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A NEW 'UNIFIED' CPT-BASED AXIAL PILE CAPACITY DESIGN METHOD FOR DRIVEN PILES IN SAND

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ABSTRACT

This paper outlines the development of a new 'Unified' CPT-based method for estimating the axial capacity of driven piles in sand. The method adapts key features of the four CPT-based methods currently in the API and ISO guidelines. The new method was calibrated with the Unified database of pile load tests developed as part of an earlier joint industry research project (Lehane et al. 2017). Key factors known to influence pile capacity are incorporated in the new Unified method formulation, including (i) the degree of soil displacement (plugging) during installation, (ii) the influence of relative pile tip depth, (iii) sand-pile interface friction angle, (iv) changes in radial stress during loading and (v) the influence of loading direction. It is shown that the new method provides more reliable predictions of the capacities of the pile load tests in the Unified database than any of the existing axial pile capacity design methods in the API and ISO guidelines.

Keywords: cone penetration test (CPT), driven piles, siliceous sand, pile design, pile load test database.

INTRODUCTION

Following completion of a Joint Industry Project (JIP) that examined the reliability of axial pile capacity design methods and included the compilation of a 'Unified database' of reliable load tests on driven piles (e.g. Lehane et al. 2017, Liu et al. 2019), a new JIP project was initiated with one of its aims being to bring together the four CPT-based methods that are recommended as alternatives in API (2011) for design of piles in sand into a single, agreed and calibrated, 'Unified' CPT-based method. This new JIP is supported by Equinor AS, Lundin Norway AS, Ørsted, ONGC, BP, Total, ExxonMobil, EnBW, EDF, Aramco, SSER and HDEC.

This paper outlines the development of the new Unified CPT-based method, which was calibrated using the 'Unified database' comprising 71 static pile load tests in siliceous sand deposits. The majority of piles in the database have diameters between 300 and 800 mm. It was therefore important that the proposed method reflected established physical mechanisms, supported by observations obtained in instrumented pile tests, to enable reasonable and safe extrapolation to the larger diameter piles commonly used offshore. The new Unified CPT-based method incorporates trends implicit in existing CPT-based methods and involves specific assessments of:

(i) the contribution of dilation to shaft friction (key for the interpretation of small diameter pile load tests in the database)
(ii) shaft friction distributions, placing particular emphasis on conditions close to the pile tip
(iii) factors affecting the pile shaft-sand interface friction angle
(iv) the filling ratio of pipe piles (important for small diameter piles)
(v) methods for averaging cone resistance q_c near the pile tip for evaluation of end bearing

Specific limitations of the method arising from shortcomings of the database include:

- The database comprises siliceous sands and the applicability of the new method to other sand types has not been assessed.
- The new method may over-estimate pile capacity in silty sands (where cone penetration is not fully drained) and under-estimate capacity in gravelly sands (where the presence of gravels leads to higher average q_c values).
- The set-up of shaft friction is not considered explicitly and the new method is intended to provide an estimate of medium-term static capacity corresponding to a set-up period of 2 weeks (which was the median set-up time for the pile load tests in the Unified database).
- The method is only applicable to piles driven in a conventional manner and should not be used for vibrated or jacked installed piles.

Full details of the Unified database and of the predictive performance of existing CPT-based methods (ICP-05, Fugro-05, NGI-05 and UWA-05) are provided in Lehane et al. (2017).

This paper first presents the basis for adoption of the selected formulations for shaft friction and end bearing. Best-fit calibration coefficients for these formulations derived from the Unified pile test database are then obtained and the rationale behind the selection of the most appropriate formulation is described. A summary of the recommended equations for the new Unified CPT-based method closes the paper. The calibration work was carried out jointly by the University of Western Australia (UWA) and the Norwegian Geotechnical Institute (NGI). They also did parallel calculations to provide an independent check of the results.

CORRELATIONS FOR SHAFT FRICTION

General trends

Field experiments with the Imperial College instrumented piles, reported by Lehane et al. (1993), Chow (1997) and Lim and Lehane (2015), such as shown on Fig. 1, have confirmed that the local ultimate shaft friction developed on the shaft of a displacement pile (τ_f) obeys Coulomb's law, i.e.

$$\tau_f = \sigma'_{rf} \tan \delta_f = (f_t/f_c)\,(\sigma'_{rc} + \Delta\sigma'_{rd}) \tan \delta_f \quad [1]$$

where σ'_{rf} is the radial effective stress at peak friction, σ'_{rc} is the (stationary) radial effective stress, $\Delta\sigma'_{rd}$ is the increase in radial effective stress that occurs during pile loading and δ_f is the ultimate (constant volume) interface friction. The f_t/f_c coefficient is unity for piles loaded in compression while a value of 0.75 is adopted for tension loading due to the Poisson ratio effect as the pile extends under tension loading (de Nicola and Randolph 1993) and due to the principal strain and stress axis reversals that take place between compressive pile driving and tension load testing, e.g. Lehane et al. (1993) and Galvis-Castro et al. (2019). A single f_t/f_c coefficient is applied, for simplicity, to both components of σ'_{rf}. O'Neill (2001), and others, provided evidence indicating that an appropriate range for f_t/f_c is 0.7 to 0.8.

The instrumented Imperial College pile tests have also shown that the stationary radial effective stress (σ'_{rc}) and ultimate shear stress (τ_f) acting at a particular level on the shaft of a displacement pile are proportional to the CPT cone resistance (q_c) value at that location. This observation, which is in evidence from the similarity of the q_c and τ_f profiles on Fig. 2, provides the basis for the shaft friction correlation of the ICP-05, UWA-05 and Fugro-05 CPT-based methods as well as for the new Unified CPT-based method.

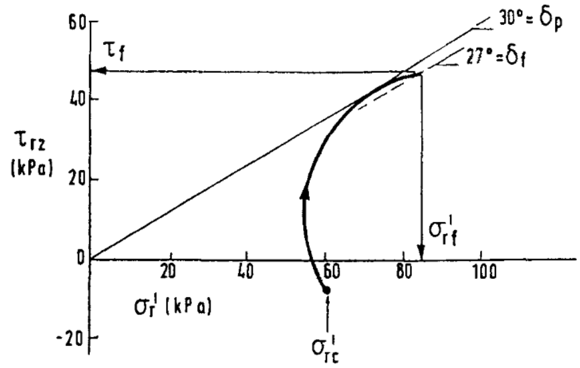

Fig. 1. Typical variation of radial effective stress and local shear stress during pile loading (Lehane et al. 1993)

There are, however, a number of other important factors controlling the magnitude of τ_f, which were considered in the development of the new Unified method:

(i) The radial effective stress and shear stress that can develop in any given soil horizon reduce as the distance from the pile tip to that horizon (*h*) increases; this is evident on Fig. 2 which shows τ_f values recorded by instrument clusters located at various *h/D* ratios, where *D* is the outer pile diameter. The phenomenon, which is sometimes referred to as friction fatigue, arises most dramatically close to the pile tip due to geometrical effects (e.g. Jardine et al. 2013). It continues as the pile tip penetrates to greater depths, due to contraction of the sand at the interface (leading to a reduction in radial stress) induced by the shearing cycles imposed by driving/jacking increments of a displacement pile (White and Lehane 2004); this is accompanied by grain crushing due to ongoing interface shear abrasion (Yang et al. 2010). Creep straining in the surrounding sand mass contributes additional radial contraction and promotes an arching mechanism around the pile shaft (Jardine et al. 2013).

