpha_{pile\text{-}soil}$ is an empirical coefficient, which is related to the pile construction method and soil type through the a, b and c parameters. Equation 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 non-displacement piles are 250 in sand, 100 in sand-silt mixtures, 40 in silt and 20 in clay; see Figure. 1.

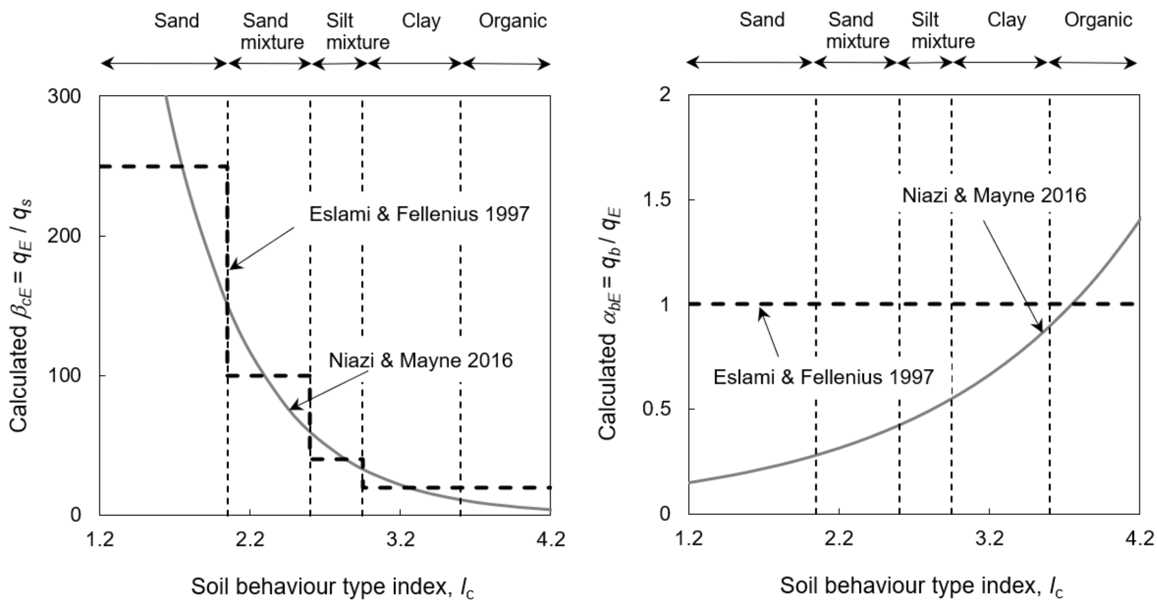

Figure 1. Proposed correlations for shaft and base capacities with the effective cone resistance

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 behaviour type index, I_c, where:

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

and Q_{tn} is the stress normalised q_t value and F_r is the normalised 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 on Figure 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 above, 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 non-displacement 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 0.004mm/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. 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 on Figure 2, and a mean diameter (D), length (L) and slenderness ratio (L/D) of about 0.6m, 12m and 20 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).

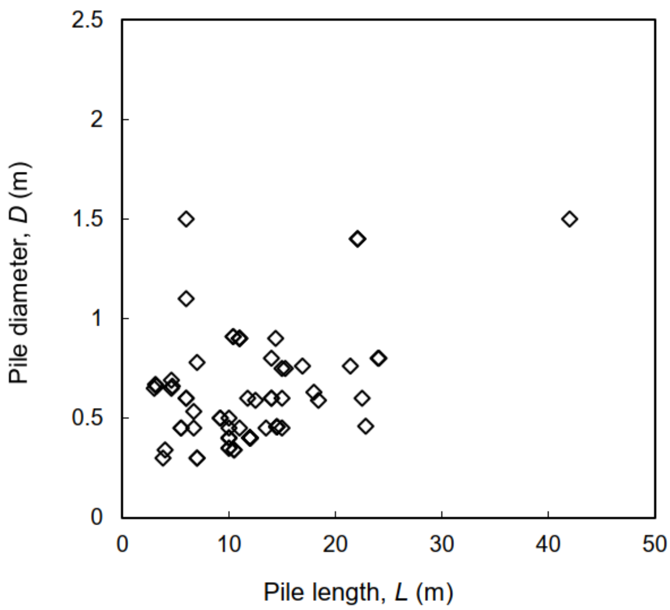

Figure 2. Pile dimensions in the database compiled for this study

CPT results including corrected end resistance (q_t), sleeve friction (f_s) and pore pressure (u_2) closest to each test pile were digitised 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 normalised 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 normalised cone 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 on Figures 3(a) and 3(b). These results were used to evaluate the soil behaviour 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 normalised 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).

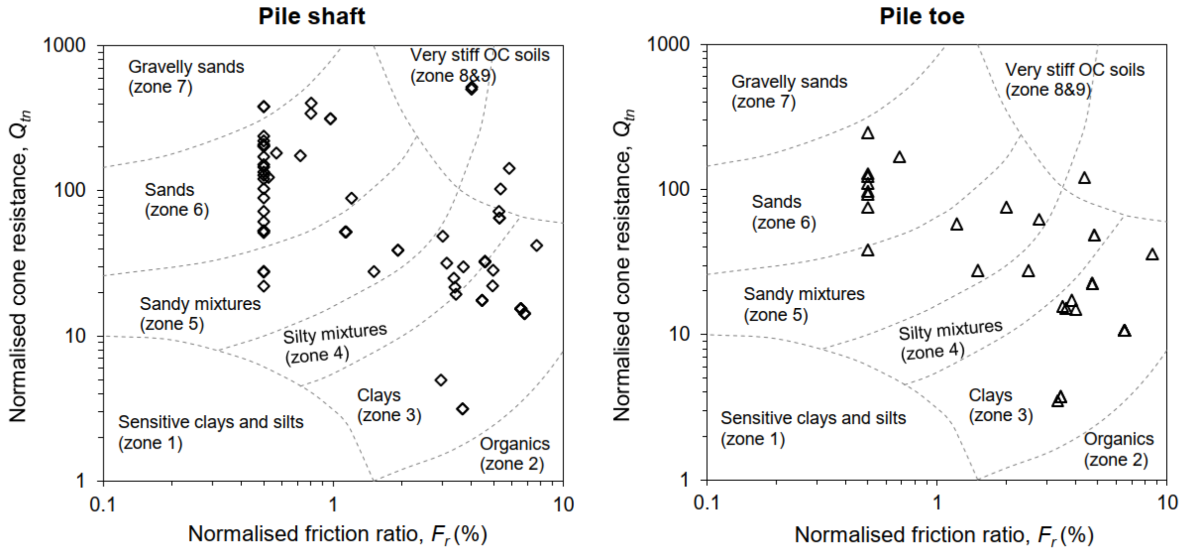

Figure 3. Soil behaviour types along the pile shafts and at the bases of the database piles

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 and 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, LCPC-2012, Eslami and Fellenius (1997), and Niazi and Mayne (2016); see Equations 3 to 10.

PERFORMANCE OF EXISTING METHODS

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

(i) *The* q_s values in materials with CPT end resistances in excess of about 2 MPa are over-estimated significantly by LCPC-1982; see Figure 4.

