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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 differences 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).

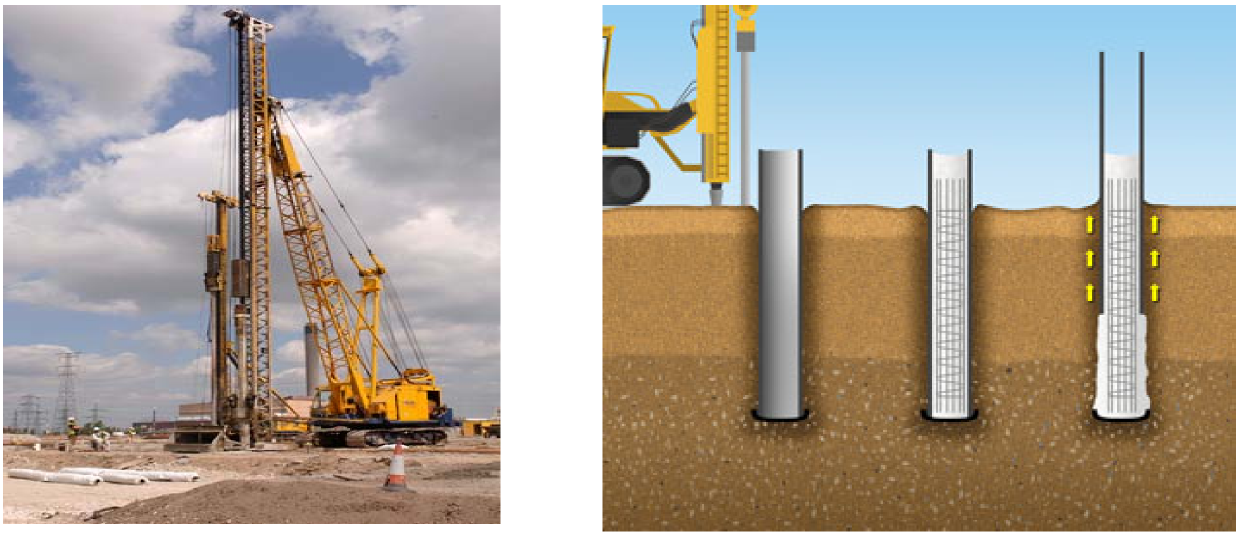

Figure 1(a) Installation of hollow steel tube and (b) Schematic of driven cast-in-situ pile installation (images courtesy of Keller Foundations, UK)

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, σ'_v is the vertical effective stress and δ 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 $Q_{0.1Db,m}$ (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[a]	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[a]	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[a,b]	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)

[a] Permanent steel casing.; [b] 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_{ctip} 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.

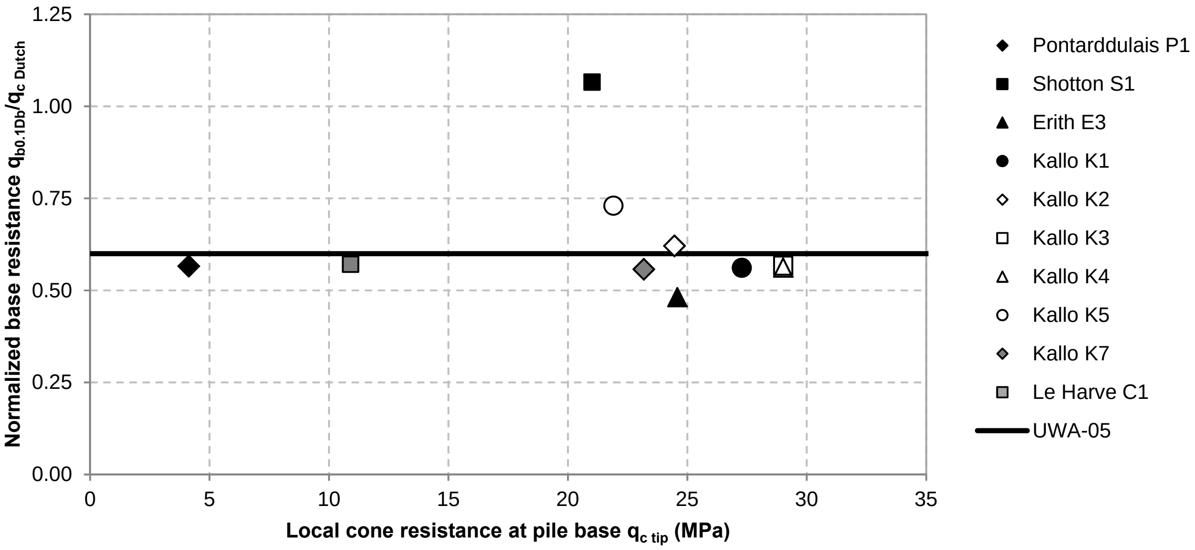

Figure 2. Variation in measured normalized base resistance with local cone resistance at pile base.