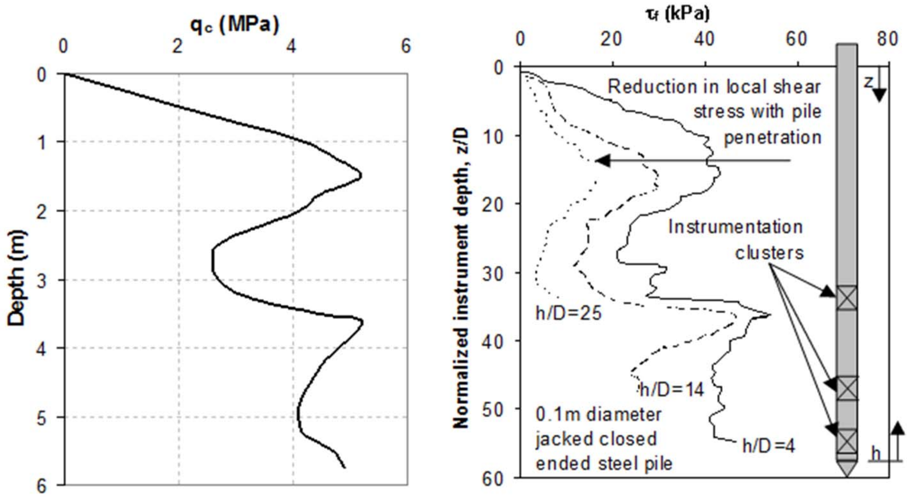

Fig. 2. Instrumented test data indicating $\tau_f = f(q_c, h/D)$ (Lehane et al. 1993)

(ii) The local radial contraction is constrained by the cavity contraction stiffness of the sand mass, which varies inversely with the pile diameter, while the number of shearing cycles and interface shear abrasion experienced in any given soil horizon increases with *h*.

Consequently, instrumented displacement piles of different geometries have shown that there is progressive reduction of radial stress (and hence available shear stress) as *h*/D increases. The new method examined dependencies of τ_f on both *h* and *h/D*.

(iii) Gavin and Lehane (2003) confirmed that τ_f varies with the degree of soil displacement imparted during installation and is lowest for fully coring pipe piles and largest for closed-ended piles. This degree of displacement is often quantified using the plug length ratio (*PLR*), which is the ratio of the plug length to the pile embedment or by the incremental filling ratio (*IFR*), which is the ratio of the incremental change in plug length to an increment of embedment. The *PLR* (which is the average *IFR* during installation) is primarily a function of the internal pile diameter (D_i) e.g. Gudavalli et al. (2013). The following approximate expression was derived based on available records (where d_{CPT} is the diameter of a standard CPT probe)

$$PLR \approx IFR = \tanh\left[0.3\left(\frac{D_i}{d_{CPT}}\right)^{0.5}\right]; \quad d_{CPT} = 35.7mm \qquad [2]$$

White et al. (2005) used a cavity expansion analogy to deduce that the equalized lateral effective stress acting on the pile shaft (σ'_{rc}) varies with the effective area ratio (A_{re}) raised to a power of 0.3 to 0.45, where

$$A_{re} = 1 - PLR\,(D_i/D)^2 \qquad [3]$$

(iv) Dilation of the sand at the pile shaft interface during loading leads to an increase in radial stress ($\Delta\sigma'_{rd}$) due to the constraint provided by the sand mass surrounding the pile. The magnitude of $\Delta\sigma'_{rd}$ varies with the cavity expansion stiffness and hence, while making a significant contribution to the shaft friction of small diameter piles, its influence on the shaft friction reduces systematically with pile diameter.

(v) The shaft capacity of driven piles in sand increases with time over a period of about 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. 2018). Gavin et al. (2015) and Zhang and Wang (2016) note that interface dilation is enhanced through ageing while Carroll et al. (2020) show that the ageing is scale dependent and is dominated, for small steel piles, by corrosion processes. The JIP database of pile load tests has a typical set-up time of between 1 week and 2 months with a median of 14 days. The proposed Unified CPT-based method is intended to provide an estimate of shaft friction available at around 14 days after driving.

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

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 (Eq. 1). 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, *y/2D* is the cavity strain and k_n is the normal stiffness:

$$\Delta\sigma'_{rd} = 4G \cdot y/D = k_n \cdot y \qquad [4]$$

The UWA-05 and ICP-05 methods assume that the cavity strain is small enough for conditions to be fully elastic and hence *G* was assumed equal to 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 operational *G* value is equivalent to the shear modulus measured in a self-boring pressuremeter test (assuming no significant disturbance to the sand mass due to pile driving or pressuremeter insertion) and varies non-linearly with the cavity strain in a format that can be described approximately by Eq. 5 (Lehane, 2009).

Constant normal stiffness direct shear tests on silica sands sheared against rough steel interfaces indicate that the maximum dilation reduces non-linearly with normal stiffness k_n. Lehane et al. (2005) indicated that Eq. 5, using a reference normal stiffness (k_{nref}) of 500 kPa/mm, provided a reasonable fit to dilation values measured in direct shear interface tests on a uniform silica sand, where y_0 is the maximum dilation measured in a constant normal load test (with k_n=0) and the exponent α in Eq. 5 is typically between 0.4 and 0.7 (Lehane 2009).

$$G = \frac{G_o}{\left[1 + \left(\frac{2y/D}{0.0001}\right)^{\alpha}\right]} \qquad y = \frac{y_0}{\left[1 + \left(\frac{k_n}{k_{nref}}\right)^{\alpha}\right]} \qquad [5]$$

The relationship between the small strain stiffness (G_0) and q_c adopted in the ICP-05 and UWA-05 formulations was combined with Eqs 4 and 5 to predict $\Delta\sigma'_{rd}/q_c$ ratios in sands with normalised cone resistances (q_{c1N}) of 70 and 220 (corresponding to sand relative densities of about 40% and 80% respectively). These (non-linear) calculations showed a relatively low sensitivity to the selected value of α, y_0 and normal effective stress. Figure 3 presents the results of the calculations for the two relative densities, assuming a y_0 value of 0.15mm (as measured for sand with D_{50}=0.3 ± 0.1 mm), α=0.7, k_{nref} =500 kPa/mm and σ'_v=100 kPa. The predicted $\Delta\sigma'_{rd}$ value reduces dramatically as the pile diameter increases and varies directly with q_c (as G_0 varies with q_c). The $\Delta\sigma'_{rd}/q_c$ ratios are greater in the looser sand.

A database of about 40 $\Delta\sigma'_{rd}$ measurements compiled for this study indicated the following best-fit relationship (where σ'_v is the vertical effective stress and d_{CPT} is the diameter of the standard CPT probe = 35.7mm):

$$\Delta\sigma'_{rd} = \left(\frac{q_c}{10}\right)\left(\frac{q_c}{\sigma'_v}\right)^{-0.33}\left(\frac{d_{CPT}}{D}\right) \qquad [6]$$

This $\Delta\sigma'_{rd}$ database largely comprised small diameter piles jacked piles with a median set-up time of less than one week. Larger values of $\Delta\sigma'_{rd}$ can be expected after significant driving cycles (such as at large *h/D* values), lengthy set-up times and where significant changes in pile surface roughness occur.

