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New CPT methods for evaluation of the axial capacity of driven piles

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ABSTRACT: High costs associated with offshore foundation installations have provided strong impetus to the offshore energy sector in the search for more reliable design methods. This paper provides a summary of an Industry sponsored project that led to the development of new CPT-based design methods for the evaluation of the axial capacity of driven piles. Particular attention was given to the need for the new methods to be applicable to large diameter offshore piles given that many existing methods are derived by calibration with capacities measured in static pile load tests on smaller diameter onshore piles. The basic mechanisms supporting the general format of the expressions proposed for shaft friction and end bearing in sands and clays are described. It is shown how the new expressions, which are calibrated against a database of the most reliable load tests reported in the literature, lead to better predictions of capacity compared to other methods and can also satisfactorily predict the capacity of piles driven into deposits comprising interbedded layers of sand, silt and clay. Recommendations for the prediction of pile displacements at working loads using CPT data are also presented.

1 INTRODUCTION

The popularity of the CPT and the similarity between the mode of penetration of a cone and a driven pile have provided strong motivation in the search for direct correlations between the CPT q_t value and axial pile capacity. The best known of initial correlations proposed for pile shaft and base resistance in a range of soil types were developed about 40 years ago and include methods recommended by de Beer (1972), Schmertmann (1978), de Ruiter & Beringen (1979), Zhou et al. (1982), Bustamante & Gianeselli (1982) and Van Impe (1986).

These and other methods have since been assessed using specific databases of static load tests by workers such as Briaud & Tucker (1988), Niazi & Mayne (2013), Eslami et al. (2014), Hu et al. (2012) and Amirmojahedi & Abu-Farsakh (2019). These studies show that there is no consensus of opinion regarding the relative reliability of the methods with some methods providing good predictions for a given database but poor predictions for other databases. This lack of consensus is partly due to significant inconsistencies between the respective databases which arise due to (i) different definitions of ultimate axial capacity, (ii) contradictory interpretations of load test data, (iii) uncertainty in CPT q_t values at test pile locations, (iv) inclusion or exclusion of layered/mixed soil deposits and different pile types and (v) separate assumptions relating to the effects of time, loading direction, re-testing and loading rate. In addition, the formulations of most methods do not explicitly incorporate many characteristics of driven pile behaviour that have been well proven in experimental research programmes (discussed below).

The Offshore Energy sector recognized the advantages of a CPT-based method for axial capacity assessment but also understood the need to quantify the reliability of such a method using a database that had the backing of much of the profession. A Joint Industry Project (JIP) was set up in 2014 to achieve this objective. The JIP was managed by the Norwegian Geotechnical Institute (NGI) with support from the University of Western Australia (UWA) and was sponsored by Equinor, Ørsted, Lundin, ONGC, Petrobras and DNV GL. A 'Team of Experts' worked over a period of 3 years with guidance from the JIP steering committee to assess the suitability of about 600 pile load tests for inclusion in the database. A good CPT coverage close to pile tests was required and only first-time load tests on driven piles that were statically loaded to a displacement of 10% of the pile diameter were included. The final tests selected make up, what is referred to as, the 'Unified database' and comprises 71 test piles in silica sand and 49 pile tests in clay. Full details of the 'Unified database' and the steps followed in its compilation are provided in Lehane et al. (2017).

A subsequent phase of the JIP began in 2017 (sponsored by Equinor, Lundin Norway, Ørsted, ONGC, BP, Total, ExxonMobil, EnBW, EDF,

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Aramco, SSER and HDEC) with the aim of developing new 'Unified' methods for the prediction of the axial capacity of driven piles in sand and clay. This initiative was motivated by the desire of Industry to replace the 4 CPT-based methods for driven piles in sand included in the API (2011) recommendations with a single method. The term 'unified' is employed as the method was developed with input from the proponents of the 4 most current CPT-based sand methods used offshore. Although no CPT method for driven piles in clay and silt are currently provided in API/ISO documentation, the inclusion of such a method was seen to be an important step forward to reduce dependence on laboratory measurements of undrained strength required by the existing 'alpha' method.

The JIP was concluded successfully in 2021 and the methods developed will be incorporated in the next versions of the ISO-19901-4 and API recommendations.

This paper provides an overview of the new CPT methods with a focus on justification of the basis of the formulations employed. A full description of the methods is provided in Lehane et al. (2020, 2022) and in Nadim et al. (2021). The 'sand' and 'clay' formulations were calibrated using the Unified database and their application to interbedded deposits comprising sand, silt and clay layers is examined in this paper. Recommendations are also provided to assist prediction of the displacement of driven piles at working loads using CPT data.

2 CPT METHODS FOR CAPACITY OF DRIVEN PILES IN SAND

2.1 *Approaches for evaluation of shaft friction*

Traditional CPT methods relate the local ultimate friction (τ_f) directly to the cone resistance (q_t) via a single factor, β:

$$\tau_f = q_t/\beta \qquad (1)$$

Bustamante & Gianeselli (1982), for example, recommend β values for driven steel tubular piles of 120 and 200 in loose and dense sands respectively and propose limiting τ_f values in each of these deposits to respective maximum values of 35 kPa and 120 kPa. These and other similar recommendations reported in the literature ignore the following well-known characteristics of driven piles in sand observed from instrumented pile tests (Lehane et al. 2020):

i. Ultimate shaft friction (τ_f) varies is direct proportional to the tangent of the interface friction angle between the sand and pile shaft (tan δ) i.e. shaft friction is governed by Coulomb's law.
ii. τ_f developed in any soil horizon reduces with the distance of that soil horizon from the pile tip (*h*); this arises largely due to the progressive cycling of sand at any particular level as installation progresses.
iii. Smaller diameter piles generate larger frictions due to constrained dilation under shear at the interface.
iv. Open-ended pipe piles generate lower shaft frictions than closed-ended piles due to lower levels of displacement imparted to the sand mass.
v. Ageing effects lead to increases in shaft friction with time after pile installation.

