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Shaft Capacity of Continuous Flight Auger Piles in Sand

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The Shaft Capacity of CFA Piles in Sand

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Abstract: This paper presents the results of a series of field experiments performed to study the development of shaft resistance on Continuous Flight Auger piles installed in sand. The test piles were instrumented in order to separate the shaft and base resistance, and to allow the determination of the distribution of shaft resistance along the pile shaft. The tests highlighted the importance of accurate calculation of the shaft resistance for non-displacement piles. At a typical maximum allowable pile head settlement of 25 mm, more than 71 % of the pile resistance was provided by shaft friction. Conventional methods of estimating shaft resistance were assessed. It was found that methods which incorporated parameters directly interpreted from in-situ test results provided the most consistent estimates. In the final section, differences between the shaft resistances mobilised on displacement and non-displacement piles are considered.

CE Database subject headings: Field Experiments, Non-Displacement Piles

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Background

The peak unit shaft resistance (τ_f) mobilised by a pile in sand can be estimated using earth pressure theory as:

[1] $$\tau_f = K\,\sigma'_v \tan\delta_f$$

where K is the earth pressure coefficient, σ'_v is the in-situ vertical effective stress and δ_f is the soil-pile interface friction angle. A common difficulty with the application of Equation 1 is the choice of an appropriate K value for design. Paikowsky (2004) notes that design methods proposed by Reese and O'Neill (1999) are in widespread use. They suggest K/K_O (where K_0, is the coefficient of earth pressure at rest), varies with the pile construction method, varying from 0.67 when the pile is excavated using slurry, to 1.0 in a dry excavation. K_0 is notoriously difficult to measure but can be estimated using the method proposed by Mayne and Kulhawy (1982):

[2] $$\begin{aligned} K_0 &= (1 - \sin\phi_p) && \textit{for normally consolidated soil} \\ K_0 &= (1 - \sin\phi_p)\,OCR^{\sin\phi_p} && \textit{for overconsolidated soil} \end{aligned}$$

where ϕ'_p is the peak friction angle and OCR is the Over-Consolidation Ratio.

Where accurate estimates of K_0 are unavailable, Reese and O'Neill suggest an empirical correlation based on a conservative estimate of the shaft resistance (in kPa) measured from a series of field tests:

[3a] $$\tau_f = \beta\,\sigma'_v$$

[3b] $$\beta = \lambda\,[1.5 - 0.245\,(z)^{0.25}]$$

where: $\lambda = 1.0$ when the Standard Penetration Test blowcount (corrected for energy and stress level effects), N_{60} is ≥ 15, and $\lambda = N_{60}/15$, when $N < 15$, and z is the depth in metres. β values predicted using Eqn 3b should be within the range $0.25 \leq \beta \geq 1.2$, whilst τ_{fmax} is ≤ 200 kPa.

Research on the interface characteristics of sand-steel interfaces by Ramsey et al. (1998) found that the soil-pile interface friction angle depends on the mean particles size (D_{50}) of the sand and the surface roughness of the interface. For concrete piles, where the interface roughness is relatively large, the slip surface migrates into the sand mass, and $\delta_f = \phi'_{cv}$, the constant volume (or critical state) friction angle of the soil.

Because of the difficulties associated with obtaining high quality samples of sand with which to estimate parameters such as; ϕ', OCR and δ_f, the use of in-situ tests such as the Standard Penetration Test (SPT) and Cone Penetration Test (CPT) are widespread. Many correlations exist between the SPT blowcount (N_{60}) and the CPT end resistance (q_c) and soil properties, whilst direct correlations between the unit shaft resistance and in-situ test results have also been proposed:

[4a] $$\tau_f = \kappa N$$

[4b] $$\tau_f = \alpha\, q_c$$

Meyerhof (1976) suggested κ = 1.0 for bored piles and 2.0 for driven piles when the unit shaft resistance has units of kPa. However, Robert (1997) compiled a large database of bored and driven piles in sand and concluded that there was no systematic difference between the shaft resistance mobilised by bored and driven piles, with κ = 1.9 giving the best-fit to the available data. Values for α of between 0.004 and 0.005 have been proposed for bored piles (Bustamante and Giannessli 1982). Values for driven piles are typically assumed to be double the values used for bored piles.