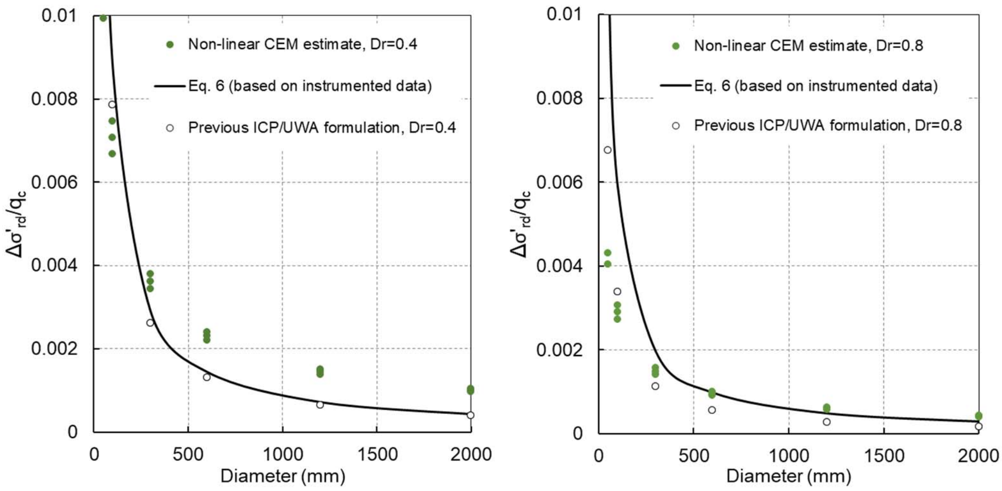

Fig. 3. Comparison of predictions of $\Delta\sigma'_{rd}/q_c$ from CE theory with result of regression analysis from instrumented data (Eq. 6) for two relative densities of a sand

Equation 6 has a relatively large coefficient of variation (COV) of about 0.45 for the $\Delta\sigma'_{rd}$ database (which can be partly attributed to the range of set-up times and pile surface roughness in this data set), but is still an improvement on the fit provided to the same data obtained with the UWA-05 and ICP-05 formulations. It is also encouraging to see on Fig. 3 that the same equation provides a reasonable match to the typical trend derived independently using the cavity expansion method (CEM) for the denser sand. The contribution of dilation predicted using Eq. 6 to the shaft friction of the database piles calculated using the Unified CPT-based method (described later) typically reduces from about 35% to 10% in medium dense sand as the diameter increases from 350 to 800 mm, for piles about 20m in length (i.e. the typical range in the database). The relative influence of dilation is greatest in looser sands and for longer piles.

Interface friction angle

The ICP-05 method proposes that site-specific tests should be conducted to establish ultimate interface friction (δ_f) angles, but notes that, in the absence of such tests, the variation with the mean effective particle size (D_{50}) indicated by the dashed curve labelled Jardine et al. (1993) in Fig. 4 can be employed. This variation was deduced from direct shear box tests that did not include pre-shearing to large relative displacements.

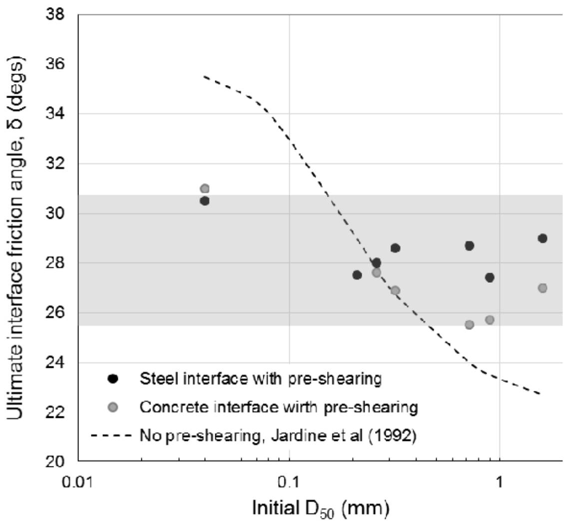

Fig. 4. Variation of interface friction angle with D_{50}

It is noteworthy that the Fugro-05 and NGI-05 methods do not specify a dependence of δ_f on D_{50}, stating that crushing of sand at the pile tip during installation reduces the grading to that of a fine sand, as demonstrated by Yang et al. (2010). Interface shear investigations conducted in the Imperial College Bishop ring shear apparatus and presented by Barmpopoulos and Ho (2009) and Ho et al. (2011) confirmed the relatively low sensitivity of δ_f to D_{50}. These experiments adopted interfaces with the roughness of industrial piles and applied the ring shear procedures recommended in Ramsey et al. (1998) involving a high level of fast pre-shearing. More recent testing using these procedures at NGI and Imperial College (Liu et al. 2019) has also shown a relatively low sensitivity of δ_f to (non-plastic) fines contents, mineralogy and normal stress level variations and that, in the absence of site specific ring shear interface tests, adoption of the mean measured δ_f value of 29° is a reasonable option for calibration of the Unified pile database.

Investigation of suitable formulation for ultimate shaft friction (τ_f)

The formats for the equalised (stationary) radial effective stress considered for the development of the Unified CPT-based method are summarised in Table 1. These were selected after examining formulations of the current CPT-based methods, and those proposed by Randolph et al. (1994), Salgado et al. (2011) and Alm and Hamre (2001). In all, five formulations given in Table 1 were considered, where the constants *a*, *b*, and *c* could be varied to match observations. The value of *p'* in Equation F5 was calculated assuming a normally consolidated coefficient of earth pressure at rest, K_0, value of 0.45. The central assumption of all formulations and the new Unified CPT-based method is that the equalised (stationary) radial effective stress (σ'_{rc}) varies directly with the CPT cone resistance (q_c).

Table 1. Equation formats employed for σ'_{rc} estimation

$\sigma'_{rc} = A_1\, q_c \left(\frac{\sigma'_{v0}}{p_a}\right)^a A_{re}^b \left(\frac{\text{h}}{D}\right)^{-c}$	[F1]
$\sigma'_{rc} = A_2\, q_c \left(\frac{\sigma'_{v0}}{p_a}\right)^a \left(\frac{\text{h}}{D^*}\right)^{-c} \qquad D^* = (D^2 - {D_i}^2)^{0.5}$	[F2]
$\sigma'_{rc} = A_3\, q_c \left(\frac{\sigma'_{v0}}{p_a}\right)^a A_{re}^b \exp(-f\text{h}/D)$	[F3]
$\sigma'_{rc} = A_4\, q_c \left(\frac{\sigma'_{v0}}{p_a}\right)^a A_{re}^b\, exp(-g\text{h})\,; g = constant, \text{h}\ in\ (m)$	[F4]
$\sigma'_{rc} = A_5\, q_c \left(\frac{\sigma'_{v0}}{p_a}\right)^a A_{re}^b \exp(-g\text{h})\,; g = \frac{1}{80}\left(\frac{q_c}{p'}\right)^{0.5}, \text{h}\ in\ (m)$	[F5]

Close inspection of measured shaft friction distributions in good quality case histories indicated that friction increased with proximity to the tip and, in uniform sands, increased from ground level to within at least about one diameter of the tip. The calibrations were therefore performed with a maximum friction cut-off at *h/D*=1 for both open and closed-ended piles.

