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# Driven cast-in-situ piles in granular soil: applicability of CPT methods for pile capacity

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**ABSTRACT:** Most CPT-based design methods for driven piles are heavily influenced by research on preformed driven piles. In this paper, a critical review is made of the ability of these methods to predict the total capacity of driven cast-in-situ (DCIS) piles. To this end, a DCIS database of 16 piles (with adjacent CPT profiles) is developed which incorporates new DCIS load-test data from UK sites. Initially, instrumented piles from the database are used to illustrate that DCIS and preformed piles have comparable shaft and base resistance characteristics. The database is then used to study of the performance of three simplified and four advanced CPT-based displacement pile design methods in estimating the total capacity of DCIS piles. Statistical analyses show that in general, the advanced CPT methods provide the best estimates of DCIS total capacity, although capacities tend to be under predicted by most methods. This tendency may be attributed to enhanced interface friction between the cast-in-situ concrete and the soil compared to that developed with preformed piles.

## 1 INTRODUCTION

The prediction of displacement pile capacity in sand is hampered by the extreme changes in stress which occur in the immediate vicinity of the pile during installation (Randolph, 2003). Furthermore, the absence of high-quality undisturbed sand samples in routine foundation projects has led to a heavy reliance on empirical design methods which correlate pile capacity directly to the results of in-situ tests such as the Cone Penetration Test (CPT). As the penetration of the cone is analogous to the installation of a displacement pile, a significant number of CPT-based design methods which derive shaft and base capacities from the in-situ cone resistance  $q_c$  profile using simplified coefficients have been developed (Lehane, 2009), such as the LCPC-82 (Bustamante & Gianeselli, 1982), EF-97 (Eslami & Fellenius, 1997) and Van Impe-86 (Van Impe, 1986) methods.

High quality instrumented tests on steel model piles, such as those reported by Lehane (1992) and Chow (1997), have helped identify other factors that have a bearing on displacement pile behavior in sand. These factors include the extent of soil displacement during installation and loading, the reduction in shaft friction due to increasing load cycles during installation (referred to as friction fatigue), increases in radial stresses due to dilation at the pile-soil interface, differences in shaft resistance with loading direction (i.e. compressive and tensile loading) and increases in shaft capacity with time, i.e. pile aging. The majority of these phenomena have now been incorporated in four new advanced CPT-based design methods – Fugro-05 (Kolk et al. 2005), ICP-05 (Jardine et al. 2005), NGI-05 (Clausen et al. 2005) and UWA-05 (Lehane et al. 2005). Schneider et al. (2008) evaluated the relative merits of these methods in predicting displacement pile capacity and showed that the UWA-05 method fared best.

Unlike preformed displacement piles, the driven cast-in-situ (DCIS) category of pile has not received exclusive attention from those developing CPT-based methods. This may be due in part to important dif-

ferences between the installation processes for DCIS and traditional displacement piles. This paper presents a brief summary of the results of a series of instrumented DCIS pile tests (including some new data from UK sites) which demonstrate similarities in axial behavior between DCIS and preformed displacement piles in terms of shaft and base resistances. A database of 16 DCIS piles with adjacent CPT data at 8 sites is then presented which is used to study the predictive performance of the three simplified and four advanced CPT-based displacement design methods in estimating the total capacity of DCIS piles.

## 2 DRIVEN CAST-IN-SITU PILES

The type of DCIS pile installation considered here begins by driving a hollow steel tube using a pile driving hammer (Fig. 1a). In order to prevent ingress of soil and water during driving, the tube is fitted with a sacrificial steel circular driving shoe which has a slightly larger diameter than that of the tube. A schematic of the remainder of the process is shown in Figure 1b. When the required depth is reached a high slump concrete is introduced into the tube; reinforcement may be placed either before or after concreting. The tube is then slowly extracted, with the hammer providing additional blows to compact the concrete during withdrawal. The concrete is then left to cure *in-situ*. A particular version of DCIS pile, known as a Franki pile, is driven internally using a mandrel, with dry batches of concrete placed at the base of the tube prior to installation to enable a plug to form during driving. The plug is hammered out through the base of the tube at design depth and additional batches of zero-slump concrete are added to form the expanded base (Neely, 1990).