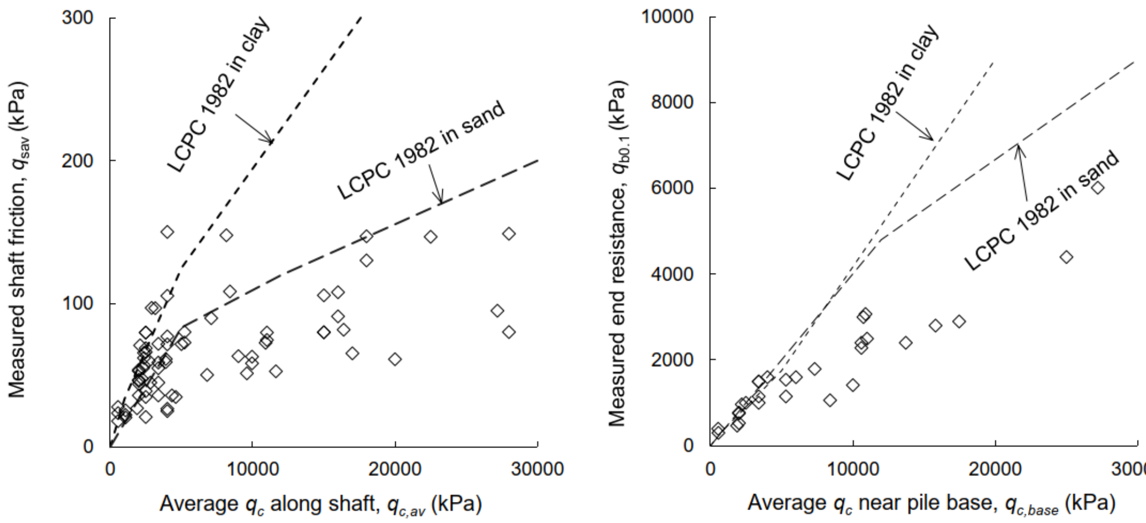

Figure 4. Comparison of measured shaft friction and base resistance with LCPC – 1982 method

(ii) 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 under-predict shaft friction in sands and silty sands; see Figure 5.

(iii) Base capacities tend to be over-predicted by all methods. The Eslami and Fellenius (1997) approach over-predicts significantly in all soil types while that of LCPC-1982 overestimates for $q_{c,base}$ in excess of about 5 MPa. The Niazi and Mayne (2016) base capacity predictions are reasonable in sands but over-estimate $q_{b0.1}$ values in silt and clay soils (I_c >2.2); see Figure 5.

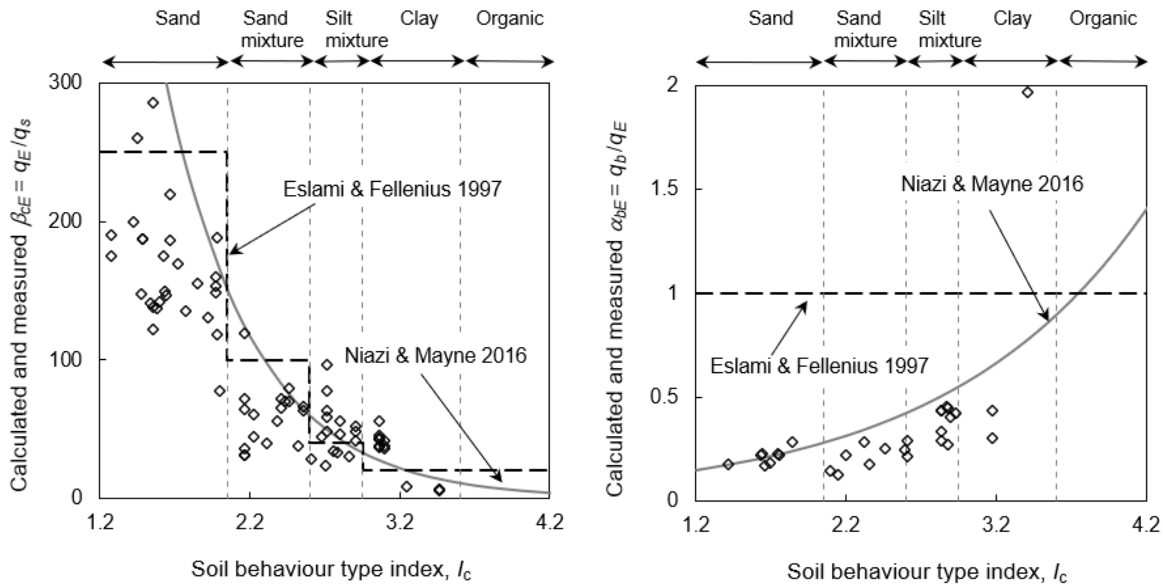

Figure 5. Relationship between the shaft and base resistances of the database piles with the effective cone resistance

(iv) As illustrated in Figure 6, the ratio of calculated to measured capacities for the methods are close to unity and therefore the significant under and over-predictions 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).