These characteristics are not incorporated in Eqn. 1 and prompted the development of a new generation of CPT methods included in API (2011), namely those referred to as 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). While all of these newer methods do not include all of the characteristics listed above, each incorporates a direct proportional relationship between τ_f and q_c and a reduction of τ_f with distance from the pile tip (*h*). These dependencies were first clearly revealed in experiments conducted in 1989 with the Imperial College instrumented pile in medium dense sand at Labenne, France (Lehane 1992). Figure 1 plots bounds to a large number of equalised radial stress measurements (σ'_{rc}) obtained in these experiments and displays an obvious trend for the σ'_{rc}/q_c ratio to be constant in a given soil horizon and to reduce with increasing distance of that horizon above the pile tip (noting that there is a direct relationship between τ_f and σ'_{rc} via Coulomb's law).

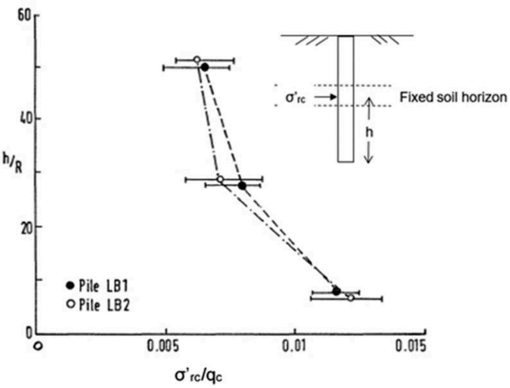

Figure 1. Dependence of equalised radial effective stress on q_c and normalised distance from the pile tip (*h/R*) at Labenne, France (Lehane 1992) [R=D/2 and τ_f varies directly with σ'_{rc}].

The trend shown on Figure 1, which has since been confirmed by Chow (1997) and others, is the key element of the shaft friction formulations of the

UWA-05, NGI-05, ICP-05 and Fugro-05 methods. These shaft frictions correspond to frictions that can be developed in the 2 to 4 week period following driving as the methods' calibrations employed static load test data recorded in the same period.

2.2 *Approaches for evaluation of base resistance*

Given the comparable modes of penetration, all historical CPT methods for closed-ended driven piles in sand relate the ultimate end bearing directly to the q_c value in the vicinity of the pile tip. However, the ultimate end bearing stress, defined at a displacement of 0.1D ($q_{b0.1}$), is typically only about 50% of q_c for a closed-ended piles driven in homogeneous sand deposits because a displacement of the pile tip of order of 1D is required to reach steady state penetration conditions (as exist during cone penetration). In addition to partial mobilisation at a displacement of 0.1D, Van Mierlo & Koppejan (1952) recognised the importance of the scale difference between a pile and a penetrometer. Recognition of this effect subsequently led to a variety of proposals for q_c averaging techniques, the most popular of which relates $q_{b0.1}$ to a simple average value of q_c values in a zone extending 1.5D above and 1.5D below a pile tip.

Compared with closed-ended piles, pipe piles induce lower levels of displacement (or disturbance) to the sand near their bases during installation. The degree of partial plugging (and hence disturbance) reduces as the pile diameter increases. Consequently, as shown by Gavin & Lehane (2003), the end bearing of large diameter piles under static loading (when the plug remains stationary) reduces to that of a bored pile for which $q_{b0.1}$ is typically about 15% of the q_c value (Lehane & Randolph 2002). This diameter dependence is incorporated in a number of different ways in the Fugro-05, ICP-05, NGI-05 and UWA-05 correlations for $q_{b0.1}$. No set-up of end bearing resistance has been observed for driven piles in sand.

2.3 *Performance of CPT methods in API (2011)*

The Fugro-05, ICP-05, NGI-05 and UWA-05 CPT methods included in API (2011) were calibrated using databases compiled specifically for the development of these methods. It is therefore of interest to examine their performance against the 'Unified database', which is fully supported by all researchers involved in the four methods. The findings from this exercise are summarised in Table 1, which presents statistics for ratios of measured to calculated capacities (Q_m/Q_c) for the four methods as well as for the earth pressure theory method in API RP2A (2011) and the new 'Unified method' (discussed below).

The results in Table 1 show that, while the earth pressure approach in API over-predicts pile capacities by an average of 66%, the mean Q_m/Q_c ratio of measured to calculated capacity for the four API CPT methods is close to unity. The spread in predictions for the earth pressure approach, as measured by the CoV for Q_m/Q_c, is also higher than for the CPT methods and is indicative of a significantly lower level of reliability. The UWA-05 and ICP-05 methods are the best performing methods with the lowest CoVs.

Table 1. Mean (μ) and coefficient of variation (CoV) of Q_m/Q_c for the Unified database of driven piles in silica sand.

	All open & closed-ended piles (total capacity) 71 Piles	
Method	μ	CoV
API (2011), K tan δ (β) approach	1.66	0.56
Fugro-05	0.99	0.40
ICP-05	1.04	0.27
NGI-05	0.99	0.34
UWA-05	1.06	0.26
Unified Method	1.05	0.24

3 UNIFIED CPT METHOD FOR DRIVEN PILES IN SAND

While the UWA-05 and ICP-05 methods performed relatively well against the 'Unified database', a primary aim of the JIP was to develop a unified and un-affiliated method that had the support of those involved in the derivation of the API β method and the four API CPT methods. The first point of agreement was that, in line with the findings of Lehane et al. (1993), and others, the following expression should form the basis of the correlation for peak shaft (τ_f):

$$\tau_f = (\sigma'_{rc} + \Delta\sigma'_{rd}) \tan \delta_f \qquad (2)$$

where σ'_{rc} is the stationary (equalised) radial effective stress, $\Delta\sigma'_{rd}$ is the increase in radial effective stress during pile loading (attributed to dilation) and δ_f is the constant volume sand-pile interface friction angle. On review of the state-of-the-art, it was agreed that the Unified method should incorporate considerations described in the following.

3.1 *Interface friction angle (δ_f)*

In the absence of site-specific tests to measure ultimate interface friction angles (δ_f), the ICP-05 and UWA-05 methods propose the variation with the mean effective particle size (d_{50}) indicated by the curve shown in Figure 2. This variation was deduced from direct shear box tests on steel interfaces with a roughness typical of industrial piles but that did not include preshearing to large relative displacements. Yang et al.