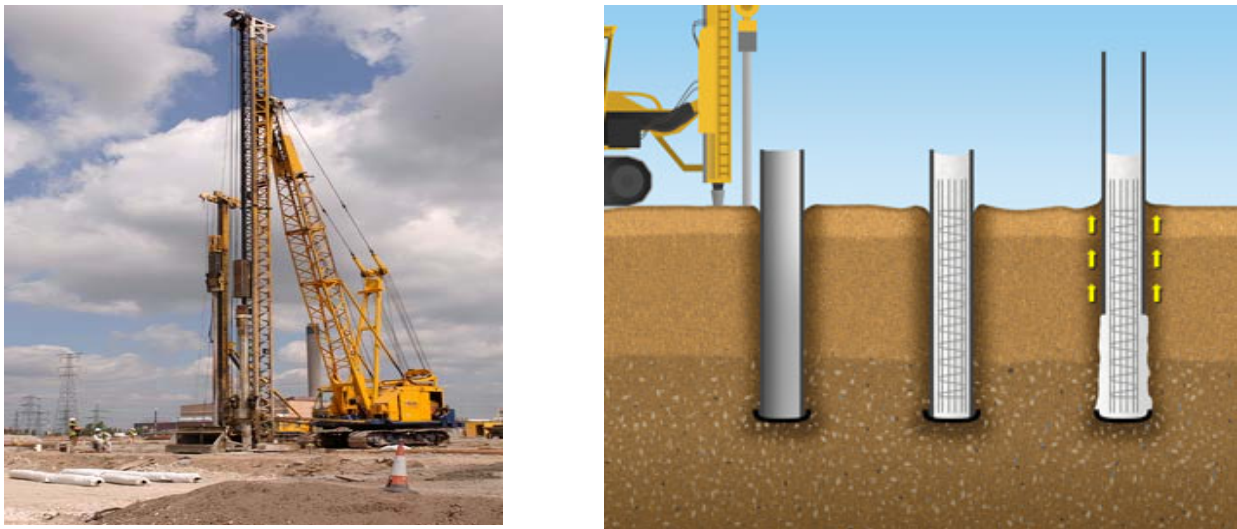


Figure 1(a) Installation of hollow steel tube and (b) Schematic of driven cast-in-situ pile installation (images courtesy of Keller Foundations, UK)

The base resistance of DCIS piles is considered to be no worse than that of preformed displacement piles. For example, the results of the DCIS tests at Kallo, Belgium by De Beer et al. (1979) were included in the database of 20 piles used to formulate the base resistance equation for closed-ended displacement piles for the UWA-05 method (Xu et al. 2008b). The unit shaft resistance ( $q_s$ ) of displacement piles can be calculated from  $q_s = K_s \sigma'_v \tan \delta$ , where  $K_s$  is a coefficient of earth pressure,  $\sigma'_v$  is the vertical effective stress and  $\delta$  is the angle of interface friction. Fleming et al. (2008) suggest that the extraction of the driving tube will lead to a reduced  $K_s$  value (assuming wet concrete is placed). However, it is the product  $K_s \tan \delta$  that controls  $q_s$  and there is no evidence to indicate that  $K_s \tan \delta$  for DCIS piles falls below that of preformed piles. However, any increase in the mobilized friction angle will compensate for

reduced  $K_s$  values: this has not been investigated to date. The LCPC-82 method pessimistically recommends coefficients for estimating the shaft capacity of DCIS piles which are only 50% of those recommended for preformed driven piles.

Although DCIS piles have been used extensively in many countries throughout the world over the past 60 years or so, the number of published case histories of static load tests is surprisingly limited. Moreover, the majority of published data involve non-instrumented piles, precluding accurate assessments of shaft and base behavior during loading which in turn inhibits verification of design methods.