Although one of the existing CPT methods (NGI-05) specifies a minimum shaft friction ($\tau_{f,min}$) of $0.1\sigma'_v$, frictions measured at large distances from the pile tip in a number of case histories were close to zero e.g. $0.03\sigma'_v$ in 1.5m diameter piles in Tokyo (Kikuchi et al. 2007) and $0.06\sigma'_v$ for the 760mm diameter EURIPIDES piles (Kolk et al. 2005). As the minimum radial effective stress that can exist around the shaft of a driven pile is indeterminate (and is less than the plane strain active value), it was considered prudent not to specify a value of $\tau_{f,min}$. Furthermore, unlike the main text method in API (2011), there is no upper limit placed on the magnitude of shaft friction.

END BEARING FORMULATIONS

Although most offshore piles are driven open-ended, almost half (31) of the pile tests in the Unified database involved closed-ended piles tested in compression. Calibration of a new method against this database therefore required a method for estimating end bearing for piles with various end conditions.

Closed-ended piles

It has been shown by White and Bolton (2005), and others, that the bearing stress acting at the base of a closed-ended driven pile, defined at ultimate conditions at a displacement of 10% of the pile diameter ($q_{b0.1}$), varies in direct proportion to the average q_c value in the vicinity of the tip ($q_{c,avg}$). The ICP-05 and Fugro-05 methods define $q_{c,avg}$ as the average q_c value in a zone extending 1.5*D* above and below the pile tip ($q_{c,1.5D}$) whereas NGI-05 adopts the cone resistance at the pile tip ($q_{c,tip}$).

The average q_c value considered for the calibration of the new method is the end bearing resistance expected for an 'imaginary cone' that has the same diameter as the pile being considered, and is referred to here as q_p. As described in Bittar et al. (2020), the value of q_p is determined by applying a filter proposed by Boulanger and DeJong (2018) to the measured q_c profile, where the filter is a function of the distance above and below the cone/pile tip normalized by the penetrometer diameter ($z' = z/d$). The filter formulation is based on observations made in a large range of experimental and numerical investigations on penetrometers in layered soils and gives (i) soils above the cone/pile tip at z'<0 a lower relative influence on end resistance and (ii) weaker soils remote from the cone/pile tip a greater relative influence on the end resistance mobilized.

The values of $q_{b0.1}$ and $q_{c,avg}$ were evaluated for a database of closed-ended piles using an extended version of a database compiled by Xu et al. (2008), described in Bittar et al. (2020). Ratios of $q_{b0.1}/q_{c,avg}$, where $q_{c,avg}$ was taken equal to $q_{c,1.5D}$ (as used in the ICP-05 and Fugro-05 methods), $q_{c,Dutch}$ (described by Schmertmann 1978 and recommended by the UWA-05 method) and equal to q_p, are plotted versus pile diameter on Fig. 5. The $q_{b0.1}/q_{c,avg}$ values vary between 0.3 and 1 and are less than unity on average due to the development of ultimate conditions requiring greater displacement than 0.1D. Although the measured base capacities contain uncertainties related to the interpretation of residual loads, regression analyses show a clear trend for $q_{b0.1}/q_{c,1.5D}$ values to reduce as the pile diameter increases while $q_{b0.1}/q_p$ and $q_{b0.1}/q_{c,Dutch}$ ratios do not show any diameter dependence. The diameter dependence of $q_{b0.1}/q_{c,1.5D}$ is a consequence of disregarding the influence of sand layers at distances larger than 1.5D from the tip.

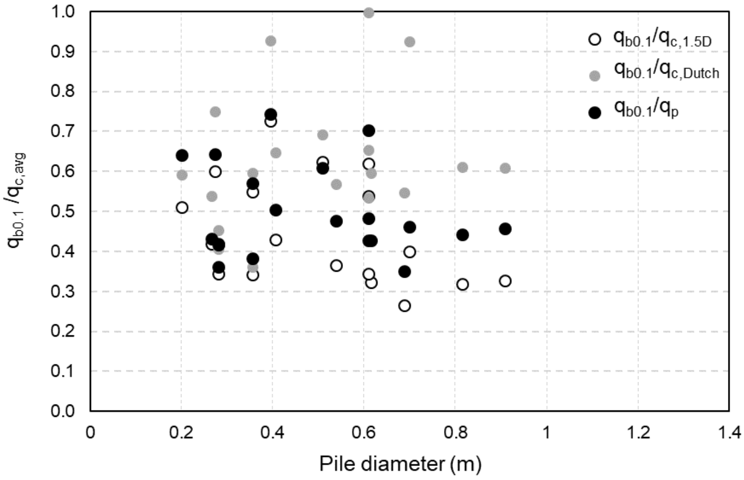

Fig. 5. $q_{b0.1}$ values compared with two q_c averaging technique for database of end bearing measurements

Equation 7a, which was used for calibration of the Unified database, provides the best fit relationship for $q_{b0.1}$ with the lowest coefficient of variation for this set of end bearing results, displaying no bias with respect to pile diameter or sand relative density. Equation 7b has a larger coefficient of variation but can be considered a reasonable approximation for end bearing calculations.

$$q_{b0.1} = 0.5\ q_p \qquad [7a]$$

$$q_{b0.1} = 0.6\ q_{c,Dutch} \qquad [7b]$$

Open-ended piles

The plug of a typical pipe pile 'locks-up' under static compression loading (Lehane and Randolph 2002). Sand plugs do not, however, remain static under the dynamic forces induced by pile driving and the level of plugging during installation varies from full coring (i.e. no plugging, PLR=1) for large diameter piles to full plugging (PLR=0) for small diameter piles. As a consequence, the pile base stress that can be developed at a settlement of 0.1D under subsequent static loading ($q_{b0.1}$) has been shown by Gavin and Lehane (2003), and others, to vary with the level of pre-stressing induced by sand displacement generated near the pile base during the installation process. The level of sand displacement near the base can be described by the final filling ratio (*FFR*), which is taken here to be the average incremental filling ratio (*IFR*) over the final 3 diameters of installation. The effective area ratio of a given open ended pile (i.e. the ratio of the displacement induced to that of a fully plugged pile) is then given as (where D_i is the internal diameter):

$$A_{re} = 1 - FFR\,(D_i/D)^2 \approx 1 - PLR\,(D_i/D)^2 \qquad [8]$$

An expression for end bearing capacity was examined using a database of $q_{b0.1}$ values which was extended from that originally described by Xu et al. (2008) and comprised 17 open-ended piles. Most satisfactory results were obtained when $q_{b0.1}$ values were related to the effective area ratio (A_{re}) and the q_p value (for an 'imaginary cone with a base area = $A_{re}\,\pi D^2/4$), as shown by the systematic trend in Fig. 6. This q_p value was determined using the Boulanger and DeJong (2018) filter for an 'imaginary cone with a base area = $A_{re}\,\pi D^2/4$ (following the transformation from an open-ended pile to an equivalent closed-ended pile proposed by Randolph 2003). Values of A_{re} were derived using either the plug length ratio (*PLR*) or the final filling ratio (*FFR*) when *IFR* data were available.