(2010), and others, have since shown that crushing of sand at the pile tip and subsequent shearing during installation reduces the grading to that of a fine sand. Interface shear angles measured in the Bishop ring shear apparatus in tests that induced a large level of pre-shearing are plotted on Figure 2 confirmed the relatively low sensitivity of δ_f to the initial d_{50} value. Similar tests reported by Liu et al. (2019) also show that δ_f has virtually no dependence on the (non-plastic) fines content of typical siliceous sands and on the normal stress level. It was therefore concluded that, in the absence of site specific ring shear interface tests, adoption of a constant δ_f value of 29° is a reasonable assumption for all piles in the load test database.

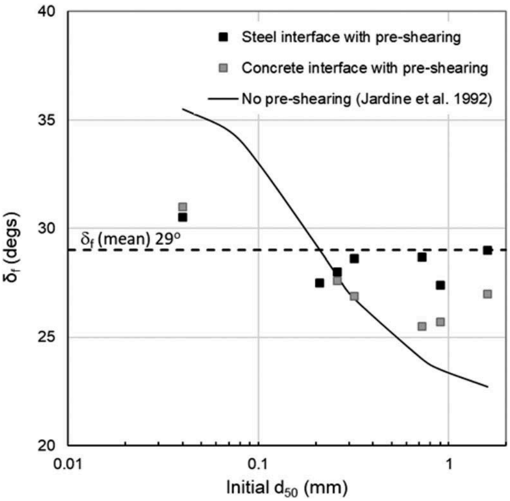

Figure 2. Interface friction angle and d_{50} for roughness typical of industrial piles (centreline average roughness of 5 to 20μm) (Barmpopoulos & Ho, 2009); Ho et al., 2011).

3.2 *Increase in radial stress ($\Delta\sigma'_{rd}$) during pile loading*

The restraint to dilation at the pile shaft during pile loading provided by the surrounding sand leads to an increase in radial stress on the pile shaft ($\Delta\sigma'_{rd}$) and hence to the peak shaft friction. This increase can be assessed from cavity expansion (CE) theory, where G is the operational shear modulus of the sand mass, y is the dilation of the sand at the shaft interface and $y/2D$ is the cavity strain:

$$\Delta\sigma'_{rd} = 4Gy/D \tag{3}$$

The UWA-05 and ICP-05 methods assume that the cavity strain is small enough for conditions to be fully elastic and hence the methods equate G with the small-strain elastic value (G_0). However, data from constant normal stiffness direct shear interface tests and tests on centrifuge piles with a range of diameters presented in Lehane et al. (2005) show that the cavity strains can be relatively large and that the operational G value is less than G_0 for typical pile diameters in the 'Unified database'. The following revised approximate expression was deduced using $\Delta\sigma'_{rd}$ measurements on jacked piles and parallel numerical analyses (Lehane et al. 2020):

$$\Delta\sigma'_{rd} = \left(\frac{q_c}{10}\right)\left(\frac{q_c}{\sigma'_v}\right)^{-0.33}\left(\frac{d_{CPT}}{D}\right) \tag{4}$$

with d_{CPT} =35.7mm, which is the usual CPT diameter.

Most piles in the Unified database have a diameter (D) between 350mm and 800mm. Equation (4) predicts that the increase in peak friction due to dilation (=$\Delta\sigma'_{rd}$ tanδ_f) for a 20m long pile in medium dense sand is about 35% for D=350mm but only 10% for D=800mm. The relative influence of dilation is clearly an important consideration when extrapolating from the smaller diameter piles in the database to larger diameter offshore piles. The relative influence of dilation is also greatest in looser sands and for longer piles.

3.3 *Allowance for partial plugging*

Many of the pipe piles in the Unified database (with D= 550 ± 250mm) experienced partial plugging during pile driving. The additional shaft and base capacity that such plugging induces (e.g. see Gavin & Lehane 2003) needs to be accounted for correctly in the database analysis to ensure safe extrapolation to the capacity of full scale offshore piles.

White et al. (2005) used a cavity expansion analogy to deduce that the equalised lateral effective stress acting on the pile shaft (σ'_{rc}) varies with the effective area ratio (A_{re}) raised to a power of between about 0.3 and 0.45 while Xu et al. (2008) present experimental data showing how $q_{b0.1}$ varies in proportion to A_{re}.

A_{re} provides a measure of the level of soil displacement in any given soil horizon, and is defined as:

$$A_{re} = 1 - IFR(D_i/D)^2 \tag{5}$$

where IFR is ratio of the change in plug length to change in pile embedment and D_i is the internal pile diameter. A_{re} varies from unity for a closed-ended or fully plugged pile to a value of $4t/D$ for a large diameter coring pile (where t is the pile wall thickness). As the IFR is not measured routinely, the analysis of the unified database substituted the plug length ratio (PLR) for IFR, noting that the PLR is the average IFR during pile driving and equal to the ratio of the final plug length to the embedded pile length.

The PLR is primarily a function of the internal pile diameter (D_i) (e.g. see Gudavalli et al. 2013) and the following approximate expression was derived based on available records (d_{CPT} =35.7mm):

$$PLR \approx IFR = \tanh\left[0.3\left(\frac{D_i}{d_{CPT}}\right)^{0.5}\right] \qquad (6)$$

The database analysis examined the influence of various exponents of A_{re} in the search for a best-fit expression for τ_f.

3.4 *Time effects*

The shaft capacity of driven piles in silica sand increases with time over a period of least one year (e.g. Chow et al. 1998, Jardine et al. 2006, Karlsrud et al. 2014, Gavin et al. 2015). Such increases are not exhibited by bored piles and may be viewed as a recovery process following the 'trauma' of driven pile installation (Lim and Lehane 2014, Anusic et al. 2019). The new CPT method is calibrated using the Unified database comprising static load tests with a median equalisation period (or set-up time) of about two weeks. It is therefore likely to underestimate long term capacities and over-estimate short term capacities (including driving resistance).