### 3 DCIS PILE DATABASE

In an attempt to gain a better insight into the axial load behavior of DCIS piles, a research project has been undertaken at the National University of Ireland, Galway (NUIG), in collaboration with Keller Foundations UK, in which several instrumented DCIS piles were installed at UK sites and subjected to maintained-load tests. A full description of each test is beyond the scope of this paper; readers are referred to Flynn et al. (2012, 2013) and Flynn (2014) for further details. These test data have been supplemented with other published DCIS pile test data from the literature to form a new database; the subset of the database for which nearby CPT profiles are available is presented in Table 1. The pile capacities quoted are the loads corresponding to a displacement of  $0.1D_b$  where  $D_b$  is the pile base diameter. Given that base resistances require much greater displacements than shaft resistances to be mobilized and some of the piles in the database have  $D_b$  values significantly greater than their pile shaft diameters ( $D_s$ ), comparison of loads at  $0.1D_b$  is more appropriate than at  $0.1D_s$  because shaft and base resistances will be significantly mobilized at this displacement.

The aforementioned traditional driven pile design methods are appraised against this DCIS pile database in Section 5. However, to provide some justification for making this appraisal, the shaft and base resistance behavior of the instrumented piles in Table 1 are considered in more detail in Section 4.

Site; Pile Ref	Pile Type	Shaft Diameter $D_s$ (mm)	Base Diameter $D_b$ (mm)	Length $L$ (m)	Pile Capacity $Q_{0.1D_b,m}$ (kN)	Instrumented	Reference
Pontarddulais; P1	DCIS	340	380	8.50	949	✓	Flynn (2014)
Shotton; S1	DCIS	340	380	5.75	2214	✓	Flynn et al. (2013)
Dagenham; D1	DCIS	340	380	7.70	2493	✓	Flynn et al. (2012)
Erith; E1	DCIS	340	380	10.80	2827	×	Flynn (2014)
Erith; E3	DCIS	340	380	11.10	1879	✓	Flynn (2014)
Kallo; K1	Franki	520	908	9.69	6103	×	De Beer et al. (1979)
Kallo; K2 <sup>a</sup>	Franki	323	539	9.71	2697	×	De Beer et al. (1979)
Kallo; K3	Franki	406	615	9.82	3205	×	De Beer et al. (1979)
Kallo; K4 <sup>a</sup>	Franki	406	815	9.80	5975	×	De Beer et al. (1979)
Kallo; K5	DCIS	406	406	9.33	1666	×	De Beer et al. (1979)
Kallo; K6 <sup>a,b</sup>	DCIS	406	406	11.39	4306	×	De Beer et al. (1979)
Kallo; K7	DCIS	406	609	9.37	2795	×	De Beer et al. (1979)
Le Havre; A4	DCIS	430	430	10.50	1673	✓	Evers et al. (2003)
Le Havre; C1	DCIS	410	430	10.50	1882	✓	Evers et al. (2003)
Ringsend; TP20	DCIS	425	425	12.50	3275	×	Suckling (2003)
Franki-Grundbau; FG1	DCIS	420	420	26.00	5199	×	Franki-Grundbau (2013)

<sup>a</sup>Permanent steel casing.; <sup>b</sup>Increase in diameter to 558 mm from 8.3 m to 9.6 m bgl

Table 1. Database of DCIS piles tests with adjacent CPT data.

## 4 DCIS PILE SHAFT AND BASE BEHAVIOR

### 4.1 Base resistance

Figure 2 presents a summary of the measured unit base resistance  $q_{b0.1Db}$  at a displacement equivalent to 10% of the pile base diameter  $D_b$ , normalized by the average cone resistance at the base obtained using the Dutch method  $q_{cDutch}$  (Schmertmann 1978), against the local cone resistance at the pile base  $q_{c tip}$  for the instrumented DCIS piles in Table 1. The piles at Kallo were not instrumented, but the base resistance was estimated by Chow (1997) by assuming minimal load transfer along the shaft of the piles in the soft clay layers. The use of  $q_{cDutch}$  for normalization purposes, derived using a systematic averaging methodology is important as the majority of sites in Table 1 have highly-layered soil stratigraphies which can have a significant influence on the mobilized base resistance due to the effect of nearby soft/loose or stiff/dense layers (Xu et al. 2008a).