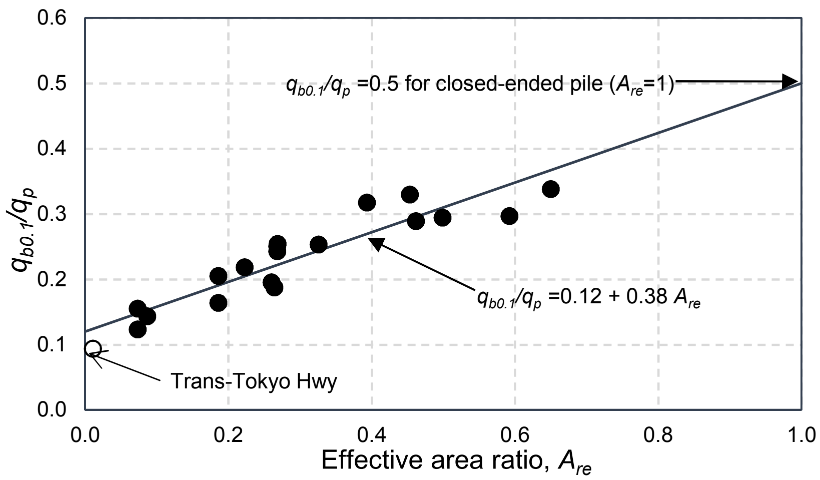

Fig. 6. $q_{b0.1}/q_p$ as a function of effective area ratio

A consistent linear increase in $q_{b0.1}/q_p$ with A_{re} is apparent on Fig. 6 with an extrapolated $q_{b0.1}/q_p$ ratio at A_{re} = 1 corresponding to the best estimate for closed-ended piles (as indicated in Fig. 5 and Eq. 7). The 2m diameter pipe pile at the Trans-Tokyo Highway indicated a $q_{b0.1}/q_p$ ratio of only 0.09, as seen on Fig. 6, but this is likely to be due to the base's proximity to an underlying clay layer (e.g. Randolph 2003). This data point was therefore not included in the regression analysis, which gave the following the best fit relationship given in Eq. 9a, with a coefficient of variation of measured to calculated ratios of 0.11. This expression displays no bias with respect to diameter and sand relative density for this set of end bearing measurements. This equation was used in the estimation of end bearing capacity for the piles in the Unified database. Filling ratios to determine A_{re} in Eq. 8 were not, however, measured in about half of the tests on open-ended piles and, for these cases, *PLR* values were estimated using Eq. 2.

$$q_{b0.1} = q_p\,(0.12 + 0.38\,A_{re}) \qquad [9a]$$

Additional analysis showed that Eq. 9b could be used with little loss in accuracy as an alternative to Eq. 9a to estimate end bearing of pipe piles.

$$q_{b0.1} = q_{c,Dutch}\,(0.15 + 0.45\,A_{re}) \qquad [9b]$$

Full scale offshore pipe piles core during driving and the value of A_{re} is approximately 4/(*D*/*t*), where *t* is the pile wall thickness. For typical *D/t* ratios of between 30 and 60, Eq. 9a then gives an average $q_{b0.1}/q_p$ ratio of 0.15, which is in good agreement with the $q_{b0.1}/q_c$ ratios of between 0.15 and 0.2 used for bored piles (drilled shafts) in sands, where q_c is the average CPT resistance between the pile base and 1*D* below the pile base (De Cock et al. 2003, Lehane 2019). Lehane and Randolph (2002) show that the higher end bearing developed beneath the annulus compensates for the reduced resistance associated with compression of the sand plug so that $q_{b0.1}$ values are closely comparable to those for bored piles in sand. Therefore, while Eq. 9a is most suitable for calibration with the Unified database of pile load tests, a reasonable estimate of the end bearing resistance of full-scale offshore piles can determined more directly from the following simple relationship:

$$q_{b0.1} = 0.15\ \overline{q_c} \qquad [10]$$

As the loading of a deep footing or pile base is analogous to the expansion of a spherical cavity below the base (Lehane 2019), the appropriate value of $\overline{q_c}$ to employ in Eq. 10 is the average CPT end resistance between the pile tip and 1*D* below the tip. However, as cone resistances in sand can have significant spatial variability, it is proposed that $\overline{q_c}$ for design purposes be taken as a cautious estimate of the cone end resistance in a zone extending 1.5*D* above and below the pile tip.

DEVELOPMENT OF FORMULATIONS FOR NEW UNIFIED CPT-BASED METHOD

Suitability of shaft friction formulations

The suitability of formulations F1 to F5 was first assessed by examining their ability to predict profiles of ultimate shaft friction measured in ten good quality case histories. A number of well documented cases were examined. They included piles load test from the Unified database and pile load tests outside the database, in particular 1.5m diameter piles in Tokyo (Kikuchi et al. 2007) and 660mm diameter piles in Lafayette (Han et al. 2019). Shaft frictions were calculated using Eqs 1, 2, 3 and 6 combined with best-fit coefficients for each of the equations in Table 1. It was observed that:

(i) Each of the σ'_{rc} equations led to relatively poor predictions of measured local frictions ($\tau_{f,meas}$) e.g. the coefficient of variation, COV, of the measured to calculated local friction ratio are about 0.5. It should be noted, however, that $\tau_{f,meas}$ values are very sensitive to the interpreted load distributions inferred from strain gauges.
(ii) The COV of the ratio of the measured to calculated total shaft friction was significantly lower and was approximately 0.14 ± 0.02 for formulations F1, F2, F3 & F4 but almost twice this value for formulation F5.
(iii) Allowing the constant '*g*' to vary with q_c/p' (as proposed by Alm and Hamre 2001) in Eq. F5 led to much poorer estimates of measured shaft frictions; this equation was therefore not considered for calibration against the Unified database.
(iv) The measured shaft frictions justify the selection of a 'cut-off' for τ_f that is closer to the pile tip than assumed in the ICP-05 and UWA-05 methods, which adopt cut-offs of 4*D* and 2*D* respectively for closed-ended piles.

One of the most notable observations from this stage of the method development was that, even though the shaft friction distribution may be poorly represented, reasonable fits with the measured shaft capacities could be obtained equally well by adopting a wide range of calibration constants. Multiple choices of *a, b*, *c*, *f* and *g* in conjunction with a different constant, A_i (Table 1), could give closely comparable coefficients of variation for ratios of measured to calculated shaft capacities. This is considered to arise because all formulations incorporate the two most important features of driven piles, namely (i) a proportional relationship between σ'_{rc} and q_c and (ii) and allowance for σ'_{rc} to degrade with distance above the tip (*h*). The available field data and systems of equations evidently do not lend themselves to a precise inverse analysis that could identify unambiguously the most representative formulation.

Method calibration procedure

The method calibration was conducted using two separate approaches to serve as a check on the analyses. Following an initial round of calibrations and sensitivity studies, it was concluded that the exponent to the σ'_v term in the σ'_{rc} formulations (i.e. '*a*' in Table 1) could, for simplicity, be set to zero. Only very slight improvements of the fit to the database (typically 1%) were found through inclusion of a non-zero '*a*' term.