3.5 *Formulation for σ'_{rc}*

The observed reduction in the equalised radial effective stress (σ'_{rc}) with the distance from the pile tip (*h*) or normalised distance from the pile tip (*h/D*) can be described as a power law relationship (as in the ICP-05 and UWA-05 methods) or as an exponential relationship (proposed by Randolph et al. 1994 and Salgado et al. 2011). Lehane et al. (2020) show that the bias in the database, with respect to diameter, of the ratio of measured to calculated capacities (Q_m/Q_c) can be removed when a *h/D* term is used instead of '*h*' (as proposed by Alm & Hamre 2001). Lehane et al. (2020) also show that while the power law and exponential variations with h/D lead to very similar statistics for the Q_m/Q_c ratios, the exponential relationship tended to predict marginally larger capacities for piles with L/D<20 compared with the power law form. The power law form was retained as many piles used in the offshore wind Industry have lower L/D values.

3.6 *Tension compression ratio*

The database piles clearly showed that shaft friction measured in tension are, on average, about 75% of the shaft friction of compression piles. Limited data exist in relation to differences in the distribution of τ_f along test piles and, in the absence of other information, the global (best-fit) tension to compression ratio (f_L) of 0.75 was employed in the new method.

3.7 *Formulation for end bearing*

The Unified method relates the ultimate end bearing ($q_{b0.1}$) with the effective area ratio (A_{re}) for pipe piles in line with observations of Gavin & Lehane (2003) and as adopted in the UWA-05 method. The end bearing formulation of ICP-05 incorporates this dependency indirectly by allowing $q_{b0.1}$ for pipe piles to reduce with pile diameter.

The Unified method relates $q_{b0.1}$ with q_p, where q_p is the end bearing resistance expected for an 'imaginary cone' that has the same diameter as the pile being considered (or equivalent diameter for a pipe pile= $(A_{re}/\pi)^{0.5}$). The value of q_p is determined using a component of a 'thin-layer' procedure described by Boulanger and DeJong (2018) and its application in piling calculations is explained by Bittar et al. (2020). Although q_p is a rational and objective way of obtaining an averaged cone resistance near a pile base of given diameter, its determination requires use of software (freely downloadable from *https://faculty.engineering.ucdavis.edu/boulanger/research-interests/*) which is not appealing to many practitioners. The new ISO-19901-4 recommendations therefore suggest taking q_p as the average q_c value in the zone extending 1.5D above and below the pile tip, unless conditions at the pile tip are highly variable. Separate studies show that q_p for the Unified database piles is, on average, 20% higher than q_{Dutch}, where q_{Dutch} is the CPT resistance determined using the 'Dutch' averaging technique (Van Mierlo & Koppejan 1952)

3.8 *Formulation of Unified CPT method in sand and predictive performance*

The final formulations decided upon for the Unified CPT method for piles in sand (Zone 6 of the soil behaviour type chart) are provided in Table 2. When these are applied to the Unified database, the average ratio of measured to calculated capacities (Q_m/Q_c) is 1.05 while the CoV for Q_m/Q_c is 0.24. These statistics are a marginal improvement on those for the UWA-05 and ICP-05 methods (see Table 1). The method is, however, considered to be a significant step forward as it replaces the four API CPT methods with full support of the authors for these methods and it incorporates state-of-the-art understanding of the mechanisms controlling the axial capacity of driven piles in sand. It should be noted that the method potentially under-estimates axial capacity in gravelly sands (Zone 7 of the SBT chart), where the presence of gravels leads to higher average q_c values. The method is only applicable to piles driven in a conventional manner and should not be used for jacked piles or piles installed by vibration.

Table 2. General formulations for Unified method in sands, silts and clays *(see https://pile-capacity-uwa.com/).*

$Q_{shaft} = \pi D \int_0^L \tau_f \, dz$
$Q_{base} = q_{b0.1} (\pi D^2/4)$

Capacity estimate for piles for flexible piles in strain-softening clays requires load-transfer analysis using load transfer curves given in API (2011)

Sands: Zone 6 of SBT chart (I_c< 2.1)

$\tau_f = f_L (\sigma'_{rc} + \Delta\sigma'_{rd}) \tan 29°$
$q_{b0.1} = [0.12 + 0.38A_{re}]q_p$; for plugged base (expected when L/D>5)
$q_{b0.1} = A_{re} \, q_p$; for unplugged base
$\sigma'_{rc} = (q_c/44) \, A_{re}^{0.3} \, [Max[1, h/D]]^{-0.4}$
$\Delta\sigma'_{rd} = (q_c/10) \, (q_c/\sigma'_v)^{-0.33} \, (d_{CPT}/D)$
$A_{re} = 1 - PLR \, (D_i/D)^2$
$PLR \simeq \tanh [0.3 \, (D_i/d_{CPT})^{0.5}]$
f_L= 0.75 in tension, 1.0 in compression
q_p can be taken as the average q_c within a zone 1.5D above and below the pile tip or determined using the procedure described Boulanger & De Jong (12018) and Bittar et al. (2020)

Clays: Zones 1,2,3 & 4 of SBT chart

$\tau_f = 0.07 \, F_{st} \, q_t \, [Max[1, h/D^*]]^{-0.25}$
$q_{b0.1} = [0.2 + 0.6A_{re}]q_p$

F_{st} =1 for clays with I_{z1}>0, in Zones 2, 3 and 4 on the SBT Chart ($I_c \geq 2.6$)
F_{st} =0.5±0.2 clays with I_{z1}<0, in Zone 1 on the SBT Chart
$D^* = (D^2 - D_i^2)^{0.5}$ for an open-ended pile and D^*=D for a closed-ended pile
$I_{z1} = Q_{tn} - 12exp^{(-1.4F_r)}$
q_p =average q_t value in zone between the pile tip and 1D below the pile tip (closed-ended/ plugged pile) or average q_t to a depth of 20*t* below the pile tip (large diameter, unplugged pile)

Silts: Zone 5 of the SBT chart (2.1< I_c <2.6)

Apply equations as for Zone 6 using corrected q_c value determined as:
$q_c = [3.93 \, I_c^2 - 14.78 \, I_c + 14.78] \, q_t$