The measured data in Figure 2 shows excellent agreement with a normalized base resistance  $q_{b0.1Db}/q_{cDutch} = 0.6$  recommended for closed-ended driven piles using the UWA-05 method (Xu et al. 2008b). The only exception to this trend is the data point for the test pile installed at Shotton, the results of which were described in detail by Flynn et al. (2013). Analysis of the base resistance of the test pile yielded a normalized base resistance ratio  $q_{b0.1Db}/q_{cDutch} = 1.07$ , which is typically only observed in jacked piles. The high ratio may be due to an increase in soil density following nearby anchor pile installation, in which case the value of  $q_{cDutch}$  will have been underestimated.

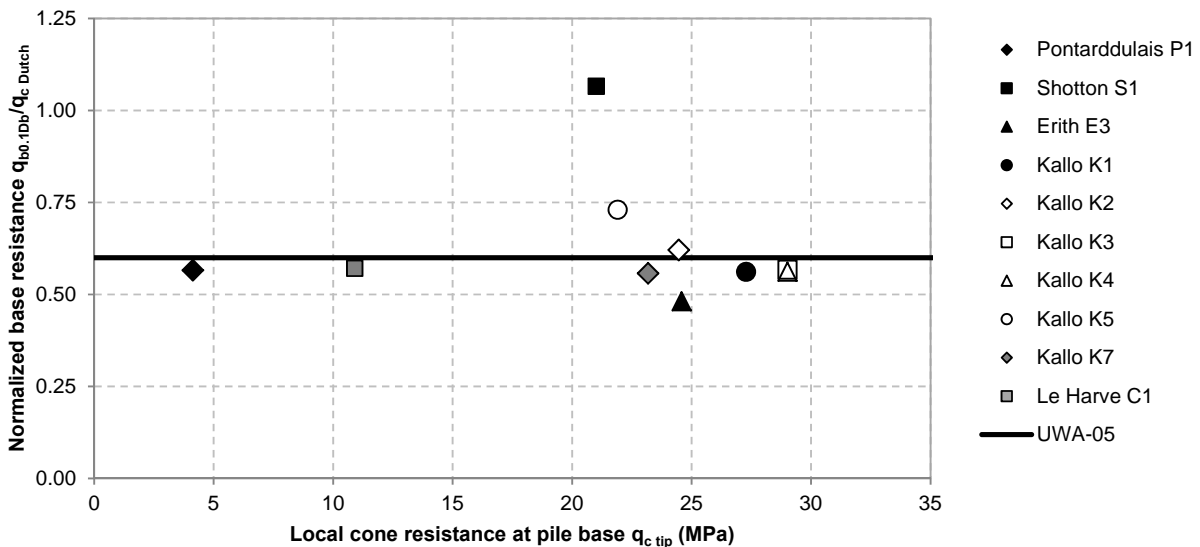


Figure 2. Variation in measured normalized base resistance with local cone resistance at pile base.

### 4.2 Shaft resistance

Figure 3a shows the variation in peak local unit shaft friction  $q_{sult}$ , normalized by the average cone resistance  $q_{cavg}$  between two successive gauge levels along the shaft, with normalized distance from the pile base  $h/D_s$  for the authors' instrumented tests in Table 1. A clear reduction in  $q_{sult}/q_{cavg}$  with increasing  $h/D_s$  is evident, suggesting that friction fatigue (i.e. the reduction in radial effective stress at a particular location on the shaft as tube embedment increases) may be a feature of the DCIS pile type, although further instrumented testing is required for clarification. This may initially seem surprising given that the driving tube is extracted at the end of installation and is replaced by high-slump concrete. Nevertheless,

the surrounding soil is subjected to intense load cycling during installation as the tube embedment increases.