Measurement of the horizontal effective stress (σ'_h) during installation of the Imperial College instrumented displacement pile (ICP) at a dense sand site in Dunkirk (Chow 1997) and a loose to medium-dense sand at Labenne (Lehane 1992), have resulted in advances in our understanding of the mechanisms controlling the development of shaft resistance on displacement piles in sand. This led to the development of effective stress design methods for displacement piles. Jardine et al. (2005) show that the local shaft resistance is given by:

[5] $$\tau_f = (\sigma'_{hc} + \Delta\sigma'_{hd}) \tan \delta_f$$

where σ'_{hc}, is the fully equalized horizontal effective stress after pile installation and $\Delta\sigma'_{hd}$ is a component derived by dilation during loading. Chow (1997) found that σ'_{hc} values at a given location on the ICP at both sites were almost directly proportional to the q_c value at that level and the distance from the level to the pile base (h) normalised by the pile diameter (D), See Figure 1. These findings were incorporated into the widely used design method for displacement piles known as the Imperial College design method (Jardine et al. 2005) and a similar approach known as the University of Western Australia (UWA) method (Lehane et al. 2005), where:

[6] $\sigma'_{hc} = 0.03\ q_c\ (h/D)^{-0.5}$

A minimum h/D value of 2 should be used is Equation 6. White and Lehane (2004) note that the ICP was installed using the same jacking sequence at both Labenne and Dunkirk. Using centrifuge model piles, they investigated the effect of the number of load cycles (N) experienced during installation on the horizontal stress mobilised. They found that σ'_{hc}/q_c was not unique at a given h/D level. Rather, the value varied with the number of loading cycles experienced during installation. Gavin and O'Kelly (2007) report field tests on instrumented model piles installed using a range of jacking stroke lengths in dense sand. Their data agreed with the earlier centrifuge tests finding that a pile installed in a single long jacking stroke (N=1) as shown in Figure 1, developed much higher σ'_{hc}/q_c values than piles which experienced a greater number of load cycles during installation. However, when the pile was subjected to a relatively small number of additional load cycles, the horizontal stresses at all h/D levels reduced rapidly to residual values, which were similar (at a given h/D value) to those measured on the ICP. They noted that even after a large number of cycles, σ'_{hc}/q_c values were highest near the pile tip, an effect that was due in part to high residual stresses built up around the base of the displacement pile during pile installation.

A feature of the field experiments with the ICP and Gavin and O'Kelly's model pile tests was that due to interface dilation, σ'_h values measured on the small diameter model piles increased during loading. Lehane (1992) suggests that the dilation induced increase in horizontal stress ($\Delta\sigma'_{hd}$) could be predicted using cavity expansion theory:

[7] $$\Delta\sigma'_{hd} = \frac{4\,G\,\delta_h}{D}$$

where G is the shear modulus of the soil mass and δ_h is the horizontal displacement of a soil particle at the pile-soil interface. As $\Delta\sigma'_{hd}$ is inversely proportional to the pile diameter, Lehane (1992) concluded that while dilation effects may dominate the shaft resistance measured in model displacement pile tests, it is unlikely to contribute more than 5% of the shaft resistance of full-scale displacement piles ($D \geq 300$ mm). However, in later work Lehane et al. (2005) illustrate the effect of interface dilation on the shaft resistance mobilised during centrifuge tests on piles buried in sand with diameters ranging from 3 to 18 mm. Their results show that the maximum shear stress on the pile decreased as the pile diameter and stress level increased. The authors suggest that interface dilation in dense sand, even on large diameter bored piles, may influence the α values back-figured from pile load tests such that α should vary with pile diameter and sand relative density (given that G is affected by relative density) in a manner compatible with Equation 7.

In light of the uncertainties over the differences between the shaft resistance mobilised by displacement and non-displacement piles in sand, and the contribution of interface dilation to the shaft resistance mobilised by a full-scale pile, field-testing of a 450 mm and 800 mm Continuous Flight Auger (CFA) piles were performed and the shaft resistance developed by these piles is discussed in this paper. Two static load tests on full-scale instrumented piles are described. Two pile diameters were considered to assess whether the mobilised shaft resistance was affected by pile diameter. Strain gauges placed at intervals along the pile shaft allowed the shear stress distribution along the pile to be determined. The mobilised shaft shear stress is compared to Cone Penetration Test data measured adjacent to the test piles and the results are compared with measurements made on full-displacement piles in an attempt to better understand the different mechanisms affecting the mobilisation of shaft resistance on CFA and driven piles.