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.

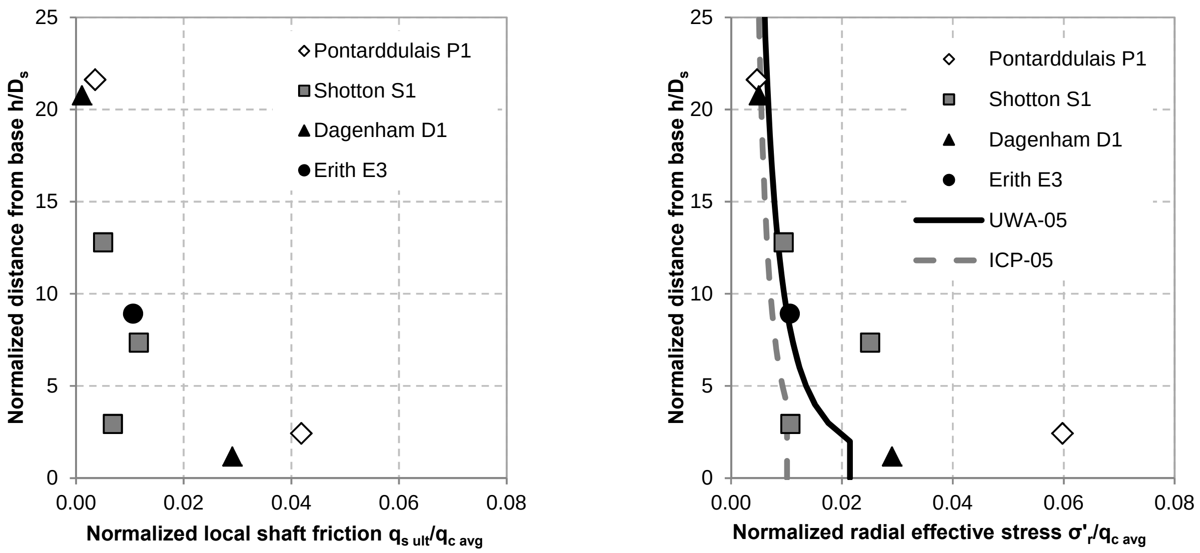

Figure 3(a) Variation in normalized local shaft friction and (b) normalized radial effective stress, with normalized distance from the pile base.

Figure 3b shows the corresponding radial effective stress at failure σ'_{rf} (= q_{sult}/tanδ_f) normalized by q_{cavg} for the dataset in Figure 3a, where δ_f is taken as the constant-volume friction angle ϕ_{cv} because the high shaft roughness dictates that shearing takes place within the soil. Values of ϕ_{cv} of 33^o for sand and 38^o 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 σ'_{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, δ_f was assumed to be equivalent to ϕ_{cv} (33^o in sand and 38^o in sandy gravel as mentioned previously). Two piles at Kallo had permanently-cased steel shafts, and a value of δ_f = 29^o 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	Q_c/Q_m							
	EF-97	**Fugro-05**	**ICP-05**	**LCPC-82 Cat. 1B**	**LCPC-82 Cat. 2A**	**NGI-05**	**UWA-05**	**Van Impe-86**
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[a]	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[a]	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[a,b]	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
All sites in Table 1 (excluding Dagenham D1)								
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
Excluding Franki piles, Dagenham D1 and Shotton S1								
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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REFERENCES

Bolton, M.D. 1986. The strength and dilatancy of sands. *Géotechnique* 36(1): 65-78.

Bustamante, M. & Gianeselli, L. 1982. Pile bearing capacity prediction by means of static penetrometer CPT. In *Proc. 2nd European Symposium on Penetration Testing*, Amsterdam, The Netherlands. pp. 493-500.

Chin, F. 1972. The inverse slope as a prediction of ultimate bearing capacity of piles. In *Proc. 3rd South-East Asian Conf. on Soil Eng.*, Hong Kong. pp. 83-91.

Chow, F.C. 1997. *Investigations into the behaviour of displacement piles for offshore foundations*. PhD thesis, Department of Civil & Environmental Engineering, Imperial College, London.

Clausen, C.J.F., Aas, P.M. & Karlsrud, K. 2005. Bearing capacity of driven piles in sand, the NGI approach. 2005. In *Proc. 1st Int. Symposium on Frontiers in Offshore Geotechnics*, Perth, Australia. pp 677-681.