Notation

h	*L – z,* where *L* is pile embedment length and *z* is the depth)
D	Pile outer diameter
D_i	Internal diameter of a pipe pile
f_s	Cone sleeve friction
F_r	Friction ratio (expressed as a percentage)=$f_s/(q_t-\sigma_{v0})$
q_t	Total (corrected) CPT end resistance (=q_c in sands)
Q_{tn}	Normalised cone resistance $[(q_t-\sigma_{v0})/p_a]/(p_a/\sigma'_{vo})^n$; p_a=ref. stress=100 kPa
I_c	Consistency index, function of Q_{tn} and F_r; see Robertson (2009)
n	Stress exponent for Q_{tn}, function of I_c and σ'_{v0}; see Robertson (2009)
t	Pile wall thickness
SBT	Soil Behaviour type; see Robertson (2009)
σ_{v0}	Total vertical stress
σ'_{v0}	Effective vertical stress

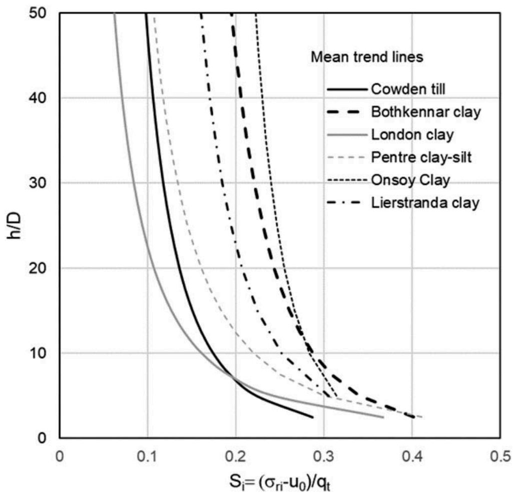

Figure 3. Mean recorded variations of normalised radial total stresses (S_i) with h/D measured during installation of closed-ended instrumented piles.

4 CPT METHODS FOR CAPACITY OF DRIVEN PILES IN CLAY

4.1 *Shaft friction*

Relationships between shaft friction and the measured and corrected cone resistances (q_c and q_t) for driven piles in clay have been proposed for many years e.g. Bustamante & Gianeselli (1982), Almeida et al. (1996), Lehane et al. (2000, 2013), Eslami & Fellenius (1997) and Niazi & Mayne (2016). The newly proposed Unified CPT method for clays builds on these methods but is based primarily on observations made in high quality instrumented pile experiments conducted by a number of institutions, most notably at the Norwegian Geotechnical Institute and Imperial College London.

The basis of the correlation between shaft friction and the (corrected) CPT end resistance (q_t) for the new Unified method is the observation that, in any particular clay, the radial total stress developed during installation (σ_{ri}) on the shaft of a closed-ended displacement pile varies directly with q_t and reduces with the normalised distance from the pile tip (h/D). Examples of observed trends are shown on Figure 3, which plots the mean measured variation of the normalised radial total stress in six different clays against h/D, where the normalised radial total stress (S_i) is defined as follows and u_0 is the hydrostatic or ambient pore pressure:

$$S_i = (\sigma_{ri} - u_0)/q_t \tag{7}$$

The mean trend lines indicated on Figure 3 have a typical standard deviation of 25% and are evidently dependent on the clay type. Following installation, instrumentation on driven piles shows that radial total stresses fall as excess pore pressure dissipate while radial effective stresses (σ'_r) increase. σ'_r reaches a fully equalised value (σ'_{rc}) after equalisation of radial total stresses and full pore pressure dissipation. When a pile is loaded to failure after equalisation, radial effective stresses reduce attaining a value of σ'_{rf} at peak local shear stress (τ_f).

These stages in the life of a driven pile are incorporated in the following expression for τ_f, which is based on Coulomb's friction law:

$$\tau_f = \sigma'_{rf}\tan\delta = q_t S_i (S_c/S_i)\ f_l \tan\delta_f \tag{8a}$$

where

$$S_c = (\sigma_{rc} - u_0)/q_t = \sigma'_{rc}/q_t \tag{8b}$$

$$f_l = \sigma'_{rf}/\sigma'_{rc} \tag{8c}$$

Equation (8) illustrates that the relationship between τ_f and q_t depends on the degree of relaxation of radial total stresses during equalisation (expressed by S_c/S_i), the relative change in σ'_r during load testing (f_l) and the interface friction angle between the pile and clay (δ_f). Numerical simulations such as those of Whittle & Baligh (1988), as well as experiments, indicate that S_c/S_i ratios are lower in sensitive and low OCR clays where installation causes a significant degree of remoulding and reduction in effective stress. The S_c/S_i values measured experimentally range from 0.15 in (highly sensitive) Lierstranda clay to 1.0 in the heavily overconsolidated London clay. Measured values of f_l are in the range of 0.8 to 1.0 in all clays but average at about 0.6 in Lierstranda clay. The operational coefficient of friction ($\tan\delta_f$) is strongly affected by the clay mineralogy, amongst other factors, and for the six clays considered in Figure 3 varies by more than a factor of 2 (from a $\tan\delta_f$ value of 0.23 in London Clay to 0.55 in Bothkennar clay).

The compounding effects of differences in respective clays of the S_c/S_i, f_l and $\tan\delta_f$ values as well as the S_i relationship with h/D (shown on Figure 3) reveal a complex relationship between τ_f and q_t. Nevertheless, when ratios of τ_f/q_t with h/D measured in instrumented pile tests are plotted, as shown in Figure 4a, it is evident that a number of compensatory factors lead to a relatively consistent relationship between τ_f/q_t and h/D in all of clays apart from Lierstranda, where the product of S_c/S_i and f_l is, on average, about four times less than the other clays. Best-fit mean trend lines for the instrumented data included in Figure 4a are presented on Figure 4b and, as shown later, are consistent with best-fit expressions for the capacities of the piles in the Unified database.