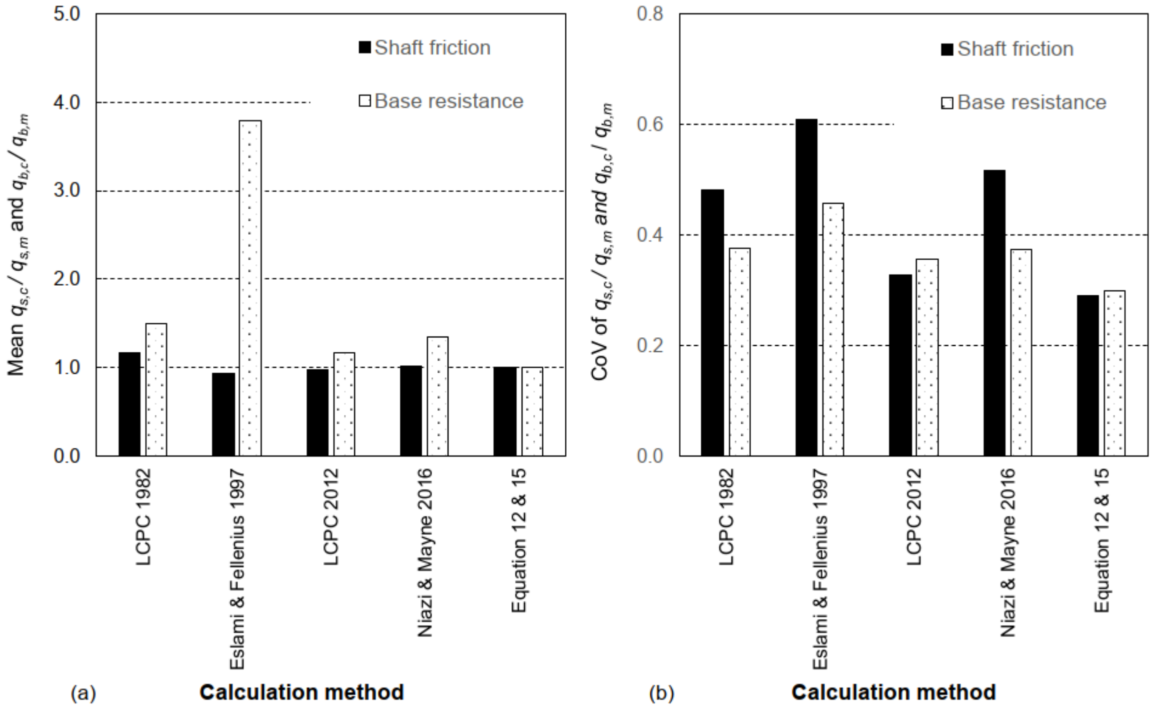

Figure 6. Performance of different calculation methods

Table 1. Database of pile load tests

No	Reference	Loading direction	Boring methods	Test Site	Soil description	D	L	Ls	Pile shaft				Pile toe			
									σ'_{vo}	q_t	F_r	I_c	σ'_{vo}	q_t	F_r	I_c
						(m)	(m)	(m)	(kPa)	(kPa)	(%)	(or Ic*)	(kPa)	(kPa)	(%)	
1	Carvalho & Albuquerque 2013	T	Dry	SP, BR	Sediment	0.35	10.0	3.5	57	1070	4.4	2.9				
	Carvalho & Albuquerque 2013	T	Dry	SP, BR	Residual clay	0.35	10.0	9.0	155	2360	6.8	3.1				
2	Carvalho & Albuquerque 2013	T	Dry	SP, BR	Sediment	0.40	10.0	3.5	57	1070	4.4	2.9				
	Carvalho & Albuquerque 2013	T	Dry	SP, BR	Residual clay	0.40	10.0	9.0	155	2360	6.8	3.1				
3	Carvalho & Albuquerque 2013	T	Dry	SP, BR	Sediment	0.40	10.0	3.5	57	1070	4.4	2.9				
	Carvalho & Albuquerque 2013	T	Dry	SP, BR	Residual clay	0.50	10.0	9.0	155	2360	6.8	3.1				
4	Albuquerque et al. 2011	C	Dry	SP, BR	Residual/Colluvial clay	0.40	12.0	2.5	34	2500	1.1	2.2				
	Albuquerque et al. 2011	C	Dry	SP, BR	Residual clay	0.40	12.0	8.5	122	2000	6.6	3.1	177	2060	6.5	3.2
5	Albuquerque et al. 2011	C	Dry	SP, BR	Residual/Colluvial clay	0.40	12.0	2.5	34	2500	1.1	2.2				
	Albuquerque et al. 2011	C	Dry	SP, BR	Residual clay	0.40	12.0	8.5	122	2000	6.6	3.1	177	2060	6.5	3.2
6	Albuquerque et al. 2011	C	Dry	SP, BR	Residual/Colluvial clay	0.40	12.0	2.5	34	2500	1.1	2.2				
	Albuquerque et al. 2011	C	Dry	SP, BR	Residual clay	0.40	12.0	8.5	122	2000	6.6	3.1	177	2060	6.5	3.2
7	Albuquerque et al. 2011	C	CFA	SP, BR	Residual/Colluvial clay	0.40	12.0	2.5	34	2500	1.1	2.2				
	Albuquerque et al. 2011	C	CFA	SP, BR	Residual clay	0.40	12.0	8.5	122	2000	6.6	3.1	177	2060	6.5	3.2
8	Albuquerque et al. 2011	C	CFA	SP, BR	Residual/Colluvial clay	0.40	12.0	2.5	34	2500	1.1	2.2				
	Albuquerque et al. 2011	C	CFA	SP, BR	Residual clay	0.40	12.0	8.5	122	2000	6.6	3.1	177	2060	6.5	3.2
9	Albuquerque et al. 2011	C	CFA	SP, BR	Residual/Colluvial clay	0.40	12.0	2.5	34	2500	1.1	2.2				
	Albuquerque et al. 2011	C	CFA	SP, BR	Residual clay	0.40	12.0	8.5	122	2000	6.6	3.1	177	2060	6.5	3.2
10	da Fonseca & Santos 2003	C	Casing	PO, PT	Residual silt	0.60	6.0	3.0	54	3899	5.3	2.6	108	5298	4.8	2.6
11	da Fonseca & Santos 2003	C	CFA	PO, PT	Residual silt	0.60	6.0	3.0	54	3899	5.3	2.6	108	5298	4.8	2.6
12	Zein & Ayoub 2012	C	Dry	KRT, SU	Clayey sand and stily clay	0.20	3.5	1.8	32	4200	5.6	2.4	63	4200	5.7	2.6
13	Mayne & Harris 1993	C	Dry	GA, USA	Residual silt & sand	0.76	16.9	8.5	137	5205	1.9	2.4	262	7320	2.5	2.6
14	Mayne & Harris 1993	C	Dry	GA, USA	Residual silt & sand	0.76	21.4	8.5	137	5205	1.9	2.4	283	31990	0.7	1.6
15	Park et at. 2011	C	CFA	KS, USA	Sand, silt, clay	0.46	22.9	11.4	206	27200	0.6	1.6	411	27200	2.0	2.2
16	Brown 2002	C	Casing	AL, USA	Residual clayey-silt	0.90	11.0	5.5	100	3364	4.6	2.7	141	3382	4.7	2.8

Table 1. (*Continued*)

17	Brown 2002	C	Bentonite	AL, USA	Residual clayey-silt	0.90	11.0	5.5	100	3364	4.6	2.7	141	3382	4.7	2.8
18	Brown 2002	C	Dry Polymer	AL, USA	Residual clayey-silt	0.90	11.0	5.5	100	3364	4.6	2.7	141	3382	4.7	2.8
19	Brown 2002	C	Liquid Polymer	AL, USA	Residual clayey-silt	0.90	11.0	5.5	100	3364	4.6	2.7	141	3382	4.7	2.8
20	Brown 2002	C	CFA	AL, USA	Residual clayey-silt	0.45	11.0	5.5	100	3364	4.6	2.7	141	3382	4.7	2.8
21	Briaud et al. 2000	C	Dry	TX, USA	Pleistocene stiff clay	0.91	10.4	5.2	92	4000	7.7	2.8	162	6009	8.6	2.9
22	Briaud et al. 2000	C	Bentonite	TX, USA	Pleistocene sand	0.91	10.4	5.2	90	8400	1.2	2.0	159	8406	1.2	2.1
23	Brown et al. 2006	C	Dry	GSY, UK	Stiff to firm Silty Clay	0.60	11.8	5.9	59	2120	3.1	2.6	118	2020	3.6	2.9
24	Elbanna et al. 2007	T	Dry	AB, Can	Silt and clayey Silt	1.40	22.1	12.3	158	4100	3.4	2.7				
	Elbanna et al. 2007	T	Dry	AB, Can	Silt and clayey Silt	1.40	22.1	14.8	178	4100	3.4	2.8				
	Elbanna et al. 2007	T	Dry	AB, Can	Silt and clayey Silt	1.40	22.1	17.0	196	4100	3.4	2.8	237	4100	3.5	2.9
25	Finno et al. 1989	C	Bentonite	IL, USA	Sand, silty sand	0.46	14.5	3.7	29	18007	1.0	1.6				
	Finno et al. 1989	C	Bentonite	IL, USA	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	IL, USA	Sand, silty sand	0.46	14.5	3.7	29	18007	1.0	1.6				
	Finno et al. 1989	C	Casing	IL, USA	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	MA, USA	Varved clay	0.9	14.4	11.5	92	656	2.9	3.2	115	656	3.3	3.4
28	O'Neill & Reese 1970	C	Dry	TX, USA	Beaumont clay	0.78	7.0	3.5	70	2240	3.7	2.7	120	2220	3.8	2.9
29	Doan & Lehane 2019	C	Dry	WA, AU	Silt mixtures, High OCR	0.30	3.8	1.9	48	8140	5.8	2.4	87	10890	4.4	2.3
30	Durham 2006	C	CFA	WA, AU	Sand and silt mixtures	0.30	7.0	3.5	38	10914	0.7	1.6				
31	Durham 2006	C	CFA	WA, AU	Sand and silt mixtures	0.30	7.0	6.0	58	1886	1.5	2.5	66	1886	1.5	2.5
32	Lehane 2009	C	CFA	WA, AU	Sand	0.23	4.0	2.0	36	9000	0.5	1.6				
33	Lehane 2009	C	CFA	WA, AU	Sand	0.34	4.0	2.0	36	9000	0.5	1.6				
34	Lehane 2009	C	CFA	WA, AU	Sand	0.34	10.5	5.3	62	4007	0.5	2.0				
35	Lehane 2009	C	CFA	WA, AU	Sand	0.34	10.5	5.3	62	4000	0.5	2.0				
36	Lehane 2009	C	CFA	WA, AU	Sand	0.45	5.5	2.8	50	15000	0.5	1.5				
37	Lehane 2009	C	CFA	WA, AU	Sand	0.45	5.5	2.8	50	15000	0.5	1.5				
38	Lehane 2009	C	CFA	WA, AU	Sand	0.45	6.7	3.4	60	6800	0.5	1.8				
39	Pine 2016	C	CFA	WA, AU	Sand	0.60	15.0	7.5	135	16000	0.5	1.6				
40	Tucker 1986	T	?	CA, USA	Silty sand	0.53	6.7	3.4	60	9600	0.5	1.7				

