Cone penetration test-based load-transfer formulations for driven piles in sand

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This paper presents the development of load-transfer curves that can be used to estimate the displacement under axial load of driven piles in sand. The maximum skin friction and end-bearing stress are obtained from new ISO 19901-4 cone penetration test (CPT)-based formulations, which were calibrated using a high-quality database of static pile load tests compiled for this purpose. The load–displacement responses measured in the 71 static load tests in this database are used to derive CPT-based non-linear load-transfer curves. It is shown that good estimations of pile displacements can be obtained using CPT data and the normalised formats of shaft and base load-transfer functions provided in the American Petroleum Institute and ISO 19901-4 recommendations.

KEYWORDS: piles & piling; sands; settlement

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NOTATION

D	pile diameter (or B if square pile)
D_i	internal pile diameter (pipe pile)
D_r	relative density
G	operational shear stiffness
$G_{\max}$	small strain shear stiffness
PLR	ratio of plug length to embedded length
p_a	atmospheric pressure (= 100 kPa)
Q	pile head load
Q_{ult}	pile ultimate load
q_b	pile base stress
$q_{b0.1}$	base resistance at a displacement of 10% of the pile diameter
q_c	cone penetration test end resistance
q_{c1N}	normalised cone tip resistance
w	local pile displacement
w_b	pile base displacement
w_f	pile displacement at failure
$\Delta\sigma'_{rd}$	change of radial effective stress during shearing
δ_h	pile head displacement
δ_{hm}	measured pile head displacement
δ_{hc}	calculated pile head displacement
σ'_v	vertical effective stress
τ	local shaft shear stress
τ_f	local shaft shear stress at failure

INTRODUCTION

A joint industry project (JIP) was set up in 2013 by the Norwegian Geotechnical Institute (NGI) to examine the reliability of a range of existing methods employed for evaluation of the axial capacity of driven piles in sand. To assist with this aim, the University of Western Australia (UWA) together with NGI compiled a database of high-quality pile load tests in sand and clay, which is now referred to as the ‘Unified’ database (Lehane *et al.*, 2017). Input into the selection of an agreed set of high-quality tests for this database

was provided by representatives from Imperial College London, Fugro BV Ltd, NGI and UWA. Full details of the database are provided in Lehane *et al.* (2017), which also examines the ability of existing cone penetration test (CPT) methods to predict pile capacity. On completion of this work and motivated by the wish to achieve consensus among proponents of these CPT methods, a new JIP was formed with the objective of using this database to develop a CPT method that would supersede the existing CPT methods used to predict axial capacity of piles in sand and clay (Nadim *et al.*, 2020). The new ‘unified’ CPT method for piles in sand is described by Lehane *et al.* (2020) and replaces previous recommendations for axial capacity estimation in ISO 19901-4 (ISO, 2020).

This paper is an extension of the research on axial pile capacities in sand and uses the ‘unified’ database of load–displacement curves for calibration of CPT-based load-transfer curves that can be used to estimate the displacement under axial load of driven piles in sand. This database comprises 71 pile tests with the range of pile diameters, pile lengths, CPT end resistances (q_c) and set-up times shown in Fig. 1. Further information relating to the pile end condition, pile shape, loading direction and pile material is provided in Table 1. The transfer curves derived are aimed to allow reasonable estimates of pile displacements at typical working loads when the applied load (Q) is less than about half of the ultimate capacity (Q_{ult}) – that is the global factor of safety on ultimate shaft friction is about 2.0.

Unified method for calculation of axial capacity

The derivation of load-transfer curves requires definition of the ultimate shaft friction that can develop at any location on the pile shaft (τ_f) and the ultimate end-bearing stress acting over the full pile base area at a displacement of 10% of the pile diameter ($q_{b0.1}$). The new unified CPT method, which is described in full by Lehane *et al.* (2020), allows derivation of these parameters using the formulations provided in Table 2 and the measured q_c profiles. The basis of the formulation employed for τ_f , as evident from multiple instrumented pile tests, is the direct proportional relationship with q_c of the radial effective stress acting on a pile shaft after installation.

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Fig. 1. Pile diameters, pile lengths, set-up time sand average q_c values in the 'unified database' for driven piles in sand

Table 1. Characteristics of the unified database of driven piles in sand

Description	Number of piles		All
	Closed	Open	
Material			
Steel	20	25	45
Concrete	22	4	26
Shape			
Circular	31	29	60
Square (and others)	11	0	11
Loading			
Compression	31 (CEC)	17 (OEC)	48
Direction			
Tension	11 (CET)	12 (OET)	23
Total	42	29	71

CEC, closed-ended pile tested in compression; CET, closed-ended piles tested in tension; OEC, open-ended pile tested in compression; OET, open-ended pile tested in tension.