Ground Conditions

The pile tests described in this paper were performed in Killarney, in South-West Ireland. The area is underlain by glacial sand and gravel deposits (typically > 20 m deep) the formation of which are described by Wright (1927) and Warren (1991). A series of laboratory and in-situ tests including Cone Penetration (CPT) and Standard Penetration tests (SPT) were performed at the site. The locations of the in-situ tests are shown in Figure 2. Ground conditions at the test site consist of approximately 2 m of mixed (sand, silt and clay) deposits overlying a deep deposit of sand. The CPT end resistance, q_c and the shaft resistance, f_s measured in the vicinity of the test piles are shown in Figure 3. The sand deposit can be considered as three sub-units; an upper dense sand, with q_c ranging from 5 to 15 MPa and highly variable f_s values from 2 m to 6.5 m below ground level (bgl). A layer of loose sand underlies this, with q_c between 2 and 6 MPa and f_s between 10 and 20 kPa. Below 10-12 m bgl. both q_c and f_s values double, and rise steeply with depth. This layer of dense sand with q_c > 10 MPa was noted at depths varying from 14 – 17 m bgl. in the area in which the pile tests were performed. The depth of the various sub-units varied across the site, with the upper dense and loose sand layers thinning somewhat to the north of the pile test locations.

The SPT (N) data for all boreholes are shown in Figure 4. The large variation of SPT N values across the test site reflects the range in the depths of the various sub-units. The SPT design line for the area in which the test piles were installed is shown in Figure 4.

The results of particle size distribution tests on samples taken at various depths in the sand strata are shown in Figure 5. These show that the particle size distribution of the three sub-units is similar, with some tendency for the mean particle size (D_{50}) to reduce slightly with depth from 0.35 mm at 2m bgl. to 0.31 mm at 10 m bgl.

Since the conventional shell and auger boreholes did not provide high quality samples of sand for strength testing, the strength parameters for the sand have been inferred from in-situ test results. The parameters inferred using appropriate average values for the three sub-units are shown in Table 1.

Test Pile Details

Two instrumented test piles were installed at the site using a Soilmec CM-48 piling rig. The first pile was installed using an 800 mm diameter auger to 14 m bgl. and the second using a 450 mm auger to 15 m bgl. The piles were deliberately installed to lengths shorter than their design length (≈ 20 m) to ensure that the shaft resistance was fully mobilised during static loading. Both piles were subjected to static load tests with maximum loads of 4250 kN and 1700 kN being applied to the 800 mm and 450 mm piles respectively. Tension piles that provided the reaction for the test load were connected to a heavily stiffened load transfer beam. The jacking system was attached to a hydraulic powerpack, which in turn was connected to a data-logger. Pile settlement was monitored by four linear variable displacement transducers placed at the pile head, which were attached to an independent reference beam. The test was controlled by an automated system. This allowed for precise specification of a target load whilst simultaneously recording load and displacement data.

The load tests, which included at least two unload-reload cycles, were scheduled to last a minimum of thirty hours. The specified individual minimum time period for each load increment to be held was until the pile settlement rate was less than 0.1 mm per hour. Whilst this target was achieved for the majority of load steps, it was not possible to comply with this specification at the final load increments of the load tests where very large pile head settlements were recorded.

The 800 mm pile was reinforced along its full length with 7 No. T32 bars, whilst the 450 mm pile had 5 No. T24 bars. Vibrating-wire type (Gage Technique TES/S-J/T) embedment strain gauges were attached to the steel reinforcement cage in groups of four at fixed depths of 0 m, 3.10 m, 6.05 m, 8.90 m and 11.85 m bgl. on the 800 mm pile and 1.55 m, 4.65 m, 7.60 m, 10.45 m and 13.4 m bgl. on the 450 mm diameter pile (see Figure 6). The distribution of load in the test pile was calculated from the strain gauge readings by assuming the pile diameter was equal to the auger diameter and using a concrete stiffness which varied with strain level. The non-linear stiffness-strain response of the pile concrete was quantified using the tangent modulus approach (Fellenius 2001). In addition the effects of creep were accounted for (See

Lehane et al. 2003). Further details of the strain gauge interpretation are contained in Cadogan (2008).