Approach 1. Calibration with EXCEL-based optimisation

The EXCEL-based optimization first minimized the coefficient of variation of the weighted ratios (COV_w) of measured to calculated shaft capacity (Q_m/Q_c) of pipe piles in the Unified database (given the prevalence of pipe piles offshore). Lehane et al. (2017) described the weighting procedure, giving greater weights to higher quality pile load tests (as determined by the team of experts appointed by the previous JIP) and reducing the weighting applied to piles at sites where there were multiple pile tests conducted. Shaft frictions were calculated using formulations F1 to F4 combined with Eqs. 1 and 6 (as well as Eqs. 2 and 3 if filling ratio were not available). Eqs. 7a and 9a were employed to determine shaft friction from measured total capacity in compression tests when reliable end bearing data were not reported (which was for 70% of the cases). The optimisation included the following steps to derive optimal values for the coefficients *A, b, c, f* and *g* in formulations F1 to F4. The EXCEL tool *Solver* was used with constraints applied to limit the range of some parameters to be consistent with existing driven pile research and with the profiles of shaft friction discussed above (e.g. $0.3 \leq b \leq 0.45$; $0.4 \leq c \leq 1.0$; $0.7 < f_t/f_c < 0.85$).

(i) Determine Q_m/Q_c ratios for open-ended piles tested in tension and compression (29 pile tests).

(ii) Vary the coefficients *b*, *c*, *f* and *g* to minimize the COV_w of the Q_m/Q_c ratios. Adjust the coefficient *A* so that the mean weighted value of Q_m/Q_c ratios (μ_w) is close to unity.

(iii) Review results and amend coefficients to minimise any bias of Q_m/Q_c with diameter, pile slenderness, relative density (on both the base and shaft) and effective area ratio A_{re}.

(iv) Evaluate μ_w and COV_w for the ratios of measured to calculated *total* capacity for the entire Unified database (71 pile tests) using the optimum coefficients determined for open-ended piles. Assess whether statistics for closed-ended piles are satisfactory.

Approach 2. Calibration with Python-based minimisation procedure

In the Python-based minimisation procedure, the parameters were varied to minimise the coefficient of variation of the measured to calculated shaft capacities. Optimal parameters for the coefficients *A, b, c, f* and *g* in Equations F1 to F4 were determined by maintaining the mean of the Q_m/Q_c ratios at unity and minimising their coefficient of variation using the *Minimise* function in the *Scipy* toolbox. Constraints applied were the same as those in the EXCEL-based approach.

Calibration results

The results from the EXCEL and Python-based optimisation were almost identical, lending confidence to the accuracy of the calibrations. The optimal coefficients determined for formulations F1 to F4 for the open-ended piles in the database are provided in Table 2 and the corresponding weighted mean (μ_w) and COV values (COV_w) for shaft friction of open-ended piles and for total capacity of all piles in the Unified database are provided in Table 3.

It is encouraging to observe from Table 3 that the equations given in Table 2 derived for the open-ended piles in the database also lead to good predictions for all closed- and open-ended piles in the database.

Table 2. Calibrated equations for σ'_{rc} estimations on open-ended piles

$\sigma'_{rc} = 0.023\, q_c\, {A_{re}}^{0.3} \left(\frac{h}{D}\right)^{-0.4}$	[F1]
$\sigma'_{rc} = 0.022\, q_c \left(\frac{h}{D^*}\right)^{-0.4} \qquad D^* = (D^2 - {D_i}^2)^{0.5}$	[F2]
$\sigma'_{rc} = 0.019\, {A_{re}}^{0.33} \exp(-0.04h/D)$	[F3]
$\sigma'_{rc} = 0.014\, q_c\, {A_{re}}^{0.3} \exp(-0.04h)\,;\, g = constant\,, h\ in\ (m)$	[F4]

Table 3 also shows that the new equations (F1 to F4) are a slight improvement on the best performing existing CPT methods and have significantly lower COV_w values than the existing API method. No significant change in the COV_w values was observed when the effects of time were included using the set-up correction factor described in Lehane et al. (2017); this characteristic of the 'Unified database' is discussed in the same article.

Table 3. Q_m/Q_c statistics of pile tests in the Unified database (from Excel-based optimisation)

Method	Open-ended piles (shaft capacity); 29 piles			All open & closed-ended piles (total capacity); 71 piles		
	μ_w	σ_w	COV_w	μ_w	σ_w	COV_w
API-00	1.64	0.93	0.57	1.66	0.93	0.56
Fugro-05	1.06	0.39	0.37	0.99	0.39	0.40
ICP-05	1.00	0.25	0.25	1.04	0.28	0.27
NGI-05	1.03	0.29	0.28	0.99	0.34	0.34
UWA-05	1.00	0.23	0.23	1.06	0.28	0.26
Equation F1	1.02	0.22	0.21	1.05	0.25	0.24
Equation F2	1.02	0.23	0.23	1.08	0.26	0.24
Equation F3	1.02	0.23	0.23	1.05	0.26	0.25
Equation F4	1.02	0.23	0.23	1.01	0.25	0.25

Selection of formulation for ultimate shaft friction (τ_f)

Given the inability of the optimisation process to clearly identify the most suitable formulation for σ'_{rc} (noting the COV_w values for Formulations F1 to F4 are almost identical), the selection of the most appropriate formulation was based on agreement with measured τ_f profiles such as discussed above, with a focus particularly on the larger diameter piles. Two important examples are shown in Fig. 7 for the case 1.5m diameter pipe piles which are not included in the 'Unified database' (because of the significant effect of clay layers at shallow depth). This figure shows features common to all cases examined namely:

- Formulation F2 over-predicts shaft friction near the tip of pipe piles
- Formulation F4 and, to a lesser extent, Formulation F3 under-predict shaft friction for larger diameter pipe piles.

Calculations also revealed a systematic tendency for capacities calculated using Formulation F4 to be progressively under-estimated with an increase in pile diameter. This trend arises because F4 assumes a decay in τ_f with increasing h rather than the decay with *h/D* adopted by F1 to F3. It should be noted that experimental data with a range of smaller diameter piles, such as reported by Lim and Lehane (2014), have demonstrated that σ'_{rc} varies with *h/D*. It may also be recalled that the cavity contraction stiffness, which controls the reduction in lateral stress associated with contraction of sand at the interface under the cycling and interface shear abrasion imposed during driving, is inversely proportional to the diameter.

Formulation F2 over-predicts frictions close to the pile tips (and consequently under-predicts friction further away from the tip) while Formulation F4 does not capture the correct diameter dependence. F1, in general, provides the best match to the instrumented pile measurements although the exponential format of F3 also provides reasonable predictions. Until such time as further good quality instrumented data become available, it is proposed that F1 is adopted as:

(i) its format is comparable to three of the existing CPT-based methods in API (2011).
(ii) it errs on the cautious side compared with Eq. F3 for shorter piles (such as those installed for the wind industry) with *L/D* < 20.
(iii) it has the lowest coefficient of variation for Q_m/Q_c among the four equations considered.
(iv) its predictions of shaft friction distributions are marginally better than Eq. F3.