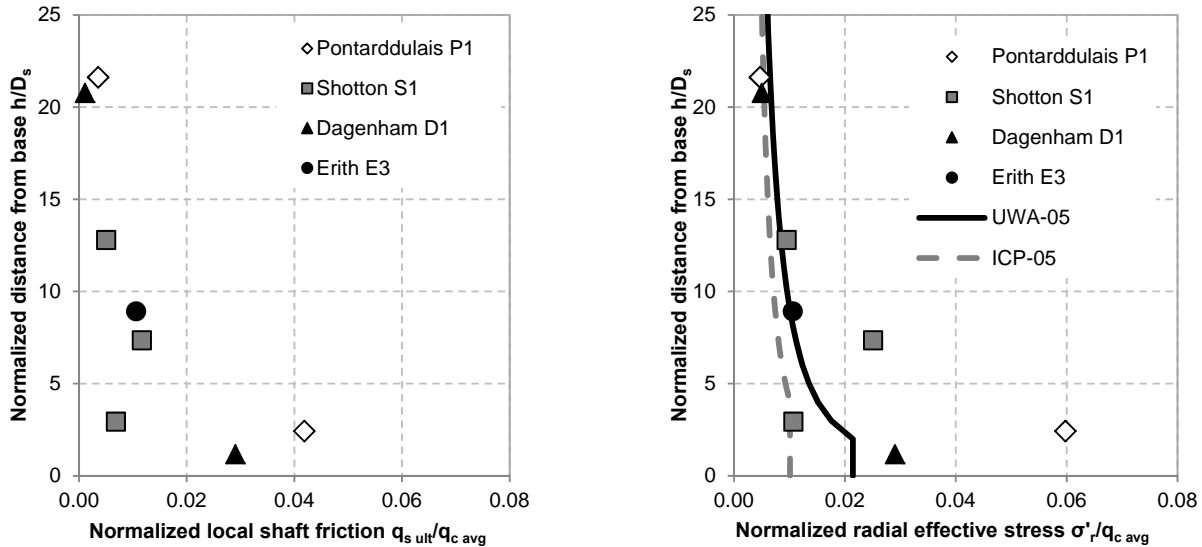


Figure 3(a) Variation in normalized local shaft friction and (b) normalized radial effective stress, with normalized distance from the pile base.

Figure 3b shows the corresponding radial effective stress at failure  $\sigma'_{rf}$  ( $= q_{sult}/\tan\delta_f$ ) normalized by  $q_{cavg}$  for the dataset in Figure 3a, where  $\delta_f$  is taken as the constant-volume friction angle  $\phi_{cv}$  because the high shaft roughness dictates that shearing takes place within the soil. Values of  $\phi_{cv}$  of  $33^\circ$  for sand and  $38^\circ$  for sandy gravel have been adopted based on results of interface shearing tests by Bolton (1986) and Paul et al. (1994), respectively. Theoretical profiles of  $\sigma'_{rf}/q_{cavg}$  predicted by the ICP-05 and UWA-05 methods are shown for comparison. While the trends in the measured data in Figure 3b are broadly similar to the theoretical profiles, some measurements near the pile base are significantly greater which may be a consequence of increased interface dilation due to the presence of gravelly material (with larger average particle sizes than sand) for these piles. While stress relaxation due to wet concrete may occur at the pile shaft after casting, it may not govern the peak shaft resistance during loading. Unfortunately, measurements of radial stress changes on the shaft of DCIS piles after casting and during loading are not possible at present due to the lack of suitable instrumentation.

The similarities identified between DCIS shaft and base behavior and those of preformed driven piles in this section indicate that the appraisal of preformed pile design methods on a DCIS database is a worthwhile exercise.

## 5 PILE CAPACITY PREDICTIONS

The database in Table 1 contains a total of 16 DCIS piles, including 4 expanded-base ‘Franki’ piles. The bases of all piles were founded in cohesionless soil, i.e. sand or gravel, and the piles were subjected to maintained-load compression tests. Three simplified CPT methods, EF-97, LCPC-82 and Van Impe-86, were assessed, together with the four advanced methods - Fugro-05, ICP-05, NGI-05 and UWA-05. The LCPC-82 method was assessed using the shaft coefficients for both DCIS piles (category 1B) and driven closed-ended piles (category 2A). The CPT at Dagenham met refusal at a depth corresponding to the base of the test pile (Pile D1), preventing appropriate averaging of the  $q_c$  profile below the base. Therefore, this site was excluded from the database during assessment of the methods.