Load Test Results

The overall load-displacement response of the piles is shown in Figure 7. The following observations can be made:

- Although no universally accepted failure criterion is available, it is clear that the rate at which the pile resistance increases slows considerably as the pile head displacement approached 10% of the pile diameter (0.1D).
- The serviceability limit state for the test piles stated that the pile head settlement should not exceed 25 mm at the working load. The proportion of the total load supported by shaft resistance at this displacement is 71% for the 800 mm pile and 78% for the 450 mm pile.
- Whilst the base resistance continues to rise with increasing pile head displacement, the peak shaft resistance reduced by between 12-15 % on both piles after reload tests were performed.
- Reloading resulted in a much stiffer pile response. For example, during first-time loading of the 450mm pile, a pile head displacement of 17 mm was required to mobilise a total pile resistance of 1000kN. This resistance was mobilised after just 4.5 mm movement in the re-load test.

The high proportion of the pile resistance mobilised along the shaft at the typical working displacement of 25 mm highlights the importance of accurate determination of the shaft load contribution of CFA piles.

Shaft Resistance

The average shaft resistance (τ_{av}) mobilised in the sand layers (below 1.55-3 m bgl.) during the static load tests is plotted against the normalised pile head settlement, w/D (%) in Figure 8. The ultimate (τ_{av}) value recorded $\approx$ 35-36 kPa was almost identical on both the 450 mm and 800 mm diameter piles, suggesting that the effect of interface dilation was not significant. A notable feature of the tests was the relatively soft initial stiffness response of the shaft resistance. Pile head movement of approximately 3% of the pile diameter ($\approx$0.03D) was required to mobilise this shaft resistance. This is higher than the values of 1.5 – 2% at which shaft resistance is expected to be fully mobilised (e.g. Fleming et al. 1990). Bearing in mind the large pile diameter, such relative movements could approach the serviceability limit criteria for the pile.

The local shaft resistance (τ_s) can be inferred from the load distribution in the pile measured by the strain gauges by assuming the load is shed uniformly along the pile shaft between the strain gauge locations. The distribution of local shaft resistance on the 450 mm diameter pile is shown in Figure 9. It is clear that the majority of shaft resistance is mobilised along the upper part of the pile (in the upper dense sand) between 4.65 and 7.6 m bgl. τ_s values over this region were 2-3 times higher than over the lower pile shaft (in the loose sand between 7.6m and 13.4 m bgl.). The distributions of τ_s were similar on the 800 mm diameter test pile. Reductions in shaft resistance (shown in Figure 7) were noted to be largely due to reductions, at large pile head movements (not shown in Figure 9 for clarity), of shaft resistance along the lower portion of the pile.

Discussion

In this section, the results of the load tests on the CFA piles are compared to predictions using the conventional design approaches commonly adopted in industry. In addition the differences between the development of shaft resistance on displacement and non-displacement piles is examined by comparing the CFA load tests to test on instrumented displacement piles installed in sand.

Conventional Earth Pressure Theory

The principle challenge in adopting a conventional earth pressure approach such as Eqn [1] in design practice lies in the choice of the earth pressure coefficient K, which links the effective horizontal earth pressure mobilised during the load test (σ'_h) and σ'_v ($K = \sigma'_h / \sigma'_v$). Although K values were not measured directly during the load tests, they can be inferred from the measured shear stress profiles using Eqn [1], assuming for the rough concrete interface that $\delta_f = \phi_{cv}$. The K values mobilised at the peak shear stress (shown as discrete circles) are plotted against depth in Figure 10. K values on both piles are seen to decrease relatively consistently with depth, reducing from 2.35 at 3m bgl. to 0.42 at 12 m bgl.

In order to predict K values using Eqn [2], an estimate of OCR and thus the maximum pre-consolidation pressure of the sand is required. Given the recent removal of 3 m of sand prior to pile construction, and also the glacial history of the area, a range of possible pre-consolidation pressures between 100 and 500 kPa were considered as lower and upper bounds to the likely value. Estimates of K using Eqn [2], with these values are shown in Figure 10. It appears that the larger value provides good estimates of the K values along the upper portion of the pile shaft (within 6 m bgl, in the upper dense sand), whilst the lower value provides a better prediction at depths > 6 m bgl. As the lower, loose sand cannot have a lower pre-consolidation stress than the upper (younger) dense sand, the difficulty in the application of Eqn [2] is apparent. If an intermediate, consistent, pre-consolidation pressure between 100 and 500 kPa was adopted it would result in underestimation of the shaft resistance along the upper pile shaft and overestimation along the lower shaft. It is clear that the strong effect of in-situ density on the mobilised K values is not reflected in the prediction of K values using Eqn [2].