4.2 *End bearing*

The contribution of end bearing to the capacity of driven piles in clay is relatively small and therefore has not been the subject of extensive research. The assumption of a simple direct relationship between $q_{b0.1}$ and the average q_t value near the pile tip is considered adequate for practical purposes. On the basis of a review of existing correlations, Lehane et al. (2022a) propose the following relationships for typical onshore driven piles (D <0.75m)

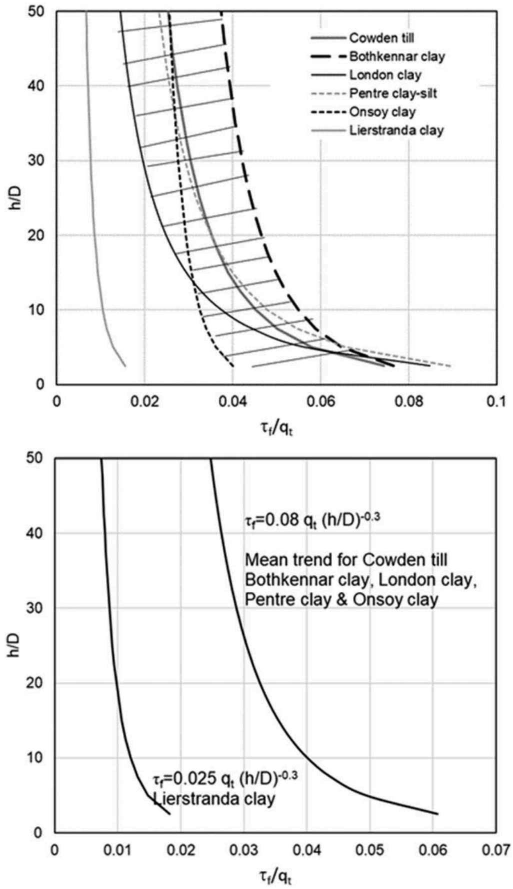

Figure 4. (a) Average measured variations of τ_f/q_t with h/D (showing envelope for all clays except Lierstranda clay), (b) best-fit mean trend lines.

$$q_{b0.1} = 0.8 q_t (\text{closed-ended pile}) \tag{9a}$$

$$q_{b0.1} = 0.4 q_t (\text{open-ended pile}) \tag{9b}$$

5 UNIFIED CPT METHOD DRIVEN PILES IN CLAY

The development of the new Unified CPT method for clays recognised the compensatory effects of factors evident on Figure 5a but investigated potential systematic dependence on (i) clay plasticity index, (ii) soil behaviour type (SBT), (iii) CPT sleeve friction (f_s) and

(iv) vertical effective stress level (σ'_v). The calibration of the method with the Unified database also included consideration of the aspects detailed below.

5.1 Dependence of τ_f on loading direction

Compression and tension tests on piles with identical configurations in seven different clays in the Unified database indicate that, in general, there is not a clear dependence of ultimate shaft friction on the load direction. A load direction factor (f_L) of unity was found to provide a best fit to the full Unified database.

5.2 *Friction on open-ended piles*

Instrumented pile test data reported by Doherty & Gavin (2011) show that lower levels of soil displacement during installation of open-ended piles lead to lower installation radial total stresses (σ_{ri}) and lower ultimate shaft frictions after equalisation. There is, however, a considerable shortage of such data in different clays and, in the absence of such data, the regression analyses conducted for the Unified method assumed the following potential dependencies (where Equation 10a is comparable to the Unified CPT sand method and Equation 10b is a similar format to the ICP-05 method for sands):

$$\tau_f = f(A_{re}, h/D) \quad (10a)$$

$$\tau_f = f(h/D*); D* = \left(D^2 - D_i^2\right)^{0.5} \quad (10b)$$

Lehane et al. (2017) show that Equations 5 & 6, which were derived from data for piles in sand, may also be used to approximate A_{re} values for pipe piles in clays.

5.3 *Progressive failure*

A number of piles in the clay database exhibit post-peak softening of shaft shear stress. The calibration of the Unified method assumed that peak shear stresses corresponded to the measured peak axial capacity or, if such a peak was not observed, to the axial resistance at a displacement of 0.1D. This approach is considered reasonable for the majority of the database (which comprised relatively rigid piles) but is moderately conservative for a few very long piles in clays where strain softening can be significant.

5.4 *Silt layers*

Occasional silt layers with a soil behaviour type (SBT) index, I_c, in the range 2.05 to 2.6 occur within the clay strata of the database. For these deposits, the calibration process employed the Unified sand method adopting the equivalent clean sand q_t value ($q_{t,sand}$) for the silt. This was derived using the following relationship which is equivalent to the proposal of Robertson & Wride (1998) but adapted to a simplified format and modified to give a correction factor of unity at $I_c = 2.05$:

$$q_{t,sand} = q_c = \left[3.93I_c^2 - 14.78I_c + 14.78\right]q_t \text{ for } 2.05 < I_c < 2.6 \quad (11)$$

5.5 *End bearing of large diameter pipe piles*

Eqn. 9 was employed in the calibration of the relatively small diameter piles in the Unified database. For large diameter offshore piles, the following expression is considered more appropriate; this was based on findings of Doherty and Gavin (2011) from measurements of twin-walled instrumented piles during installation.

$$q_{b0.1} = [0.2 + 0.6A_{re}]\ q_t \quad (12)$$

where q_t is the average corrected cone resistance value in the zone extending from the pile tip to a depth of 20*t* below the pile tip (*t* = pile wall thickness)

5.6 *Formulation of Unified CPT method in clay and predictive performance*

The formulations for the Unified clay method are provided in Table 2. These were established following various optimisation analyses which revealed that, as inferred from instrumentation pile test data, τ_f is primarily correlated to the CPT q_t value and the normalised distance from the pile tip (h/D*). The analyses also showed that the shaft friction generated in high sensitivity clays (within Zone 1 of the SBT chart) is typically 50% of the shaft friction in other clays (although there is significant variability). The formulations developed are remarkably similar to those deduced independently from instrumented pile test data (compare formulae in Figure 5 with those in Table 2). Surprisingly, the fit to the capacities of the database piles was not improved by consideration of additional factors such as cone sleeve friction, plasticity index and overconsolidation ratio (inferred by examination of a q_{net}/σ'_{v0} term).

The predictive performance for the Unified clay database of the new clay method was compared by Lehane et al. (2022a) with a number of other current methods. This comparison is summarised in Table 3, which also lists the main input parameters in the τ_f correlation for each method (noting that differences in the end bearing formulations had little impact on calculated capacities). Evidently the CoV of Q_m/Q_c for the new method is a good improvement on existing methods and is significantly lower than the corresponding CoV for the existing '$\alpha - s_u$' method recommended by API/ISO. It should be noted, however, that part of the reason for the substantial reduction in the CoV with respect to other methods arises due to application of a separate equation for Zone 1 (sensitive) clays.

