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# Cone Penetration Testing 2018

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A BALKEMA BOOK

# Use of CPT for the design of shallow and deep foundations on sand

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**ABSTRACT:** A wide range of empirical correlations linking the Cone Penetration Test (CPT) end resistance  $q_c$  and the resistance of shallow and deep foundations in sand have been published. Both National and European organisations are attempting to introduce standard methods into practice that unify these approaches. In this paper experimental data and finite element analyses are reviewed to examine the mechanisms governing foundation behaviour in a bid to move towards these unified approaches. For shallow foundations and non-displacement piles, sand creep was found to affect correlations between  $q_c$  and the mobilised bearing resistance. For pile foundations direct correlations between  $q_c$  and pile end resistance that were dependent only on pile installation method are reported. In the case of shaft resistance, constant correlation factors between  $q_c$  and average shaft resistance are possible for non-displacement piles. For the case of displacement piles, correlations that include the effects of friction fatigue are recommended.

## 1 INTRODUCTION

Whilst conventional bearing capacity and earth pressure approaches are widely used to design shallow and deep foundation in sand, many design codes are moving towards Cone Penetration Test (CPT) based design methods. This is as a result of a significant research effort in the area of foundation design in recent years. An issue facing both designers operating internationally and causing debate for those drafting unified codes such as Eurocode 7, is that many national recommendations have been published linking CPT end resistance,  $q_c$  with the bearing resistance of shallow foundations and piles. However, these values are rarely consistent, and in some cases can vary significantly. Whilst some of these differences may be caused by geology, pile types adopted, and a wide range of methods (averaging techniques) for estimating design  $q_c$  profiles, it arises at least in part due to a lack of understanding of the mechanisms controlling foundation behavior. Even internally within some countries, there is ongoing debate surrounding CPT based design.

Recent updates to the Dutch design code have caused significant debate within the geotechnical engineering community. Based on an interpretation of static load test data from across the Netherlands, CUR report 229 proposed a 30% reduction to the pile base reduction factor that will result in larger and longer piles from January 1st 2017. These changes are largely due to a lack of reliable pile load tests data in the Netherlands as the country is unusual in that it does not routinely test the axial capacity of piles. This has been met with

some resistance from industry since historical pile failure rates are very low.

In this paper, the results of experiments performed on foundations in sand and finite element analyses are compiled in an attempt to provide an insight into factors that may influence correlations between CPT  $q_c$  and the behavior of both shallow and deep foundations in sand.

## 2 SHALLOW FOUNDATIONS

### 2.1 General description

Routine design of shallow footings on sand is often undertaken using the conventional bearing capacity approach to calculate the ultimate bearing resistance ( $q_{ult}$ ):

$$q_{ult} = 0.5\gamma B S_\gamma d_\gamma N_\gamma + \gamma D S_q d_q N_q \quad (1)$$

where:  $\gamma$  = unit weight of Soil,  $B$  = foundation width,  $D$  = foundation depth,  $S_\gamma, S_q, d_\gamma$  and  $d_q$  are factors to account for footing shape and depth, whilst  $N_\gamma$  and  $N_q$  are bearing capacity factors which depend on the friction angle ( $\phi$ ) of the soil and the footing shape.

Additional factors can be included to take account of situations such as sloping ground conditions and inclined or eccentric loading, although there is a strong argument that the latter should be treated using interaction diagrams as proposed by Gottardi and Butterfield (1993) and Housley and Cassidy (2002). Designers face key challenges in the application of Equation (1). These include the choice of appropriate shape and depth factors, and most importantly bearing capacity factors:

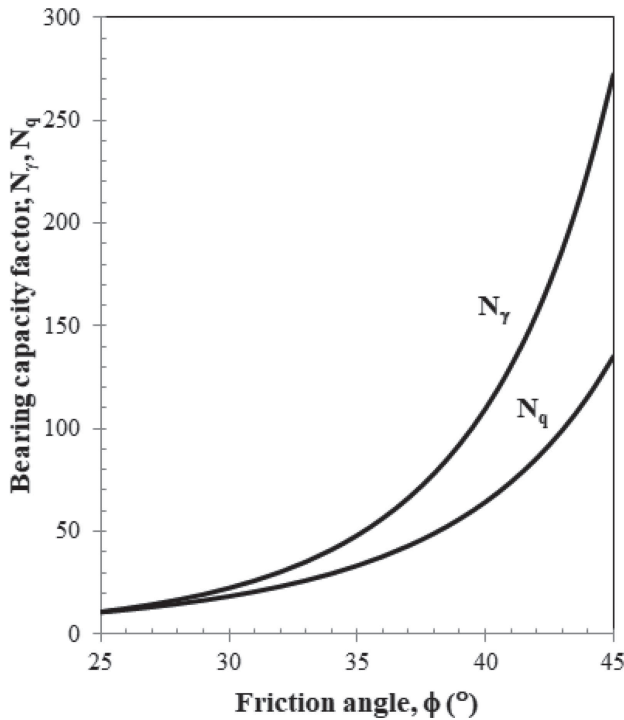


Figure 1. Typical bearing capacity factors for shallow footings.

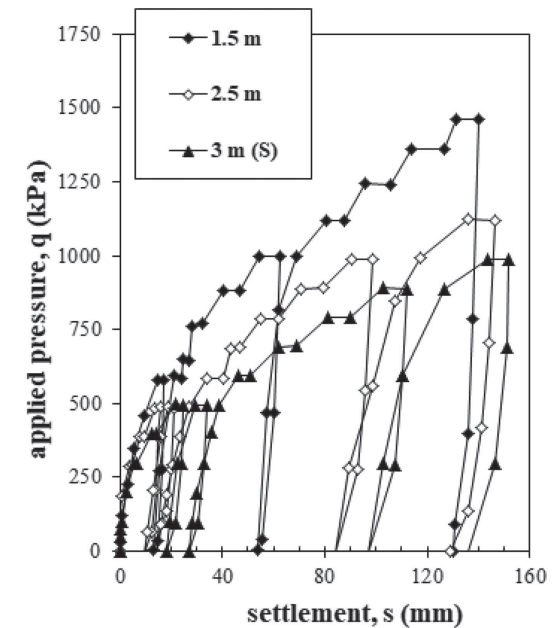
Although Randolph et al. (2004), note that the accuracy with which bearing capacity factors can be determined has increased substantially in recent years, it is clear from Figure 1 that the values increase substantially over the range of soil strength typically encountered. Because of the difficulties in obtaining high quality samples, coupled with the challenges of interpreting soil strength parameters for the complex stress and strain paths experienced during loading at the relatively low stress levels applicable to footing design, greater reliance is placed on using empirical formulae linking  $\phi'$  and  $q_c$  or using direct correlations linking  $q_c$  and the soil bearing resistance.

## 2.2 CPT based methods for shallow foundation capacity

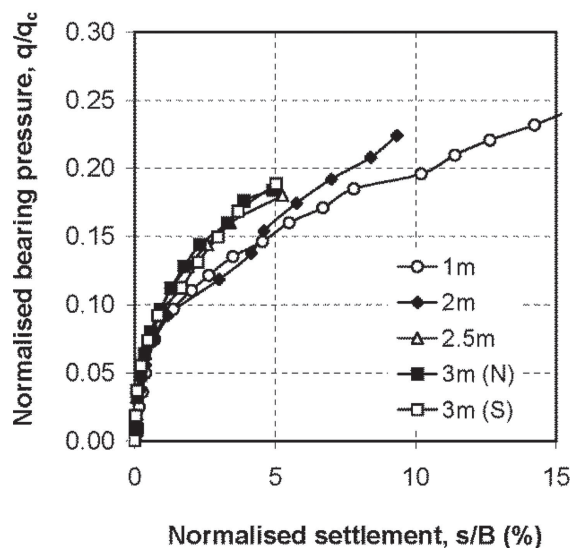
In addition to the challenges outlined above, a further driver towards the use of in-situ test data in shallow foundation design was identified by Briaud (2007) from interpretation of field test data. Briaud and Gibbens (1999) report tests performed on square footings where the footing width,  $B$  varied from 1 m to 3 m. The footings were founded 0.75 m below the ground surface in recently deposited medium-dense sand. The site was in a lightly over-consolidated state ( $OCR \approx 2$ ) following the removal of about 1.0 m overburden. The mean CPT  $q_c$  resistance ranged from 5 to 7.25 MPa in the zone of influence of the footings. The pressure-

settlement response (including un-load and re-load loops) of three of the footings is illustrated in Figure 2a. It is apparent that for a given settlement  $s$ , the mobilised bearing pressure,  $q$  reduced with increasing footing width. Another feature of the test was that for values of  $q$  greater than 650 kPa, significant creep settlement occurred during maintained load steps.