Reese and O'Neill (1999) β approach

In the β approach the earth pressure coefficient and interface friction angle are combined into a single parameter ($\beta = \tau_f/\sigma'_v$). Measured β values are compared in Figure 11 with values estimated using Eqn [3]. Whilst β values are under-predicted over the majority of the pile shaft, the difference between measured and predicted values decrease with increasing pile penetration. It was noted in the application of Equation [3b] that β depends on the depth (z) and a constant (λ). The constant (λ) is 1.0 when $N_{60} \geq 15$, and therefore at the test site (where N varies from 15-30) the shaft resistance is assumed to vary only with σ'_v. The improved fit at depths greater than 6 m bgl. may be due to the fact that the actual N value of 15 at this depth corresponds to the assumed maximum value allowed in the design equation. This suggests that if λ

were allowed to increase for N values in excess of 15 (to reflect the higher in-situ density particularly close to the ground surface) an improved prediction would result.

Direct correlation between q_f and SPT or CPT

The average shaft resistance mobilised in the sand, normalised by the average N_{av} value along the pile shaft and q_{cav}, is plotted against normalised pile displacement in Figure 12. The resulting κ (= τ_{av}/N_{av}) and α (=τ_{av}/q_{cav}) values of ≈ 1.8 and 0.008 are similar to those used for the design of displacement piles in sand. Because of the wide variation of local shaft resistance distribution noted in Figure 9, the normalised peak shaft resistance averaged over the pile length (shown in Figure 12), are compared to the normalised maximum local shear stress (τ_f) values in Table 2. It is clear from Table 2 that the average κ and α values are in good agreement with local values.

Comparison between shaft resistance on displacement and non-displacement piles

The peak horizontal stress (σ'_{hp}) mobilized during the static load tests on the test piles can be inferred from the measured maximum shear stress and the inferred interface friction angle ($\sigma'_{hp}=\tau_f /\tan\delta_f$). σ'_{hp}/q_c values inferred on the two CFA test piles are shown in Figure 13 (where h/D is taken as the mid-point between the strain gauge arrays). The σ'_{hp}/q_c values are similar on both piles, and do not appear to vary significantly along the pile shaft (despite the fact that the sand density is much higher near the top of the pile). Given that the normalized horizontal stresses are similar at all points despite pile diameter and sand density differences, this suggests that interface dilation does not provide a significant component of the σ'_{hp} values developed on these piles (and thus the values of the peak and fully equalized horizontal effective stresses mobilized by the CFA piles are similar). With this in mind, the σ'_{hp}/q_c values mobilized on the CFA piles are compared to σ'_{hc}/q_c values measured on displacement piles in Figure 13. It is clear that normalized horizontal stresses near the tip of the CFA piles (closed symbols) are not as high as in the case of the displacement piles (open symbols), as no large residual base stresses exist. However, the distribution of σ'_{hp}/q_c along the CFA pile shaft is relatively uniform, and as the pile does not experience any load cycling during installation, the ratio σ'_{hp}/q_c at points remote from the pile tip (where the displacement piles have experienced the largest number of load cycles) are slightly higher than those measured on the displacement piles.

Conclusions

A case history of two compression load tests to large displacement on CFA piles installed in sand is presented. The importance of shaft resistance in providing the majority of the load resistance at typical allowable pile head settlements was demonstrated. The ability of current design approaches to estimate the mobilized shaft resistance was assessed. In addition, differences between the shaft resistances mobilised by CFA and displacement piles were considered.

The following observations were made:

1. Estimates of shaft resistance using conventional earth pressure theory were seen to be the least reliable of the methods considered. Because high quality samples of sand are rarely available in routine design situations, accurate assessment of some of the required soil parameters such as OCR are difficult. A wide range of likely OCR values considered for the test site did not provide a reasonable fit to the measured shaft resistance.

2. The β approach (Reese and O'Neill 1999) was seen to underestimate the shaft resistance, particularly in the upper medium-dense sand layer.
3. Direct correlations between shaft resistance and in-situ N and q_c values captured the strong effect of sand density on the τ_f value mobilized along the pile shaft. However, the α and κ values mobilized by the CFA piles were approximately double the values used in routine design and were comparable to those used in the design of displacement piles.
4. Although the data is limited, a comparison of the distribution of σ'_{hp}/q_c along the shaft of the two CFA piles suggested that interface dilation effects were small in these tests. Comparing the σ'_{hp}/q_c with σ'_{hc}/q_c values recorded on displacement piles suggested that the horizontal stress distribution along the displacement piles was influenced by the stress regime created during installation, whilst normalized horizontal stresses were relatively constant along the shaft of the CFA piles.