Table 2. Unified CPT method for determination of τ_f and $q_{b0.1}$

$\tau_f = (f_t/f_c) (\sigma'_{rc} + \Delta\sigma'_{rd}) \tan 29^\circ$ $\sigma'_{rc} = (q_c/44) A_{re}^{0.3} [\text{Max}[1, (h/D)]]^{-0.4}$ $\Delta\sigma'_{rd} = \left(\frac{q_c}{10}\right) \left(\frac{q_c}{\sigma'_v}\right)^{-0.33} \left(\frac{d_{CPT}}{D}\right)$ $q_{b0.1} = [0.12 + 0.38 A_{re}] q_p$ $A_{re} = 1 - \text{PLR} (D_i/D)^2$ $\text{PLR} \approx \tanh[0.3(D_i/d_{CPT})^{0.5}]$	$f_t/f_c = 1.0$ in compression and 0.75 in tension $Q_{base} = q_{b0.1} (\pi D^2/4)$ $A_{re} = 1$ for closed-ended pile $d_{CPT} = 35.7 \text{ mm}$
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q_p , end bearing mobilised at large displacements at the level of the pile tip by a pile with a diameter of D_{eq} ($= D A_{re}^{0.5}$). In relatively homogeneous sands, q_p can be taken as the average q_c value within a zone $1.5D$ above and below the pile tip. In more variable strata, designers can assume $q_p = 1.2q_{c,Dutch}$ (Schmertmann, 1978) or adopt the technique proposed by Boulanger & DeJong (2018).

Source: Lehane *et al.* (2020)

Additional factors known to influence shaft friction are also incorporated, namely (a) the degree of soil displacement (plugging) during installation, (b) the relative pile tip depth, (c) the sand–pile interface friction angle, (d) changes in radial stress during loading and (e) the loading direction (tension or compression). The formulation for $q_{b0.1}$ is also based on a direct relationship with q_c and allows for the important influence of plugging during installation of pipe piles. The equations for shaft friction (Table 2) allow for a reduction in the relative influence of dilation (captured by the $\Delta\sigma'_{rd}$ term) and plugging (by way of the effective area ratio term, A_{re}) as the pile diameter increases, hence enabling safe extrapolation from the database piles (the majority of which had diameters less than 800 mm).

Load-transfer functions

The load-transfer method, first proposed by Coyle & Reese (1966), is a popular means of estimating axial pile displacement. This method combines the axial stiffness of multiple pile elements with associated non-linear load-transfer functions (or springs) to represent the variation of shaft friction (τ) with local displacement (w) and the variation of pile base load (q_b) with base displacement (w_b).

Shaft load-transfer function (τ – w)

The τ – w spring, which is more commonly referred to as a t – z curve, in any given soil horizon depends on the specific non-linear shear stiffness–shear strain relationships of elements of soil extending radially from the pile within the complex stress field set up following pile installation. In view of these complexities, a practical approach commonly employed is to presume either a power-law, hyperbolic or parabolic variation of τ with w and to define a displacement (w_f) at which τ attains its maximum value of τ_f . Fellenius (2018) reviews a variety of functions proposed for these load-transfer curves such as the ratio function of Gwizdala (1996), the hyperbolic function of Chin (1970) and the exponential relationship of van der Veen (1953). The parabolic form of the normalised τ – w curve, proposed by Randolph (2003), is employed in this study and is expressed as

$$\frac{\tau}{\tau_f} = \left(\frac{G}{\tau_f}\right) \left(\frac{w}{2D}\right) \left[1 - \frac{w}{2w_f}\right] \quad \text{where} \quad \frac{w_f}{D} = \frac{4}{(G/\tau_f)} \quad (1)$$

This format was selected because it closely matches the τ/τ_f against w/w_f load-transfer curve recommended in API (2011) and ISO 19901-4 (ISO, 2020), as shown in Fig. 2(a). The adoption of the normalised format implies that the operational shear stiffness of the sand mass adjacent to the pile is assumed to vary directly with the ultimate shaft friction (τ_f). In line with observations of load-transfer in sands, a post-peak strain-softening component is not required.