Table 3. Comparison of predictive performance for driven piles in clay of the Unified method with other methods (in terms of mean and CoV of measured to calculated capacity ratios).

Method	Parameters controlling τ_f	Mean Q_m/Q_c	CoV for Q_m/Q_c
API (2011)	s_u & s_u/σ'_v	1.05	0.43
Fugro-96 (Kolk & van der Velde 1996)	s_u, h/D & s_u/σ'_v	1.04	0.35
ICP-05 (Jardine et al. 2005)	OCR, σ'_v, h/D*, δ & S_t	1.12	0.55
NGI-05 (Karlsrud et al. 2005)	s_u, s_u/σ'_v, I_p & σ'_v	1.1	0.36
UWA-13 (Lehane et al. 2013)	q_t & h/D*	1.12	0.33
Fugro-10 (Van Dijk & Kolk 2010)	q_{net}, h & q_{net}/σ'_v	0.98	0.37
Unified method (Lehane et al. 2022a)	q_t, h/D* & S_t	0.99	0.23

The Unified method in clay is calibrated on a relatively small number of pile tests (49) and, as may be inferred from the instrumented pile test data, its relatively good predictive performance arises partly because of compensating factors. For example, interface friction angles (δ) for clays in the Unified database range from 12° to 30°, implying a range in capacities of about 2.5 if τ_f varied only with tan δ. However, encouragingly, Lehane et al. (2022a) examined the performance of the new method against a different 'Test database' comprising 24 pile tests in clay and found a mean Q_m/Q_c ratio of 1.09 (i.e. a 9% over-prediction, on average) and a CoV for Q_m/Q_c of 0.22; this CoV is a substantial improvement on the CoV of 0.43 determined for the API $\alpha - s_u$ method (Table 3).

6 APPLICATION TO LAYERED DEPOSITS

The Unified CPT methods for sand and clay were calibrated against test data for piles that were driven into predominately sand or clay deposits. However, many deposits encountered in practice contain sand, silt and clay layers and it is therefore important to assess the reliability of the methods at such sites. Bittar et al. (2022) conducted such a study involving 23 load tests on piles driven into mixed stratigraphies. Ten of these load tests were on piles with diameters between 0.9m and 2m in diameter which is considerably larger than the mean D value of about 0.4m for the piles in the Unified database. An average of 55% of the layers in these case histories were coarse grained and two cases involved sensitive zone 1 soils.

Sample predictions for two driven steel pipe pile case histories examined are presented in Figure 6a for a tension test in Oakland California on a 13.3m long, 610mm pile and in Figure 6b for a compression test in Minnevika, Norway on a 40m long, 405mm diameter pile. As seen with reference to the plotted I_c profiles in these figures, a 5.5m thick layer of Zone 3 clay is present along the central portion of the pile shaft whereas the stratigraphy at Minnevika comprises alternating 1.5m to 2m thick layers of sand and clay. The calculated τ_f distributions were determined in a simple spreadsheet using the I_c dependent expressions, as summarized in Table 2; calculations can also be performed using UWA freeware at *https://pile-capacity-uwa.com/*. These τ_f distributions lead to ratios of measured to calculated capacities (Q_m/Q_c) of 1.15 at Oakland and 1.16 at Minnevika (ignoring potential strain softening).

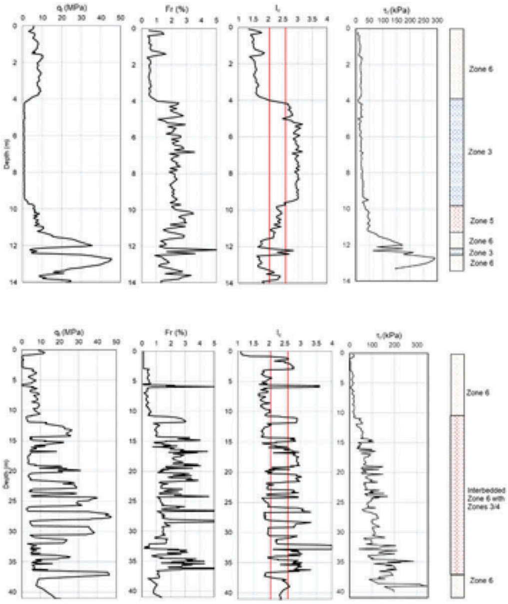

Figure 5. Stratigraphy and predicted τ_f distribution for (a) tension test on 610-mm diameter, 13.3m long pipe pile in Oakland and (b) compression tests on 405-mm diameter, 40m long pipe pile at Minneveka.

Bittar et al. (2022) show that the mean and coefficient of variation of the Q_m/Q_c values for the 23 load test database were 1.02 and 0.17 respectively i.e. the spread in predictions is less than that of the Unified databases for sand and clay sites. It can therefore be concluded that the Unified methods can be applied with the same level of confidence to layered stratigraphies as to 'single soil type' deposits. However well instrumented test piles that accurately measure the distribution of shaft friction in layered stratigraphies are needed for verification of the method. There is also a great shortage of skin friction data for piles installed in in Zone 5 (silt) of the SBT chart. Such data would help to resolve the discontinuity in the expressions for τ_f at the boundary between Zone 4 and 5 (at I_c=2.6), where τ_f calculated using the clay expression is typically double that calculated using the clean sand correction approach for silt.

7 LOAD DISPLACEMENT RESPONSE USING THE UNIFIED CPT METHOD

The load-displacement response of piles is normally predicted in a load transfer analysis where the shear stress-displacement (τ-w) springs (also called t-z springs) at various levels along the pile shaft are scaled in proportion to the ultimate shaft friction (τ_f) and the base spring (q_b-w_b) is scaled with the ultimate end bearing ($q_{b0.1}$). The application of the new Unified CPT method to this approach is examined in the following.