The pile test results suggest that the shaft distribution mobilized by a CFA pile in sand depends on the in-situ sand state as reflected by the CPT q_c or SPT N values. For displacement piles the effect of elevated base stresses and friction fatigue affect the distribution of shaft resistance, resulting in higher shaft resistance values closer to the tip of displacement piles and lower values remote from the pile tip. The effect of this is such that the average shaft resistance along a short pile (low length to breadth ratio) may be higher for a displacement pile when compared to an equivalent short CFA pile. In contrast, the average shaft resistance of a long (slender) CFA pile may exceed that of a similar displacement pile.

Although the local unit shaft resistance developed by a CFA pile at large h/D values (h/D>10) were seen to be similar to those mobilized on displacement piles, the relatively low stiffness response exhibited by the CFA piles suggests that the pile head displacement necessary to mobilize this resistance must be considered in the design procedure.

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Table 1 Soil properties from in-situ tests

Layer.	D_r (%)from q_c[1]	D_r (%) from N[2]	$\phi°$ from q_c[3]	$\phi°$ from N[4]	$\phi°_{cv}$ from D_r[5]
Upper Dense Sand	72	74	40	41	36
Loose Sand	27	52	34	35	31
Lower Dense Sand	71	74	41	41	36

[1] $D_r = \frac{1}{2.91}\ln\left(\frac{qc}{61\sigma_v'^{0.71}}\right)$ - Lunne and Christofferson (1983)

[2] $D_r = \sqrt{\frac{N_{160}}{55}}$ - Skempton (1986)

[3] $\phi_p' = 17.6^\circ + 11\log(q_{c1}) \quad where \; q_{c1} = \frac{(q_c / p_{atm})}{\sqrt{\sigma_v' / p_{atm}}}$ - Hatanaka and Uchida (1996)

[4] $\phi'_p = 15.4\ N_{160}^{0.5} + 20$ – Kulhawy and Mayne (1990)

[5] From Figure 1, Kulhawy and Chen (2007)

Table 2 Comparison of average and local normalised shaft resistance

Location	τ_f (kPa)	N	q_c (kPa)	κ	α
800 mm (τ_{av} from 3-12 m bgl.)	35	20	4,611	1.75	0.008
800 mm (τ_s from 3-6 m bgl)	59	30	6,450	2	0.008
800 mm (τ_s from 6-9 m bgl)	35	15	4,047	2.3	0.009
800 mm (τ_s from 9-12 m bgl)	20	15	3,337	1.3	0.006
450 mm (τ_{av} from 4.5-13.5 m bgl.)	36	20	4,326	1.8	0.008
450 mm (τ_s from 4.5-7.5 m bgl.)	56	23	7,325	2.5	0.008
450 mm (τ_s from 7.5-10.5 m bgl.)	30	15	3,267	2	0.0092
450 mm (τ_s from 10.5-13.5 m bgl.)	23	15	3,287	1.5	0.007

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Notation

The following symbols are used in this paper:

CPT	=	Cone Penetration Test
D	=	Pile external diameter
G	=	Shear Modulus of Soil
ICP	=	Imperial College Pile
K	=	Coefficient of earth pressure
K_0	=	Coefficient of earth pressure at rest
N	=	Number of load cycles
OCR	=	Overconsolidation ratio
P_{atm}	=	Atmospheric pressure
R	=	Pile external radius
SPT	=	Standard Penetration Test
UWA	=	University of Western Australia
bgl	=	Below ground level
f_s	=	Friction sleeve resistance measured during cone penetration test
h		Height above the pile tip
q_c	=	End bearing resistance measured during cone penetration test
α	=	A reduction factor applied to q_c when estimating shaft resistance
β	=	An empirical factor linking τ_f and σ'_v
δ_f	=	Interface friction angle at failure
δ_h	=	Horizontal displacement of a soil particle at the pile-soil interface
ϕ'_p	=	Peak friction angle
κ	=	Empirical factor linking τ_f and SPT N
λ		Empirical factor linking τ_f and CPT q_c
σ'_v	=	Vertical effective stress
σ'_h	=	Horizontal effective stress
σ'_{hc}	=	Horizontal effective stress measured when the pile is stationary
$\Delta\sigma'_{hd}$	=	Increase in horizontal effective stress during pile loading
σ'_{hp}	=	Horizontal effective stress at peak shear stress
τ_{av}	=	Average shear stress acting on the pile shaft
τ_f	=	Peak local shear stress
τ_s	=	Local shear stress