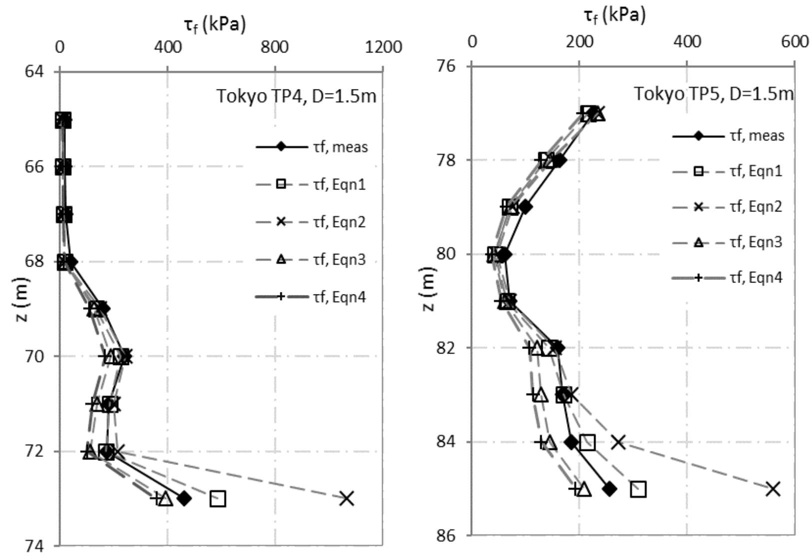

Fig. 7. Measured and calculated τ_f profiles in large diameter piles in Tokyo

It is important to consider cases where Formulation F1 over-estimates pile capacities. The Q_m/Q_c ratio obtained using this equation was less than 0.7 for 6 of the 71 piles in the database. For these 6 piles:

- Four were at the Lock & Dam site. Tests at this site had been assigned a relatively low quality rating (Lehane et al. 2017) because of variable CPT data and dubious instrumentation results.
- One pile was at the Drammen site in Norway. However, there are 12 pile tests at Drammen in the database with an average Q_m/Q_c = 1.08 and a COV for Q_m/Q_c of 0.22. The single pile (with Q_m/Q_c = 0.61) at Drammen was considered an outlier.
- The single other over-predicted pile test was at Padre Island. This test was performed just two days after driving and it is surmised that a higher Q_m/Q_c value would have been obtained if the test had a longer set-up period.

SUMMARY OF FORMULATIONS FOR THE NEW UNIFIED CPT-BASED METHOD

The equations provided in Table 4 are recommended for the assessment of the shaft capacity of a circular driven pile in clean siliceous sand at a set-up period of about 2 weeks. Larger shaft frictions can develop after set-up periods over two weeks. Direct measurement of filling ratios is preferable in any load test back-analysis than employing the empirical expression given for *PLR*. The equations provided in Table 5 are recommended for the evaluation of the contribution of the base resistance (at a base displacement of 10% of the pile diameter) to the compression capacity of a driven pile; this capacity which does not vary with set-up time.

Table 4. Equations for shaft capacity of circular driven piles in Unified CPT-based method

$$Q_{shaft} = \pi \, D \int_0^L \tau_f \; dz$$

$\tau_f = (f_t/f_c)\,(\sigma'_{rc} + \Delta\sigma'_{rd}) \tan 29°$

$\sigma'_{rc} = (q_c/44) A_{re}^{0.3} \; [Max[1,(h/D)]]^{-0.4}$

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

$f_t/f_c = 1.0$ in compression and 0.75 in tension

$A_{re} = 1 - PLR\,(D_i/D)^2$; $PLR = L_p/L$, $A_{re}=1$ for closed-ended pile

$PLR \approx \tanh[0.3(D_i/d_{CPT})^{0.5}]$; $\quad d_{CPT} = 35.7mm$

Table 5. Equations for base capacity of circular driven piles in Unified CPT-based method

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

$q_{b0.1} = [0.12 + 0.38\,A_{re}]\,q_p$

$A_{re} = 1 - FFR\,(D_i/D)^2 \approx 1 - PLR\,(D_i/D)^2$ (Expression for PLR on Table 4)

$A_{re} = 1$ for closed-ended pile

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

$D_{eq} = D\,A_{re}^{0.5}$

q_p = end bearing mobilised at large displacements at the level of the pile tip by a pile with a diameter of D_{eq}. 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 and DeJong (2018).

For full scale offshore pipe piles which fully core (so that *FFR*=1) during installation and for which the contribution of dilation can be conservatively ignored (*D*>1m), the equations for shaft friction and unit end bearing simplify to those given in Table 6.

Table 6. Equations for axial capacity of large circular offshore pipe piles in Unified CPT-based method

$$Q_{shaft} = \pi \, D \int_0^L \tau_f \; dz$$

$\tau_f = \left(\frac{q_c}{80}\right)\left(\frac{f_t}{f_c}\right)[Max[1,(h/D)]]^{-0.4}\left[1 - \left(\frac{D_i}{D}\right)^2\right]^{0.3}$ (2 weeks after driving)

$f_t/f_c = 0.75$ in tens, 1.0 in comp.

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

$q_{b0.1} = 0.15\,\overline{q_c}$ (compression piles)

where $\overline{q_c}$ is a cautious estimate of the cone end resistance in a zone extending 1.5D above and below the pile tip.

Equations presented in this paper should be used with caution when applied to soil or pile conditions that are not consistent with the database used for calibration e.g. for very large diameter piles (D>3m) for which no data exist at present. Full details of the characteristics of the database piles are provided in Lehane et al. (2017).

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REFERENCES

Alm, T. and Hamre, L. 2001. Soil model for pile driveability predictions based on CPT interpretations. Proceedings of the 15th International Conference on Soil Mechanics and Geotechnical Engineering,: 1297–1302.

Anusic I., Lehane B.M., Eiksund G., and Liingaard M. 2018. Evaluation of installation effects on the set-up of field displacement piles in sand. Canadian Geotechnical Journal, 56(4), 461–472.

API, R. 2011. 2GEO Geotechnical and foundation design considerations. American Petroleum Institute, Washington, DC, USA,: 120.

Barmpopoulos, I. and Ho, T. 2009. The large displacement shear characteristics of granular media against concrete and steel interfaces. Proc Research Symp. Characterization & Behavior of Interfaces, Atlanta, Georgia, USA Behavior of Interfaces, pp.16–23.

Bittar, E., Lehane, B.M., Boulanger, R.W., and DeJong, J. 2020. CPT filter to estimate end bearing of closed-ended driven piles in layered sands. Proc. Int. Symp. Frontiers in Offshore Geotechnics, Austin, Texas, August 2020 (in press).

Boulanger, R.W.W., and DeJong, J.T.T. 2018. Inverse filtering procedure to correct cone penetration data for thin-layer and transition effects. Cone Penetration Testing 2018: Proc 4th Int Symposium on Cone Penetration Testing (CPT'18), 21-22 June, 2018, Delft, The Netherlands,: 25.

Carroll, R., Carotenuto, P., Dano, C., Salama, I., Silva, M., Gavin, K., and Jardine, R.J. 2020. Field experiments at three sites to investigate the effects of age on steel piles driven in sand. Geotechnique, 70, *doi/10.1680/jgeot.17.P.185*

Chow, F. 1997. Investigations into the behaviour of displacement piles for offshore structures, PhD thesis, Imperial College London.

Chow, F.C., Jardine, R.J., Nauroy, J.F., and Brucy, F. 1998. Effects of Time on Capacity of Pipe Piles in Dense Marine Sand. J. Geotech. and Geoenv. Engineering, 124: 254–264.

De Nicola, A., and Randolph, M.F. 1993. Tensile and Compressive Shaft Capacity of Piles in Sand. Journal of Geotechnical Engineering, 119(12): 1952–1973.

Gavin, K., Jardine, R., Karlsrud, K., and Lehane, B.M. 2015. The effects of pile ageing on the shaft capacity of offshore piles in sand. Frontiers in Offshore Geotechnics III, 1: 129–151.

Gavin, K.G., and Lehane, B.M. 2003. The shaft capacity of pipe piles in sand. Canadian Geotechnical Journal, 40(1): 36–45.