7.1 *Piles in sand*

Lehane et al. (2020b) examined the accuracy of pile displacement predictions for driven piles in the Unified sand pile database sand using the τ-w and q_b – w_b load transfer relationships recommended in API (2011). These relationships are provided in tabular form in API (2011) but may be expressed as follows, where w_f is the displacement to peak shear stress (τ_f) and is assigned a mean value of 0.01D in API (2011):

$$\frac{\tau}{\tau_f} = 2\frac{w}{w_f}\left[1 - \frac{w}{2w_f}\right] \tag{13}$$

$$\frac{w_b}{D} = 0.01\left[\frac{q_b/q_{b0.1}}{1 - 0.9\left(q_b/q_{b0.1}\right)}\right] \tag{14}$$

Although Equation (14) is not a perfect match to the API tabulated values when $q_b/q_{b0.1}>0.5$, this is not a concern as settlement predictions are required for the serviceability limit state when applied loads are rarely greater than 70% of the pile capacity.

API (2011) recommends adoption of a mean w_f/D value of 0.01. However the analyses showed that use of a constant value did not capture the tendency for softer pile responses in tension compared with compression and for stiffer responses in denser sands and longer piles. On examination of the trends shown by the piles in the Unified database, Lehane et al. (2020b) proposed the following expression for w_f/D:

$$\frac{w_f}{D} = \frac{{q_c}^{0.5}{\sigma'_v}^{0.25}}{Ap_a^{0.75}} \tag{15}$$

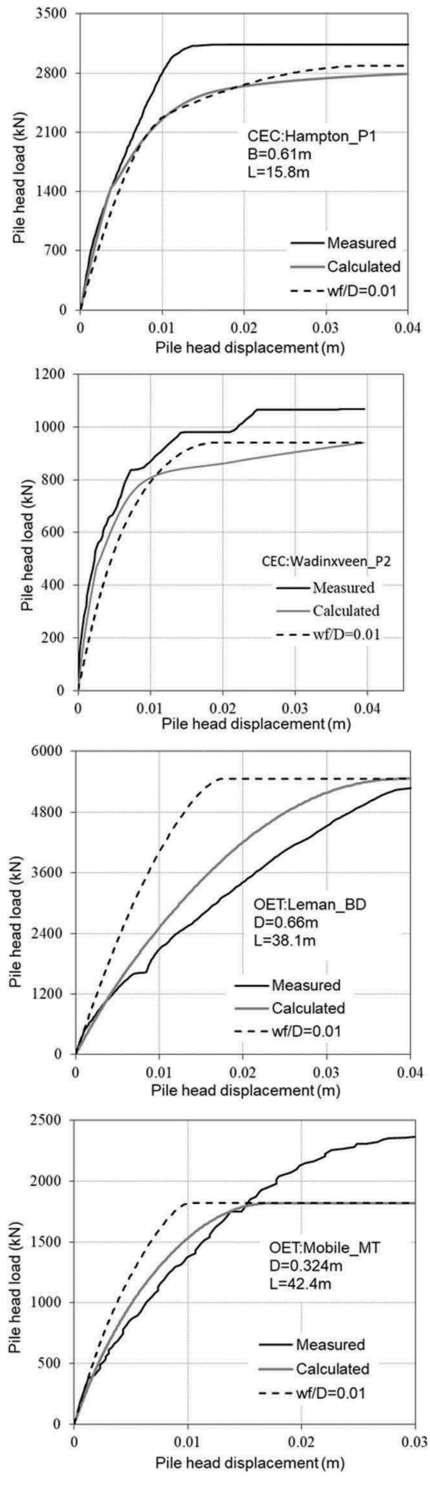

Figure 6. Comparison of measured pile load-displacement response with the calculated response using the Unified method with (i) w_f/D given by Equation 15 and (ii) w_f/D=0.01. The symbols CEC and OET denote closed ended piles tested in compression and open-ended piles tested in tension respectively.

where the constant A is 1250 in compression and 625 in tension.

Comparisons of measured and calculated load-displacement responses are plotted for typical cases from the Unified database on Figure 7, where calculations were performed using Equations 13 and 14 combined with the Unified method to determine τ_f and $q_{b0.1}$. Figure 7 presents calculated responses for a constant w_f/D value of 0.01 and for w_f/D values at various levels along the pile shaft determined using Equation 15. The improved fit obtained using Equation 15 for tension piles and for longer piles is evident on Figure 7. Lehane et al. (2020b) show that the error in calculated displacements at 50% of the ultimate capacity is typically about 0.002D and is less than 0.005D. Equation 15 will be included in the next version of the API/ISO recommendations.

7.2 *Piles in clay*

Lehane & Bittar (2022) repeated the exercise described above for the Unified database of piles in clay, for which API (2011) recommends the same format of load transfer curves apart from the allowance for post-peak softening to a shear stress of (0.8 ±0.1) τ_f at a displacement of 0.02D. It was found that a combination of the Unified CPT method in clay (Table 2) with these load transfer curves provides good predictions of the displacements of both the tension and compression piles in the database. The standard deviation of the difference between measured and calculated displacements was 0.002D.

8 CONCLUSIONS

There are clear advantages to CPT-based design approaches for the assessment of the axial capacity of driven piles. The CPT-based methods have the full support of the geotechnical community as they remove subjectivity associated with parameter assessment, provide essentially continuous profiles of soil behaviour and allow direct rapid and automated calculation of capacity at any CPT location.

There has been a concerted effort over the past decade to first quantify and then increase the reliability of design methods for piles. This was made possible by the thorough, detailed and careful compilation of a database of static pile load tests (the Unified database) that had the support of the proponents of recent existing CPT methods and the support of a large and active group of energy companies from around the world.

The creation of the Unified database highlighted the level of uncertainty associated with design methods in common use and prompted the development of the new non-affiliated 'Unified CPT method' described in this paper. The new method captures key characteristics of driven piles, as observed in high quality instrumented pile test programmes completed over the past 30 years. The method is shown to have higher reliability than existing approaches.

As for most empirical methods, further developments and improvements are still to come. Such improvements include the need to better quantify the effects of time in various soil types and how the resistance to pile driving as well as the long term capacity can be assessed with more confidence. More static load tests on large diameter instrumented piles are clearly required while greater understanding of factors controlling the influence of the installation mode and friction in silt is needed.

It is hoped that the 'Unified database' can be expanded on an ongoing basis with each additional pile test included satisfying the strict selection criteria applied in the database development to date.

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