Table 1. (*Continued*)

41	Franke & Garbrecht 1977	C	Bentonite	GE	Silty sand	1.10	6.0	3.0	54	17000	0.5	1.5				
42	Franke & Garbrecht 1977	C	Bentonite	GE	Silty sand	1.50	6.0	3.0	54	16000	0.5	1.5				
43	Konstantinidis et al. 1987	T	?	CA, USA	Silty sand and gravel	0.69	4.6	2.3	41	28000	0.80	1.4				
44	Konstantinidis et al. 1987	T	?	CA, USA	Silty sand and gravel	0.67	3.1	1.6	28	20000	0.80	1.5				
45	Konstantinidis et al. 1987	T	?	NV, USA	Silty clay	0.65	4.6	2.3	39	28000	4.00	2.0				
46	Konstantinidis et al. 1987	T	?	NV, USA	Silty clay	0.65	3.0	1.5	26	22500	4.00	2.0				
47	Konstantinidis et al. 1987	T	?	UT, USA	Silty clay	0.66	3.2	1.6	27	2500	5.3	2.5				
48	Konstantinidis et al. 1987	T	?	UT, USA	Silty clay	0.66	4.7	2.4	40	5000	5.3	2.4				
49	Kruizinga 1975	C	Bentonite	BC, NL	Sand	0.63	18.0	9.0	72	11018	0.5	1.6	144	11018	0.5	1.6
50	Kruizinga 1975	C	Bentonite	SLT, NL	Sand	0.59	12.5	6.3	50	11013	0.5	1.6	100	25025	0.5	1.4
51	Kruizinga 1975	C	Bentonite	SLT, NL	Sand	0.59	18.4	9.2	74	15018	0.5	1.5	147	2537	4.0	2.9
52	Caputo & Viggiani 1988	C	Bentonite	NAP, ITL	Silty sand	1.50	42.0	21.0	157	9998	0.5	1.9	304	10082	0.5	2.1
53	Mandolini et al. 2002	C	CFA	NAP, ITL	Silty sand	0.80	24.0	12.0	94	2835	0.5	2.2	178	15865	0.5	1.7
54	Mandolini et al. 2002	C	CFA	NAP, ITL	Silty sand	0.60	22.5	11.3	89	2748	0.5	2.2	168	10796	0.5	1.8
55	Mandolini et al. 2002	C	CFA	NAP, ITL	Silty sand	0.80	24.1	12.1	94	2314	0.5	2.3	179	17516	0.5	1.7
56	Cadogan and Gavin 2006	C	Dry	DUB, IE	Overconsolidated sand	0.10	2.0	1.0	20	17000	2.0	1.8	40	17000	2.0	1.8
57	Cadogan et al. 2010	C	Dry	DUB, IE	Overconsolidated sand	0.10	2.0	1.0	20	15000	2.0	1.8	40	15000	2.0	1.9
58	Cadogan et al. 2010	C	Dry	DUB, IE	Overconsolidated sand	0.20	3.0	1.5	30	15000	2.0	1.8	60	15765	2.0	1.9
59	Gavin et al. 2009	C	CFA	CK, IE	Loose and dense sand	0.80	14.0	7.5	60	4626	0.5	1.9	112	10628	0.5	1.7
60	Gavin et al. 2009	C	CFA	CK, IE	Loose and dense sand	0.45	15.0	9.0	72	4344	0.5	2.0	120	10630	0.5	1.8
61	Current study	C	Dry	WA, AU	Silt mixtures, High OCR	0.75	15.0	7.5	150	7073	3.0	2.5				
62	Current study	C	CFA	SA, AU	Silt	0.75	15.3	6.0	108	3170	5.0	2.8		3170		
63	Current study	C	CFA	WA, AU	Sand	0.60	14.0	7.0	91	11637	0.5	1.7				
64	Current study	C	CFA	WA, AU	Sand	0.60	14.0	7.0	91	9957	0.5	1.7				
65	Current study	C	CFA	WA, AU	Sand	0.50	9.2	4.6	83	35660	0.5	1.3				
66	Current study	C	CFA	WA, AU	Sand	0.50	9.2	4.6	83	35660	0.5	1.3				
67	Current study	C	CFA	WA, AU	Sand	0.45	10.0	5.0	40	16400	0.5	1.4				
68	Current study	C	CFA	SA, AU	Silty clay/clays	0.45	13.5	7.0	126	2913	4.9	2.9	243	13721	2.8	2.4

Table 2. Prediction for Database Piles

No	Reference	Test Site	Measured		LCPC-82		LCPC-2012		Eslami & Fellenius 1997		Niazi & 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 & Albuquerque 2013	SP, BR	26		0.7		0.8		1.0		1.2	
	Carvalho & Albuquerque 2013	SP, BR	57		1.0		0.9		2.1		1.6	
2	Carvalho & Albuquerque 2013	SP, BR	22		0.8		0.9		1.2		1.3	
	Carvalho & Albuquerque 2013	SP, BR	66		0.9		0.8		1.8		1.4	
3	Carvalho & Albuquerque 2013	SP, BR	21		0.9		1.0		1.3		1.5	
	Carvalho & Albuquerque 2013	SP, BR	62		0.9		0.9		1.9		1.5	
4	Albuquerque et al. 2011	SP, BR	39		1.1		1.1		0.6		0.7	
	Albuquerque et al. 2011	SP, BR	44		1.1		1.1		2.3		2.2	
5	Albuquerque et al. 2011	SP, BR	21		3.0		2.0		1.2		1.3	
	Albuquerque et al. 2011	SP, BR	54		0.9		0.9		1.9		1.8	
6	Albuquerque et al. 2011	SP, BR	35		1.2		1.2		0.7		0.8	
	Albuquerque et al. 2011	SP, BR	46		1.1		1.1		2.2		2.1	
7	Albuquerque et al. 2011	SP, BR	80		0.5		0.5		0.3		0.3	
	Albuquerque et al. 2011	SP, BR	47	760	1.1	0.9	1.1	1.2	2.1	2.3	2.0	1.5
8	Albuquerque et al. 2011	SP, BR	80		0.5		0.5		0.3		0.3	
	Albuquerque et al. 2011	SP, BR	53	530	0.9	1.4	1.0	1.7	1.9	3.3	1.8	2.2
9	Albuquerque et al. 2011	SP, BR	69		0.6		0.6		0.4		0.4	
	Albuquerque et al. 2011	SP, BR	36		1.4		1.4		2.8		2.7	
10	da Fonseca & Santos 2003	PO, PT	62	1149	1.1	2.1	0.9	1.4	0.6	4.6	1.3	2.0
11	da Fonseca & Santos 2003	PO, PT	59	1538	1.1	1.5	1.0	1.0	0.7	3.5	1.3	1.5
12	Zein & Ayoub 2012	KRT, SU	72	796	1.0	2.1	0.8	1.6	0.6	5.3	1.0	2.2
13	Mayne & Harris 1993	GA, USA	80	1792	0.6	1.8	0.8	1.2	0.7	4.1	1.0	1.7
14	Mayne & Harris 1993	GA, USA	73	7000	0.7	1.4	0.9	1.4	0.7	4.6	1.1	0.9
15	Park et at. 2011	KS, USA	95	6017	1.9	1.4	1.2	1.1	1.1	4.5	1.1	1.4
16	Brown 2002	AL, USA	55	1500	1.0	1.0	0.9	0.7	1.6	2.3	1.7	1.2