The following procedures and assumptions were followed when developing the database:

- The average CPT  $q_c$  profiles for the sites in Table 1 were digitized at intervals of 0.1 m or less and spreadsheets were developed to calculate the shaft and base capacity for each method. No correction was applied at Kallo to account for the use of a mechanical cone penetrometer.
- The depth to the water-table was not reported for 3 of the 8 sites, and was therefore assumed to be at the ground surface. This assumption, in keeping with that made by others using these sites in databases, leads to a lower-bound prediction of capacity for each method due to the reduction in effective stress along the shaft of the pile.
- In general,  $\delta_f$  was assumed to be equivalent to  $\phi_{cv}$  ( $33^\circ$  in sand and  $38^\circ$  in sandy gravel as mentioned previously). Two piles at Kallo had permanently-cased steel shafts, and a value of  $\delta_f = 29^\circ$  was assumed (Jardine et al. 2005) for the portions of the pile shafts in sand in the absence of grading data.
- The majority of the sites in Table 1 had a multilayered stratigraphy comprising soft clay, sand and sandy gravel. Therefore, in order to account for the clay layers in the advanced CPT methods, the peak local shaft friction  $q_{sult}$  in the clay layers was estimated using the expression  $q_{sult} = q_t/35$ , where  $q_t$  = cone resistance (kPa) corrected for end effects, based on the procedure by Schneider et al. (2008). None of the sites had clay layers which contributed more than 50 % of the shaft capacity.
- The load tests for four piles (Pontarddulais P1, Erith E1, Franki-Grundbau FG1 and Ringsend TP20) were terminated prior to reaching a displacement of  $0.1D_b$ , and results extrapolated accordingly using the hyperbolic method by Chin (1970).
- Three sites had multiple pile tests and these tests were considered individually, rather than averaged, due to the relatively limited number of pile tests in the overall database. However, it is acknowledged that this may introduce some bias into the prediction performance for each method.

Table 2 summarizes the resulting ratios of calculated to measured total capacity,  $Q_c/Q_m$ , for each pile in the database, as well as the predictive performance of each method in terms of the arithmetic average and coefficient of variation COV of  $Q_c/Q_m$ . The predictive performance was assessed for the overall database, as well as a subset of 10 DCIS piles which excluded the expanded-base Franki piles (different method of installation) and the test results at Shotton (unrepresentative CPT profile).

The following observations of predictive performance of the CPT methods were made:

1. The mean value of  $Q_c/Q_m$  varied widely with CPT method. The LCPC-82-1B method gave the most conservative estimate, with  $Q_c/Q_m = 0.75$ , while the Fugro-05 method estimate of DCIS total capacity was the least conservative ( $Q_c/Q_m = 1.20$ ).
2. The NGI-05 method provided the best estimate of capacity, with a mean value of  $Q_c/Q_m = 1.00$  and a COV = 27 %. The advanced ICP-05 and UWA-05 methods had mean  $Q_c/Q_m$  values of 0.86 and 0.93 respectively, which are slightly conservative, although the COVs (= 26 %) were in agreement with the similar study of predictive performance for preformed displacement piles using advanced CPT methods by Schneider et al. (2008).
3. The Van Impe-86 method had a COV of 44 % which indicates poor predictive performance.
4. Improved estimates of DCIS capacity were obtained for the LCPC-82 method when the driven pile shaft coefficients (category 2A) were selected ( $Q_c/Q_m = 0.88$ ) instead of the DCIS coefficients (category 1B) for calculating shaft resistance ( $Q_c/Q_m = 0.75$ ).
5. Excluding the results of the Franki piles and Shotton led to improved estimates of mean  $Q_c/Q_m$  and COV for the EF-97 and Van Impe-86 methods. However, the mean value of  $Q_c/Q_m$  for the LCPC-82-1B method (i.e. DCIS coefficients) reduced from 0.75 to 0.67. The ICP-05, NGI-05 and UWA-05 methods remained relatively unchanged.