Base load-transfer function (q_b – w_b)

The base stress–base displacement (q_b – w_b) response is also dependent on the non-linear and non-uniform stiffness of the sand in the vicinity of the pile base operating after the disturbance due to the pile driving. As for shaft friction, a simple normalised format is adopted here, for which a unique dependence of $q_b/q_{b0.1}$ with w_b/D is assumed. Reliable base stiffness data for driven piles are sparse largely due to the need to re-zero strain gauges after installation due to zero shifts in the strain gauges caused by pile driving. Data on driven pipe piles are particularly sparse due to the need to employ a strain-gauged twin-walled pile to enable separation of the relative contributions of the base plug load, annular resistance and external friction (e.g. Han *et al.*, 2020).

The available base responses measured in tests included in the unified database are plotted in Fig. 3; all of these piles were closed-ended. It is seen that $q_b/q_{b0.1}$ against w_b/D variations are generally similar when w_b/D ratios exceed 1–2%. The best-estimate trend line is shown in Fig. 3, which is also constrained by the requirement for $q_b/q_{b0.1}$ to be unity at $w_b/D = 0.1$, begins from a $q_b/q_{b0.1}$ ratio equal to zero but gives a very stiff initial response, which can allow indirectly for the presence of residual base load, such as measured at Drammen. This trend line is described by the following hyperbolic equation:

$$\frac{w_b}{D} = 0.01 \left[\frac{q_b/q_{b0.1}}{1 - 0.9 (q_b/q_{b0.1})} \right] \quad (2)$$

Equation (2) is employed together with the equation for $q_{b0.1}$ given in Table 2 to represent the contribution of the base stiffness to the pile load–displacements curves. This equation implies that, in the same sand profile, the base stiffness of a large diameter open-ended pipe pile is only

Fig. 2. Normalised pile load-transfer curves for shaft and base

Fig. 3. Measured normalised base responses for closed-ended driven piles

about 25% of that of an equivalent closed-ended pipe pile (as the respective $q_{b0.1}$ values differ by a factor of 4). Some justification for a proportional relationship between base stiffness and $q_{b0.1}$ as well as the general format of equation (2) are provided by Gavin & Lehane (2007), although further studies are required.

It is seen in Fig. 2(b) that while equation (2) matches the ISO 19901-4 (ISO, 2020) and API (2011) recommendations for $q_b/q_{b0.1}$ up to 0.4, at larger base stresses, these recommendations predict greater base displacements than equation (2) – for example the w_b value at $q_b/q_{b0.1} = 0.8$ is 60% higher than that given by this equation. Analyses using the procedures outlined below showed that the difference between both formats has a negligible influence on calculated displacements at typical working loads ($Q \leq 0.5 Q_{ult}$) when a higher proportion of the resistance is provided in shaft friction.

CALIBRATED VALUES OF W_f/D FOR DATABASE PILES

The pile head load–displacement (Q – δ_h) responses exhibited by the database test piles were compared with those calculated using the RATZ program (Randolph, 2003). This program is one of many commercially available ‘load-transfer’ programs that represent the pile as a series of one-dimensional elastic elements and the soil as a series of non-linear τ – w (or t – z) curves along the pile shaft with a q_b – w_b curve at the pile base.

The piles were sub-divided into 20 elements, each with an associated τ – w spring. Values of q_c and hence τ_f were determined for each spring using the new unified CPT method (Table 2). The $q_{b0.1}$ values were also determined from the same CPT method to enable the q_b – w_b spring to be calculated. To ensure compliance with equation (2), an iterative approach for the base response was applied owing to limitations in the range of the q_b – w_b curves that can be input to RATZ. The only soil variable remaining required for prediction of a Q – δ_h response is the G/τ_f ratio ($=4D/w_f$) in equation (1). Two preliminary exercises assisted with establishing a correlation for this ratio:

- (a) An initial set of calculations conducted for the tension piles in the database indicated G/τ_f values ($=4D/w_f$) required to match pile displacements at 50% of the ultimate capacity reduced systematically as the q_c value increased and the vertical effective stress (σ'_v) increased. The trend may be anticipated considering the

proportional relationship between τ_f and q_c (ignoring the $\Delta\sigma'_{rd}$ component of resistance; see Table 2) and a dependence of the maximum in situ shear modulus in sands (G_{max}) on $q_c^{0.33}$ and $\sigma'_v^{0.33}$ (e.g. Schnaid *et al.*, 2004).

- (b) The pile database includes tension and compression tests on identical piles in the same soil at four sites (see Lehane *et al.*, 2017). These tests showed that development of maximum shaft friction (τ_f) in tension required displacements that were, on average, twice those required for development of τ_f in compression piles – that is the G/τ_f operating in compression is double the value for tension tests. This factor of 2 was incorporated in the determination of best-fit values of the G/τ_f ($=4D/w_f$) ratio for the full database.