Table 2. (*Continued*)

17	Brown 2002	AL, USA	36	1000	1.6	1.5	1.4	1.0	2.4	3.5	2.6	1.8
18	Brown 2002	AL, USA	59	1150	1.0	1.3	0.9	0.9	1.5	3.0	1.6	1.5
19	Brown 2002	AL, USA	72	1500	0.8	1.0	0.7	0.7	1.2	2.3	1.3	1.2
20	Brown 2002	AL, USA	45		1.2		1.2		1.9		2.1	
21	Briaud et al. 2000	TX, USA	72	1598	0.9	1.5	0.8	1.1	1.4	3.7	1.7	2.0
22	Briaud et al. 2000	TX, USA	109	1065	0.8	3.2	0.6	2.4	0.3	7.9	0.6	2.4
23	Brown et al. 2006	GSY, UK	71	776	0.5	0.9	0.5	1.0	0.7	2.5	0.6	1.3
24	Elbanna et al. 2007	AB, Can	150		0.5		0.4		0.6		0.5	
	Elbanna et al. 2007	AB, Can	106		0.6		0.6		0.9		0.8	
	Elbanna et al. 2007	AB, Can	77	1600	0.9	1.0	0.8	0.8	1.2	2.3	1.1	1.2
25	Finno et al. 1989	IL, USA	147		0.8		0.7		0.5		0.5	
	Finno et al. 1989	IL, USA	24	402	0.9	0.7	0.8	0.7	0.3	0.3	0.6	0.2
26	Finno et al. 1989	IL, USA	130		0.9		0.7		0.6		0.5	
	Finno et al. 1989	IL, USA	28		0.8		0.7		0.3		0.5	
27	Iskander et al. 2003	MA, USA	18	306	1.2	0.9	1.1	0.9	0.4	0.5	0.6	0.4
28	O'Neil & Reese 1970	TX, USA	48	967	0.8	0.9	0.8	0.7	1.1	2.2	1.1	1.1
29	Doan & Lehane 2019	WA, AU	148	3071	0.6	1.4	0.6	1.1	0.6	3.6	0.8	1.2
30	Durham 2006	WA, AU	73		2.5		1.1		0.6		0.6	
31	Durham 2006	WA, AU	27	472	1.2		1.3		0.7		1.2	
32	Lehane 2009	WA, AU	96		0.9		0.8		0.4		0.4	
33	Lehane 2009	WA, AU	63		1.4		1.2		0.6		0.6	33
34	Lehane 2009	WA, AU	27		2.5		1.6		0.6		1.1	34
35	Lehane 2009	WA, AU	25		2.7		1.7		0.6		1.2	35
36	Lehane 2009	WA, AU	80		1.2		1.2		0.7		0.6	36
37	Lehane 2009	WA, AU	80		1.2		1.2		0.7		0.6	37
38	Lehane 2009	WA, AU	50		1.4		1.2		0.5		0.7	38
39	Pine 2016	WA, AU	91		1.2		1.1		0.7		0.7	39
40	Tucker 1986	CA, USA	52		1.9		1.4		0.7		0.7	40

Table 2. (*Continued*)

41	Franke & Garbrecht 1977	GE	65		1.7		1.4		1.0		0.8	
42	Franke & Garbrecht 1977	GE	108		1.0		0.9		0.6		0.5	
43	Konstantinidis et al. 1987	CA, USA	80		2.3		1.4		1.4		0.8	
44	Konstantinidis et al. 1987	CA, USA	61		2.2		1.6		1.3		0.8	
45	Konstantinidis et al. 1987	NV, USA	149		1.3		0.8		0.8		1.1	
46	Konstantinidis et al. 1987	NV, USA	147		1.0		0.7		0.6		0.9	
47	Konstantinidis et al. 1987	UT, USA	66		0.4		0.6		0.4		0.6	
48	Konstantinidis et al. 1987	UT, USA	72		0.7		0.9		0.7		0.9	
49	Kruizinga 1975	BC, NL	75	2500	1.5	1.8	1.0	0.9	0.6	4.4	0.6	0.9
50	Kruizinga 1975	SLT, NL	80	4400	1.4	1.7	1.0	1.1	0.5	5.7	0.5	1.0
51	Kruizinga 1975	SLT, NL	106	1000	0.9	0.9	0.8	1.0	0.6	2.4	0.5	1.3
52	Caputo & Viggiani 1988	NAP, ITL	63	1415	1.6	2.9	1.2	2.1	0.6	6.8	1.0	2.0
53	Mandolini et al. 2002	NAP, ITL	45	2800	1.0	1.7	1.1	1.4	0.6	5.6	0.7	1.2
54	Mandolini et al. 2002	NAP, ITL	60	3000	0.8	1.4	0.8	0.9	0.4	3.5	0.5	0.8
55	Mandolini et al. 2002	NAP, ITL	55	2900	0.7	1.8	0.7	1.5	0.4	6.0	0.5	1.2
56	Cadogan and Gavin 2006	DUB, IE	143	4500	0.8	1.1	0.7	0.8	0.5	3.8	0.6	0.9
57	Cadogan et al. 2010	DUB, IE	120	3500	0.8	1.3	0.7	0.9	0.5	4.3	0.7	1.0
58	Cadogan et al. 2010	DUB, IE	120	4090	0.8	1.2	0.7	0.8	0.5	3.9	0.7	1.0
59	Gavin et al. 2009	CK, IE	35	2400	2.2	1.8	1.4	1.1	0.5	4.4	0.9	1.0
60	Gavin et al. 2009	CK, IE	36	2280	2.0	1.9	1.3	1.2	0.5	4.6	0.9	1.0
61	Current study	WA, AU	90		0.8		0.9		0.8		1.4	
62	Current study	SA, AU	97		0.5		0.5		0.8		1.0	
63	Current study	WA, AU	53		2.2		1.6		0.9		1.0	
64	Current study	WA, AU	59		1.7		1.3		0.7		0.8	
65	Current study	WA, AU	204		1.2		0.6		0.7		0.4	
66	Current study	WA, AU	188		1.3		0.7		0.8		0.5	
67	Current study	WA, AU	82		1.3		1.2		0.8		0.6	
68	Current study	SA, AU	97	2400	0.5	1.7	0.5	1.7	0.8	5.7	1.0	2.0

The mean (μ) and *CoV* for ratios of calculated to measured shaft frictions are summarised 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 2 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 2 times the actual capacity is less than 1/750 (again assuming a normal distribution).

Table 3. Calculated to measured shaft frictions and base resistances

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

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

CONSIDERATIONS FOR FORMULATION DEVELOPMENT

Doan and Lehane (2020) examine 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) propose use of the I_c index to reflect these additional parameters and provide evidence in support of a correlation between q_s, q_t and I_c. Such a correlation is examined in the following while also allowing for the potential non-linear relationship between q_s and q_t such as indicated in Figure 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 equations (6) to (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. The measurement of u_2 is, however, 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 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 (non-linear) shear stiffness and *D* is the pile diameter. As a consequence, the relative 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 on Figure 7a and Figure 8a 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, Figure 7b and Figure 8b, 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 observations of Doan and Lehane (2020) from pile tests conducted in pressure chambers and direct shear tests who show that the dilation effects are very significant in clean sands but are negligible in clayey sands with a clay fraction exceeding about 4% .