When the mobilised pressure was normalised by the  $q_c$  value averaged over the zone of influence, and the footing settlement divided by the footing width, a relatively unique normalised pressure-settlement response was obtained for the site, see Figure 2b. Noting that the unload-reload portions



(a)



(b)

Figure 2. (a) Pressure-Settlement response for three footings (b) normalised pressure-settlement response.

of the data were removed for clarity and the maximum settlement for each load step were plotted, as it is clear from Figure 2a that the shape of the curve is affected by the load test procedure. Adopting a definition of failure as being the pressure mobilised when the footing settlement reached 10% of the footing width,  $q_{0.1}$  the data suggests the adoption of a direct method for estimation of the footing resistance of the form given in Equation 2 with an  $\alpha$  value of 0.20.

$$q_{0.1} = \alpha q_c \quad (2)$$

Briaud (2007) considered data from other test sites to investigate the effect of footing depth,  $D$  and found that the normalised pressure-settlement response was independent of both footing width and relative embedment ( $D/B$ ). He suggests that whilst Equation 1 would produce good estimates of  $q_{ult}$  of footings in soil profiles where the soil strength increased linearly with depth, in over-consolidated sand or in deposits where the near surface soil is unsaturated, the soils strength is often relatively constant with depth and the assumption that  $q_{ult}$  increases with footing width,  $B$  or footing depth,  $D$  is not valid.

Based on a review of footing tests from the literature, Eslaamizaad and Robertson (1996) found that the back-figured  $\alpha$  values varied with soil density, relative embedment and footing shape. Randolph et al. (2004) summarised the results of laboratory tests, field tests and numerical analyses performed on shallow footings and buried piles. They report a relatively wide range of  $\alpha$  values varying from 0.13–0.21. However, there was no evidence that  $\alpha$  varied with footing width or sand state.

Finite Element (FE) analyses present an ideal environment in which to consider the mechanisms governing footing response and to perform sensitivity analyses for foundations. Lee and Salgado (2005) reported a significant study using the FE program ABAQUS in which they investigated the effect of footing width and relative density ( $D_r$ ) on the mobilised bearing resistance. The CPT  $q_c$  values used to normalise the footing resistance were derived from a different program CON-POINT (Salgado and Randolph, 2001). Their analyses shown in Figure 3 indicate that  $\alpha$  decreased when the relative density of the soil increased and the footing width reduced. The rate at which  $\alpha$  increased with the footing width depended on the relative density of the soil, with an increase of 35% being noted for  $D_r = 90\%$  when the footing width increased from 1 m to 3 m, whilst the increase was only 5% for  $D_r = 30\%$ .

The variability in suggested  $\alpha$  values from previous studies arises at least in part due to the limited

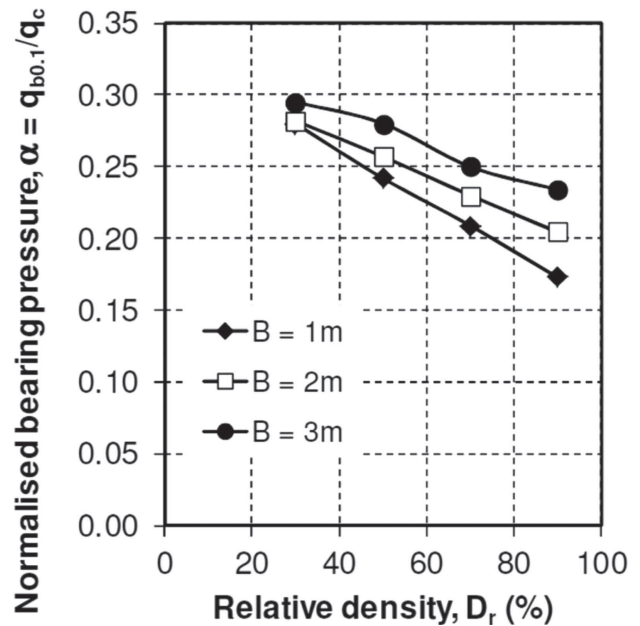
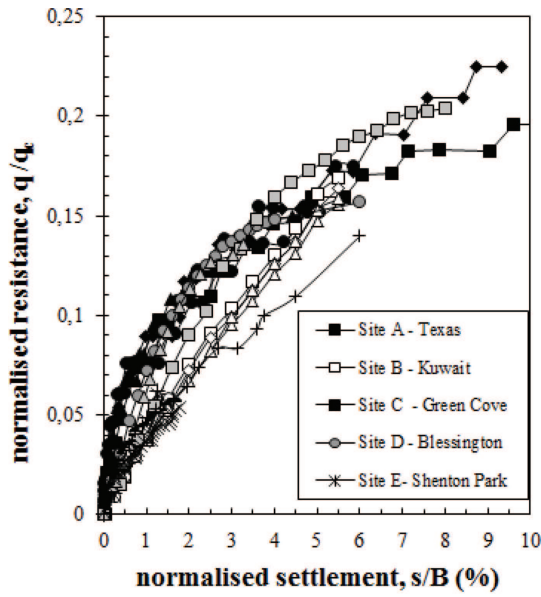