De Beer, E., Lousberg, E., De Jonghe, A., Carpentier, R. & Wallays, M. 1979. Analysis of the results of loading tests performed on displacement piles of different types and sizes penetrating at relatively small depth into a very dense layer. In *Proc. Conf. on Recent Developments in the Design and Construction of Piles*, Institution of Civil Engineers, London, UK. pp 199-211.

Eslami, A. & Fellenius, B.H. 1997. Pile capacity by direct CPT and CPTu methods applied to 102 case histories. *Canadian Geotechnical Journal* 34(6): 886-904.

Evers, G., Hass, G., Frossard, A., Bustamante, M., Borel, S. & Skinner, H. 2003. Comparative performances of continuous flight auger and driven cast-in-place piles in sands. In *Proc. 4th Int. Geotechnical Seminar on Deep Foundations on Bored and Augered Piles*, Ghent, Belgium. pp. 138-144.

Fleming, W.G.K., Weltman, A., Elson, K. & Randolph, M.F. 2008. *Piling Engineering*. Taylor & Francis, London.

Flynn, K.N. 2014. *Experimental investigations of driven cast-in-situ piles*. PhD thesis (in preparation), Department of Civil Engineering, National University of Ireland, Galway, Ireland.

Flynn, K.N., McCabe, B.A. & Egan, D. 2012. Residual load development in cast-in-situ piles – a review and new case history. In *Proc. 9th Int. Conf. on Deep Foundations and Testing*, Kanazawa, Japan. pp. 765-773.

Flynn, K.N., McCabe, B.A. & Egan, D. 2013. Axial load behavior of a driven cast-in-situ pile in sand. In *Proc. 7th Int. Conf. on Case Histories in Geotechnical Engineering*, Chicago, USA.

Franki Grundbau (2013). *Simplexpfahl*. Retrieved from http://franki.de/pdf/Franki_Simplexpfahl.pdf September 2013

Jardine, R.J, Chow, F.C., Overy, R.F. & Standing, J.R. 2005. *ICP design methods for driven piles in sands and clays*. Thomas Telford, London, UK.

Kolk, H.J., Baaijens, A.E., & Senders, M. 2005. Design criteria for pipe piles in silica sands. In *Proc. 1st Int. Symposium on Frontiers in Offshore Geotechnics*, Perth, Australia. pp. 711-716.

Lehane, B.M. 1992. *Experimental investigations of pile behaviour using instrumented field piles*. PhD thesis. Department of Civil & Environmental Engineering, Imperial College, London.

Lehane, B.M. 2009. Relationships between axial capacity and CPT q_c for bored piles in sand. In *Proc. 4th Int. Seminar on Deep Foundations on Bored and Auger Piles*, Ghent, Belgium. pp. 61-74.

Lehane, B.M., Schneider, J.A. & Xu, X. 2005. The UWA-05 method for prediction of axial capacity of driven piles in sand. In *Proc. 1st Int. Symposium on Frontiers in Offshore Geotechnics*, Perth, Australia. pp. 683-689.

Neely, W.J. 1990. Bearing capacity of expanded-base piles in sand. *Journal of Geotechnical Engineering* 116(1): 73-87.

Paul, T.S., Lehane, B.M., Chapman, T.J.P. & Newman, R.L. 1994. On the properties of a sandy gravel. In *Proc. 13th Int. Conf. on Soil Mechanics and Foundation Engineering*, New Delhi, India. pp. 29-32.

Randolph, M.F. 2003. Science and empiricism in pile foundation design. *Géotechnique* 53(10): 847-875.

Schmertmann, J.H. 1978. *Guidelines for cone test, performance and design*. U.S. Federal Highway Administration, Washington D.C.

Schneider, J.A., Xu, X. & Lehane, B.M. 2008. Database assessment of CPT-based design methods for axial capacity of driven piles in siliceous sands. *Journal of Geotechnical and Geoenvironmental Engineering* 134(9): 1227-1244.

Suckling, T. 2003. Driven cast in situ piles, the CPT and the SPT – two case studies. *Ground Engineering* 36(10): 28-32.

Van Impe, W.F. 1986. Evaluation of deformation and bearing capacity parameters of foundations, from static CPT-results. In *Proc. 4th Int. Geotech. Seminar on Field Instrumentation and In-Situ Measurements*, 25-27 November 1986, Singapore. pp. 51-70.

Xu, X. & Lehane, B.M. 2008a. Pile and penetrometer end bearing resistance in two-layered soil profiles. *Géotechnique* 58(3): 187-197.

Xu, X., Schneider, J.A. & Lehane, B.M. 2008b. Cone penetration test (CPT) methods for end-bearing assessment of open- and closed-ended driven piles in siliceous sand. *Canadian Geotechnical Journal* 45(8): 1130-1141.