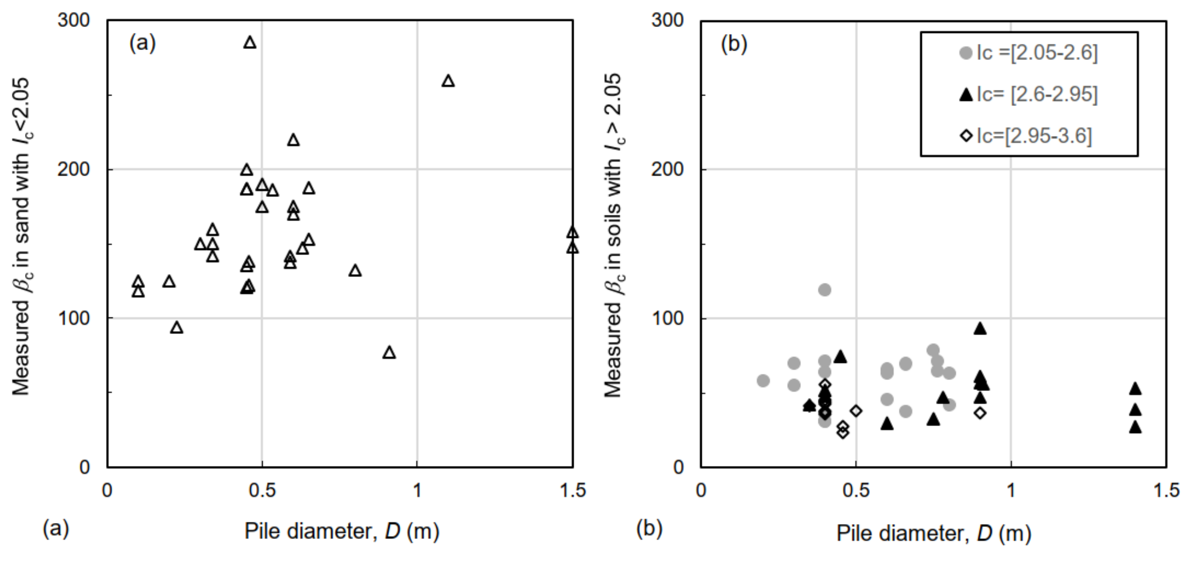

Figure 7. Effect of pile diameter on the measured β_c values

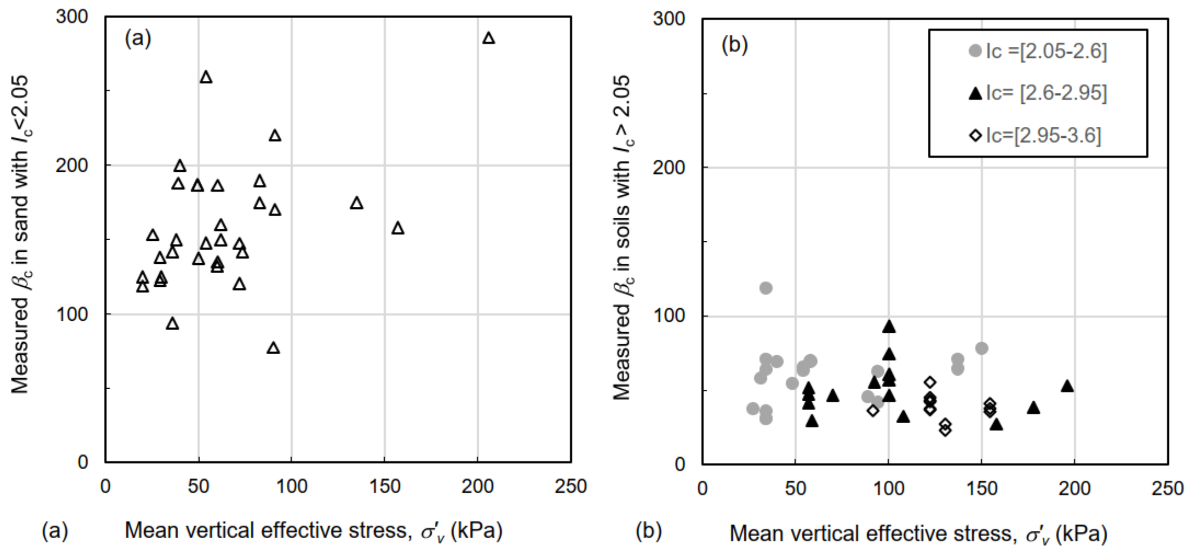

Figure 8. Effects of stress level on the measured β_c values

It follows from the foregoing that a relationship between shaft friction and CPT end resistance in sands needs to allow for the 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 which is considered representative of typical drilled shafts. This was achieved by removing piles with diameters less than 300mm and piles less than 3m in length from the regression analysis. Lehane et al. (2020) show that, for driven piles, the contribution of dilation for piles with diameters between 300mm and 1m 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 as the vertical effective stress which is used to determine Q_{tn} is unknown.

An alternative index, referred to here as I_c* is employed in the following when S_r<1 and is evaluated using equation (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 on Figure 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 below do not include the data points obtained in the partially saturated deposits.

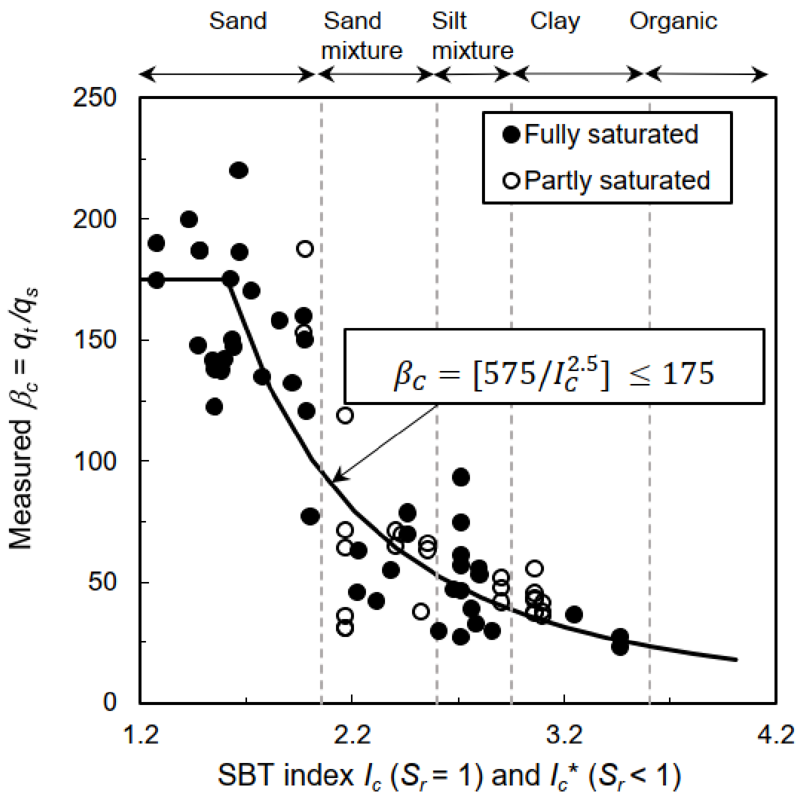

Figure 9. Comparison of β_c values in partly and fully saturated soils

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

(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 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:

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

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:

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

Observations from the analyses are summarised below and in Figures 10 to 12.