Since DCIS and preformed displacement piles have comparable base behaviors (as evident in Figure 2), it seems reasonable to conclude that the slightly-conservative predictive performances of the ICP-05 and UWA-05 methods in Table 2 are a direct consequence of increased shaft resistance of DCIS piles in

comparison to traditional displacement pile types. Such increases in shaft friction are likely to be attributed to higher interface friction angles and increased dilation during loading.

Since the proportion of the piles in Table 1 that were instrumented is relatively low (37%), this paper has concentrated on the ability of design methods to predict DCIS total pile capacity rather than shaft and base capacities separately. Further data from instrumented pile tests aimed at separating shaft and base components of load would lead to improved predictive methods.

Site; Pile ref	EF-97	Fugro-05	ICP-05	LCPC-82 Cat. 1B	$Q_c/Q_m$	NGI-05	UWA-05	Van Impe-86
					LCPC-82 Cat. 2A			
Pontarddulais; P1	1.19	0.94	0.57	0.57	0.87	0.52	0.59	0.66
Shotton; S1	0.71	0.99	0.78	0.58	0.70	0.66	0.73	0.41
Erith; E1	0.70	0.75	0.62	0.46	0.51	0.75	0.58	0.69
Erith; E3	0.95	1.29	1.10	0.83	0.92	1.07	1.01	1.23
Kallo; K1	0.56	1.53	0.84	1.07	1.10	1.17	1.13	0.50
Kallo; K2 <sup>a</sup>	0.67	1.36	0.99	0.91	0.96	0.96	1.07	0.84
Kallo; K3	0.73	1.53	1.07	1.06	1.11	1.18	1.16	0.58
Kallo; K4 <sup>a</sup>	0.48	1.31	0.79	0.92	0.95	1.02	0.97	0.41
Kallo; K5	1.00	1.43	1.11	0.88	0.97	1.04	1.04	0.89
Kallo; K6 <sup>a,b</sup>	1.01	0.88	0.66	0.48	0.58	0.76	0.78	0.85
Kallo; K7	0.76	1.61	1.10	1.06	1.12	1.16	1.18	0.66
Le Havre; A4	1.20	1.60	1.17	0.83	1.21	1.51	1.28	1.51
Le Havre; C1	1.04	1.37	1.01	0.72	1.04	1.30	1.10	1.32
Ringsend; TP20	0.44	0.73	0.54	0.41	0.59	0.71	0.69	0.55
Franki-Grundbau; FG1	0.74	0.69	0.61	0.43	0.61	1.20	0.67	0.50
<b>All sites in Table 1 (excluding Dagenham D1)</b>								
No. of piles, $n$	15	15	15	15	15	15	15	15
Average	0.81	1.20	0.86	0.75	0.88	1.00	0.93	0.77
COV	0.30	0.28	0.26	0.32	0.26	0.27	0.25	0.44
<b>Excluding Franki piles, Dagenham D1 and Shotton S1</b>								
No. of piles, $n$	10	10	10	10	10	10	10	10
Average	0.90	1.13	0.85	0.67	0.84	1.00	0.89	0.89
COV	0.27	0.33	0.32	0.34	0.30	0.31	0.29	0.40

Table 2. Performance of CPT design methods in estimating DCIS pile capacity.

## 6 CONCLUSIONS

The development of a database of load tests on DCIS piles has enabled a critical review of the applicability of 7 CPT-based design methods. The development of these design methods have been heavily influenced by research on predominately steel displacement piles. While the review suggests that CPT-based design methods have some merit for DCIS piles, they usually (with the exception of NGI-05 and Fugro-05) underestimate the measured pile capacity. This may be attributed to enhanced interface friction between the cast-in-situ concrete and the soil compared to that developed with preformed piles. The performances of the ICP-05 and UWA-05 methods are promising; however further investigation of appropriate coefficients for DCIS piles is recommended to maximize design efficiency.

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