Correlations were explored that would minimise the difference between the measured and calculated Q – δ_h responses of the database piles for loads that were less than 50% of the ultimate capacity (Q_{ult}) – that is within the typical working range. The following relationship for the normalised displacement to peak shear (w_f/D) was found to provide a good fit to a large majority of the database (where p_a is the atmospheric pressure = 100 kPa):

$$\frac{w_f}{D} = \frac{q_c^{0.5} \sigma'_v{}^{0.25}}{A p_a^{0.75}} \quad A = 1250 \text{ (compression); } A = 625 \text{ (tension)} \quad (3)$$

Sand relative density (D_r) varies with the normalised CPT end resistance, $q_{c1N} = q_c/(p_a \sigma'_v)^{0.5}$. Assuming q_{c1N} values of 70 and 220, which correspond approximately with sand relative densities (D_r) of 40 and 80%, respectively (e.g. see Jamiolkowski *et al.*, 2003), equation (3) leads to the variations of w_f/D as indicated in Fig. 4. As expected from equation (3), the w_f/D values in tension are seen to be twice those in compression. The values of w_f/D also vary significantly with the stress level and D_r value, approximately doubling with a two-fold increase in D_r and also increasing by a factor of 2 when the depth (or stress level) increases by 16.

The values of w_f/D shown in Fig. 4 for database piles tested in compression (with an average pile length of 20 m and hence mean σ'_v value of about 200 kPa) are generally in

Fig. 4. Variation of w_f/D with D_r , σ'_v and loading direction implied by equation (2)

line with those proposed by API (2011) which recommends a mean value of 0.01 for routine design purposes.

Comparison of measured and calculated load–displacement curves

The measured Q – δ_h values for a representative selection of pile tests from the database are compared in Fig. 5 with the corresponding responses determined by RATZ using equations (1)–(3) combined with the formulations from the unified CPT method (Table 2) and the structural axial stiffness of the test piles. As for the initial calibration, the soil

along each pile shaft was discretised into 20 soil horizons (i.e. springs) for which the average τ_f was first calculated.

It is seen in Fig. 5 that the measured and calculated responses are in close agreement and particularly so, when the ultimate capacity (as given by the unified CPT method) matches the measured capacity. The difference between measured pile head displacement (δ_{hm}) and calculated head displacement (δ_{hc}) is quantified in Fig. 6 at a pile head load of 50% of the ultimate capacity ($Q = 0.5 Q_{ult}$). It is seen that the error in prediction is generally less than 0.5% of the pile diameter (e.g. about 2.5 mm for a typical database pile with a diameter of 500 mm) for all closed-ended and open-ended piles tested in compression and tension.

Fig. 5. Measured and calculated load–displacement responses for 16 pile tests in the unified sand database; see Table 1 for notation of pile type

Fig. 6. Normalised differences between measured and calculated pile head displacements $(\delta_h - \delta_m)/D$ at $Q = 0.5Q_{ult}$ for driven piles in compression and tension

The calculated responses are compared with those predicted using the 'typical' w_f/D value of 0.01 recommended by API (2011) together with normalised $\tau-w$ and q_b-w_b curves shown in Fig. 2 (but noting equation (2) was adopted for the base response). This comparison shows that the API (2011) recommendation is generally reasonable for many compression database piles. However, the assumption leads to an under-estimate of pile settlement in denser sands and for longer piles and a significant under-estimate of pile displacement for almost all piles tested in tension. With adoption of a constant w_f/D of 0.01, the predicted pile head displacements of the database piles at $Q = 0.5Q_{ult}$ are, on average, 0.002D higher than measured for compression piles and 0.005D lower for tension piles. Although these differences may be acceptable for short onshore piles (typical of the database), Fig. 4 indicates that significant over-predictions of pile axial stiffness can be expected for typical long offshore piles in dense sand and hence the use of equation (3) is preferable to the assumption of a constant w_f/D value.

CONCLUSIONS

A high-quality database of static load tests was employed to determine a simple CPT-based method to estimate the load-displacement response of driven piles in sand. It is shown that the use of equations (1)–(3) combined with the ultimate

frictions and end bearing determined from the unified CPT method (Lehane *et al.*, 2020, ISO 19901-4 (ISO, 2020)) may be used in a standard load-transfer program (e.g. RATZ, Randolph, 2003) to predict displacements to an expected accuracy of better than 0.005 times the pile diameter. The analyses show that the normalised displacement at which peak shaft friction is mobilised increases with sand relative density and depth and, for tension piles, is approximately double the value required for a compression pile.

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