(i) A second expression provided a comparable fit to the database to equation (12) and is the best-fit regression line when β_c is expressed as a unique function of I_c (Figure 9):

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

Equation (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 equation (13) to provide a better fit to the shaft frictions measured in sands (see Figure 9).

(ii) Virtually identical mean and *CoV* values were obtained using I_c and I_c* in Equations 12 to 13 suggesting that the relationships are also applicable in partially saturated deposits.

(iii) 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.

(iv) The best-fit relationship obtained using the effective cone resistance (q_E) was as follows:

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

(v) Closer inspection of the effect of soil type indicated that *CoVs* for q_{sc}/q_{sm} in the clays was lower than in the sands and silts; see Figure 10. This observation is consistent with observations made by Alsamman (1995), Phoon and Kulhawy (2005); Reddy and Stuedlein (2017) and others.

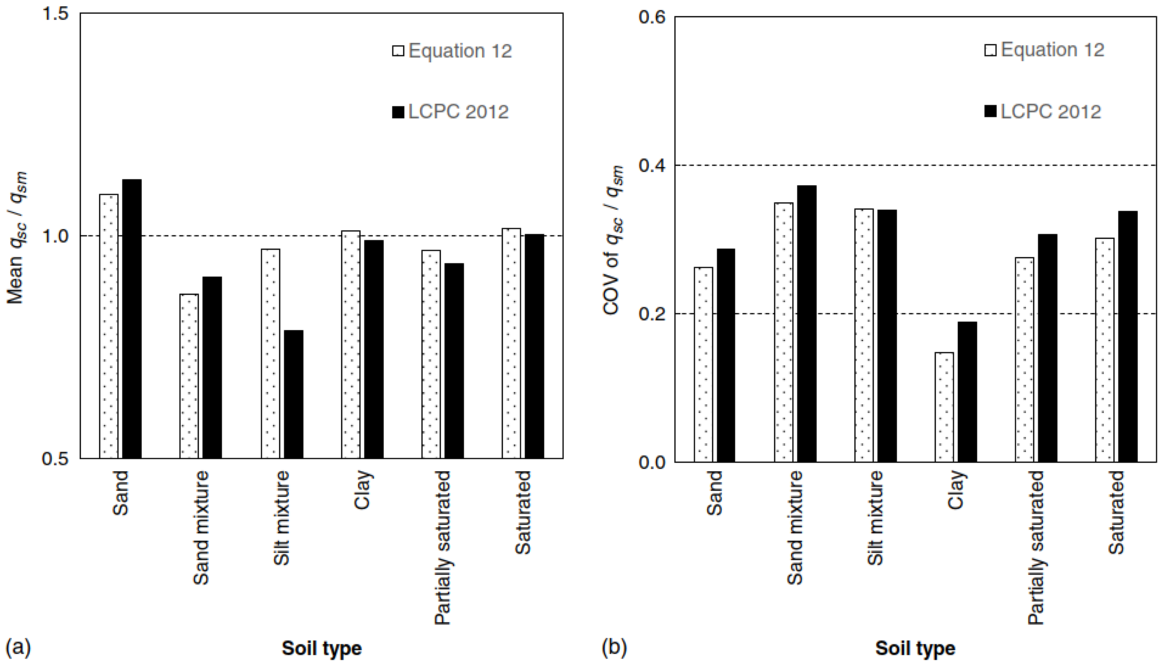

Figure 10. Performance of Equation 12 and LCPC-2012 in different soil types

(vi) An examination of the effect of the boring technique on Figure 11 shows that, on average, the ratios of calculated to measured shaft friction of the database piles using equation 12 are approximately unity for all boring techniques; a similar trend is shown by Equation 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. However, Brown (2002) did note, however, that bores stabilised with polymer lead to greater friction that those supported by bentonite in Piedmont residual soil.

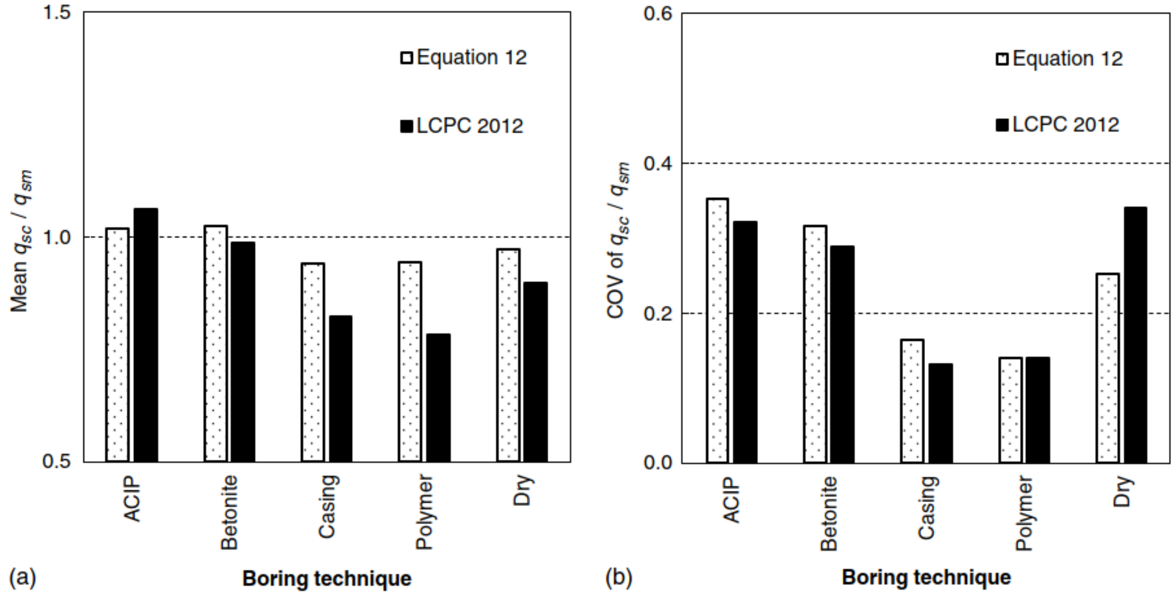

Figure 11. Performance of Equation 12 and LCPC-2012 for piles with different boring techniques

(vii) The relatively good predictive performance of Equation 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 Figure 12 which plots the ratios of shaft frictions calculated using Equation 12 to those calculated using the LCPC-2012 for a database of CPTs available to the authors from Australia and New Zealand. The deviation in the plotted ratio about unity is small indicating good agreement of the two formulations.