Galvis-Castro, A.C., Tovar-Valencia, R.D., Salgado, R., and Prezzi, M. 2019. Effect of loading direction on the shaft resistance of jacked piles in dense sand. Géotechnique, 69(1)16–28.

Han, F., Ganju, E., Prezzi, M., Salgado, R., and Zaheer, M. 2018. Axial resistance of open-ended pipe pile driven in gravelly sand. Géotechnique, 70(2), 138–152.

Ho, T.Y.K., Jardine, R.J., and Anh-Minh, N. 2011. Large-displacement interface shear between steel and granular media. Geotechnique, 61(3): 221–234.

Jardine, R.J., Lehane, B.M., and Everton, S.J. 1993. Friction coefficients for piles in sands and silts. *In* Offshore site investigation and foundation behaviour. Springer. pp. 661–677.

Jardine, R., Standing, J., and Chow, F, 2006. Some observations of the effects of time on the capacity of *piles* driven in sand. Géotechnique, 56(4), 227–244.

Jardine, R.J., Zhu, B.T., Foray, P., and Yang, Z.X. 2013. Interpretation of stress measurements made around closedended displacement piles in sand. Géotechnique, 63(8), 613–627.

Karlsrud, K., Jensen, T. G., Wensaas Lied, E. K., Nowacki, F., and Simonsen, A. S. (2014) Significant ageing effects for axially loaded piles in sand and clay verified by new field load tests. Proc. Offshore Technology Conference, Houston, TX, USA, OTC-25197-MS. doi:10.4043/25197-MS

Kikuchi, Y., Mizutani, M., and Yamashita, H. 2007. Vertical bearing capacity of large diameter steel pipe piles. *In* Advances in Deep Foundations. pp. 177–182.

Kolk, H.J., Baaijens, A.E., and Vergobbi, P. 2005. Results of axial load tests on pipe piles in very dense

sands: The EURIPIDES JIP. *In* Proceedings of the International Symposium on Frontiers in Offshore Geomechanics, ISFOG. Taylor & Francis, London. pp. 661–667.
Lehane, B.M. 1992. Experimental investigations of pile behaviour using instrumented field piles. Imperial College London (University of London).
Lehane, B.M. 2009. Relationships between axial capacity and CPT qc for bored piles in sand. Proceedings of the 5th International Symposium on Deep Foundations on Bored and Auger Piles, BAP 5. pp. 61–74.
Lehane B.M., Gaudin C., and Schneider J.A. 2005. Scale effects on tension capacity for rough piles buried in dense sand. Géotechnique, 55(10), 709–720.
Lehane, B.M., Jardine, R.J., Bond, A.J., and Frank, R. 1993. Mechanisms of shaft friction in sand from instrumented pile tests. Journal of Geotechnical Engineering, ASCE,119(1): 19–35.
Lehane B.M. (2019). CPT based design of foundations. EH Davis memorial Lecture 2017. Australian Geomechanics, 54(4), 23–48.
Lehane, B.M., Lim, J.K., Carotenuto, P., Nadim, F., Lacasse, S., Jardine, R.J., and van Dijk, B.F.J. 2017. Characteristics of Unified Databases for Driven piles. Proceedings of the 8th International Conference of Offshore Site Investigation and Geotechnics OSIG, doi.org/10.3723/OSIG17.162.
Lehane, B.M. and Randolph, M.F. 2002. Evaluation of a Minimum Base Resistance for Driven Pipe Piles in Siliceous Sand. J. Geotech. and Geoenviron. Engineering, ASCE,128(3): 198–205.
Lehane, B.M. and White, D.J. 2005. Lateral stress changes and shaft friction for model displacement piles in sand. Canadian Geotechnical Journal, 42(4): 1039–1052.
Lim, J.K. and Lehane, B.M. 2014. Characterisation of the effects of time on the shaft friction of displacement piles in sand. Géotechnique, 64(6): 476–485.
Lim, J.K. and Lehane, B.M. 2015. Time effects on the shaft capacity of jacked piles in sand. *Canadian Geotechnical Journal*, 52(11), 1637–1648.
Liu, Z., Nadim, F., Lacasse, S., Lehane, B.M., and Choi, Y.J. 2019. Unified Sand and Clay Database of Pile Load Tests. *In* Offshore Technology Conference. Houston, Texas,pp. 6–9, OTC-29514-MS.
Liu, T.F., Quinteros, V.S., Jardine, R.J, Carraro, J. A. H., and Robinson, J. (2019). A Unified database of ring shear steel-interface tests on sandy-silty soils. Proc. XVII European Conf. Soil Mech. and Geotechnical Engg. Reykjavik, Iceland. doi: 10.32075/17ECSMGE-2019-268.
De Cock, F., Legrand, C., and Huybrechts, N. 2003. Overview of design methods of axially loaded piles in Europe - Report of ERTC3-Piles, ISSMGE Subcommittee. Proc. 8th European Conf. on Soil Mech. and Geotech. Eng., Prague, 663–715.
De Nicola, A. and Randolph, M.F. 1993. Tensile and Compressive Shaft Capacity of Piles in Sand. Journal of Geotechnical Engineering, 119(12): 1952–1973
O'Neill, M.W. 2001. Side Resistance In Piles and Drilled Shafts. Journal of Geotechnical and Geoenvironmental Engineering, 127(1): 3–16.
Ramsey N., Jardine R.J., Lehane B.M., and Ridley A. 1998 A review of soil-steel interface testing with the ring shear apparatus. *Proc. VI Conf. on Offshore site investigation and foundation behaviour*, Soc. for Underwater Technology, London, 237–258.
Randolph, M.F. 2003. Science and empiricism in pile foundation design. Géotechnique, 53(10): 847–875.
Randolph, M.F., Dolwin, J., and Beck, R. 1994. Design of driven piles in sand. Geotechnique, 44(3): 427–448.
Salgado, R., Woo, S. I., and Kim, D. 2011. Development of load and resistance factor design for ultimate and serviceability limit states of transportation structure foundations." *Rep. No. FHWA/IN/JTRP-2011/03*, Indiana Dept. of Transportation and Purdue Univ., West Lafayette, US.
Schmertmann, J.H. 1978. Guidelines for cone test, performance and design. U.S. Federal Highway Administration, FHWATS-78209
White, D., Schneider, J., and Lehane, B.M. 2005. The influence of effective area ratio on shaft friction of displacement piles in sand. Proc. 1st Int. Symp. Frontiers in Offshore Geotechnics, Perth, Australia. pp. 741–747.
White, D.J. and Bolton, M. D. 2005. Comparing CPT and pile base resistance in sand. *Proceedings of the Institution of Civil Engineering - Geotechnical Engineering*, *158*(1), 3–14.
White, D.J. and Lehane, B.M. 2004. Friction fatigue on displacement piles in sand. Géotechnique, 54(10): 645–658.
Xu, X., Schneider, J.A., and Lehane, B.M. 2008. Cone penetration test (CPT) methods for end-bearing assessment of open- and closed-ended driven piles in siliceous sand. Canadian Geotechnical Journal, 45(8): 1130–1141.
Yang, Z.X., Jardine, R.J., Zhu, B.T., Foray, P., and Tsuha, C.H.C. 2010. Sand grain crushing and interface shearing during displacement pile installation in sand. Geotechnique, 60(6): 469–482.