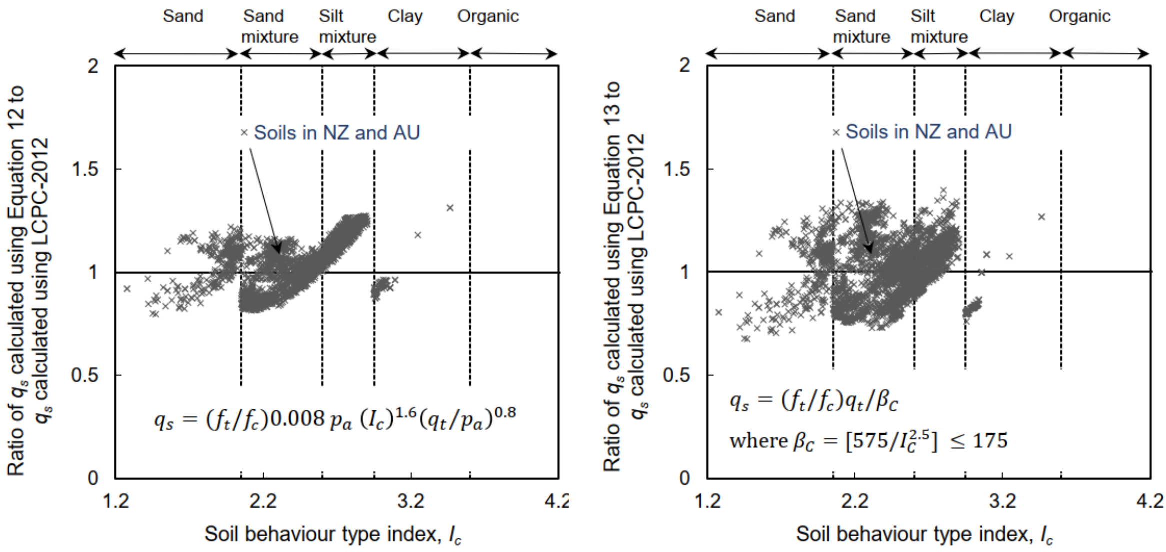

Figure 12. Comparison between Equations 12-13 and LCPC-2012 using NZ and AU soil data

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

The ability of Equation 12 to predict skin friction distributions is examined by comparing measured and calculated shaft frictions ($q_{s,m}$ 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 Figure 13a while Figure 13b plots available localised $q_{s,c}/q_{s,m}$ ratios against depth. Figure 13b indicates a spread in ratios in line with the overall CoV of Equation 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 localised shaft friction for drilled shafts. It can be shown that Equation 13 gives closely comparable trends to those indicated for Equation 12 on Figure 13.

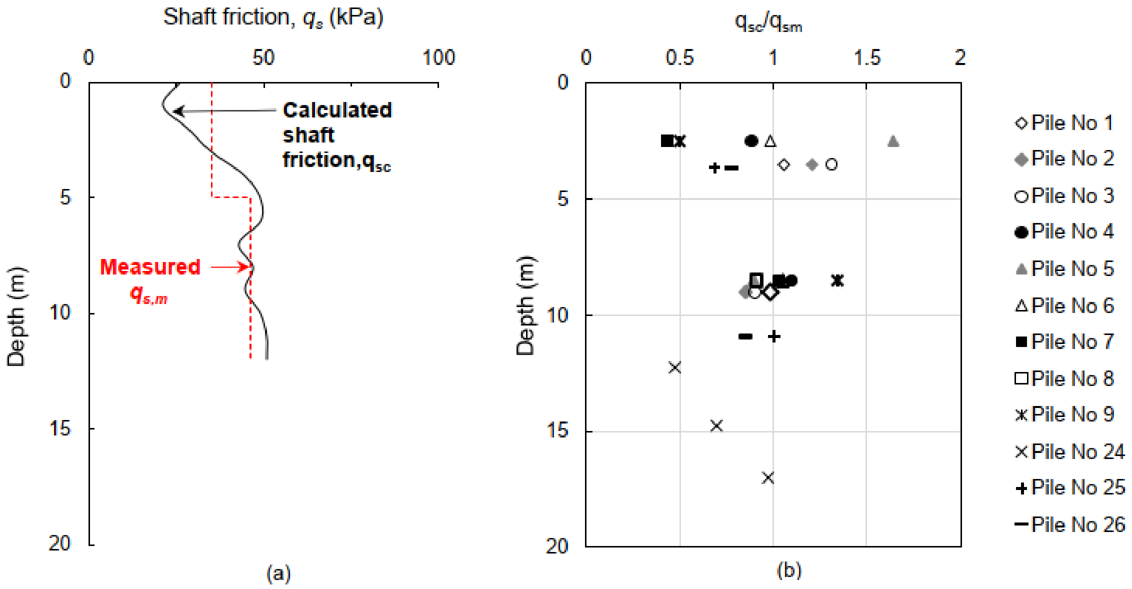

Figure 13. (a) Comparison of measured shaft friction with friction calculated using Equation 12; Pile No. 6 and (b) Variation with depth of ratios of calculated to measured localised shaft friction for selected pile tests; pile test numbers. correspond with those in Table 1

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 on Figure 5. The best fit simple correlation with I_c achieved for the database is as follows and compared with the database measurements on Figure 14.

$$q_{b0.1} = \alpha_{b0.1} q_t = 0.11\, I_c q_t \qquad 1.5 \leq I_c \leq 3.6 \qquad CoV = 0.30 \tag{15}$$

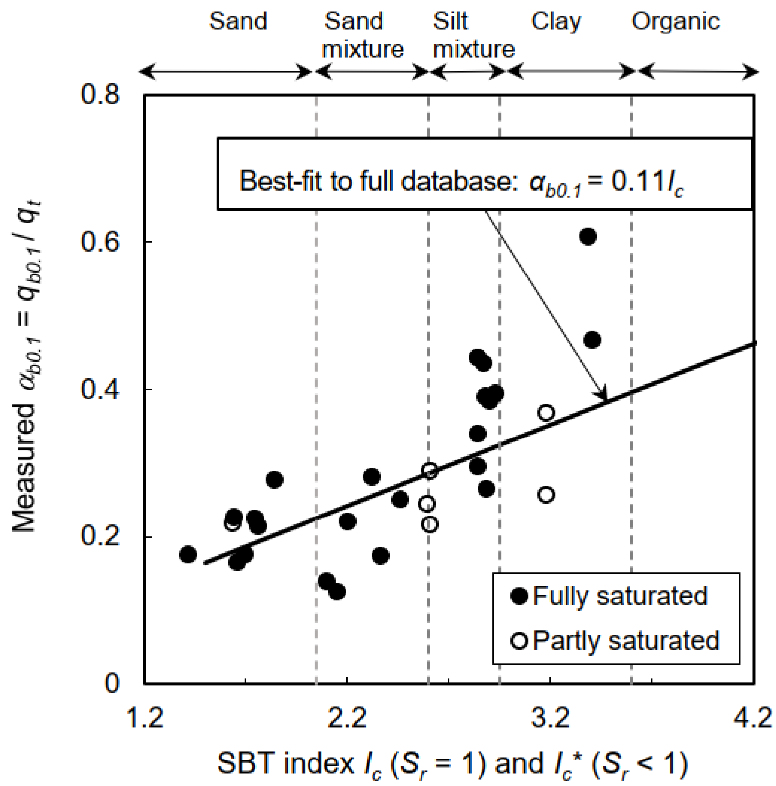

Figure 14. Database values of $\alpha_{b0.01}$ plotted as a function of soil behaviour types

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 (see Table 3). The expression is consistent with analyses such as those of Lee and Salgado (1999) who show $\alpha_{b0.1}$ (=$q_{b0.1}/q_t$) reduces with increasing soil stiffness and hence reducing I_c value. Drainage effects also clearly affect the $\alpha_{b0.1}$ ratio, noting that 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 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 behaviour type index (I_c), as seen in experimental studies, and the non-linear 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 (CoV) 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.

ACKNOWLEDGEMENTS

The first author acknowledges the Australian Department of Foreign Affairs and Trade for financial support through AAS scholarships. The authors also acknowledge the load test data and support provided by Belpile Pty Ltd to the research project.

NOTATION

The following symbols are used in this paper:

D	*pile diameter*
F_r	CPT normalised friction ratio
f_s	CPT sleeve friction
f_t/f_c	loading direction parameter
G	operational (non-linear) shear stiffness
I_c	CPT soil behaviour type index
L	pile length
L_s	length of a segment of the pile shaft
Q_{tn}	CPT normalised 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 $q_{b0.1}/q_t$
$\alpha_{pile\text{-}soil}$	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
ϕ_{tc}	coefficient of loading direction

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