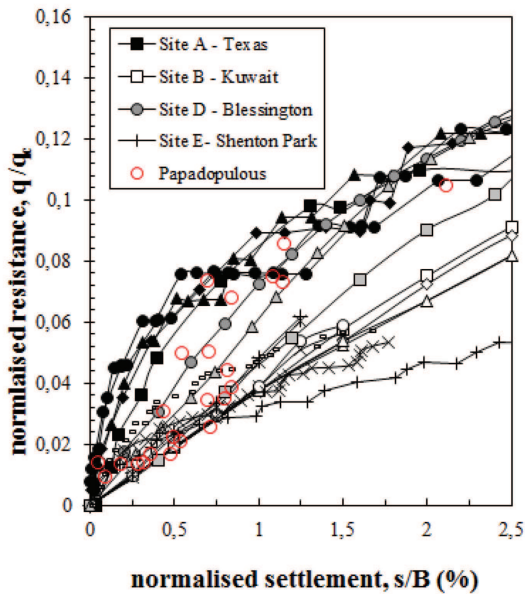
Figure 3. Normalized Base Resistance  $q_{b0.1}/q_c$  versus Relative Density  $D_r$ ; (after Lee and Salgado 2005).

database of footing tests from which to investigate factors which might influence the footing response. Large-scale footing tests are expensive and time consuming, and therefore, most field tests consider a relatively limited range of either footing width and/or depths in sand where the relative density is relatively constant. Gavin et al. (2009) compiled data from 22 field experiments on model and full-scale footings at five test sites. Data from the load tests performed by Briaud and Gibbens (1999) in Texas, Ismael (1985) in Kuwait, Anderson et al. (2007) at Green Cove, Gavin et al. (2009) from Blessington and Lehane et al. 2008, from Shenton Park were collated. The footing width varied from 0.1 m to 3 m, the  $q_c$  value (average over  $\pm 1.5B$ ) was in the range 3,330 to 14,500 kPa, and the foundation depth varied from 0.1 m to 2 m below ground level, bgl. The normalised pressure-settlement response is set out in Figure 4a.

Whilst many of the footing tests did not reach the normalised displacement level of 10%, the trend is for all sites to converge as  $s/B$  increases and tend towards  $\alpha$  of approximately 0.2 at  $s/B = 10\%$ . However, the initial stiffness response at the five sites varied considerably, and the data is recast in Figure 4b in order to examine the rate of mobilisation of the bearing resistance. The data confirms that the initial pressure-settlement response is site dependent, with a tendency for the normalised resistance to mobilise most quickly for the tests in Texas and most slowly at Shenton Park. Also included in Figure 4b are foundation settlement measurements from 23 buildings as collated by Papadopoulos (1992). The measurements relate



(a)



(b)

Figure 4. (a) Pressure settlement response from five test sites (b) Initial response of footing tests (after Gavin et al. 2009).

to foundations with  $B$  between 0.5 and 21.7 m, founded in sand where  $q_c$  ranged from 1800 to 19,000 kPa. Despite the range of parameters considered, the discrete measurements of maximum settlement are bounded by the experimental data presented in Figure 4b.

To gain further insight into the mobilisation rate, the rate of degradation of the secant stiffness measured at four sites where small strain stiffness data was available are compared in Figure 5. It is of interest to compare the normalised settlement level at which the secant stiffness reduces to 20% of the small strain Young's modulus ( $E'_s/E_0 = 0.2$ ). This occurs at  $s/B = 0.2\%$ , 1% and 10% at Shenton

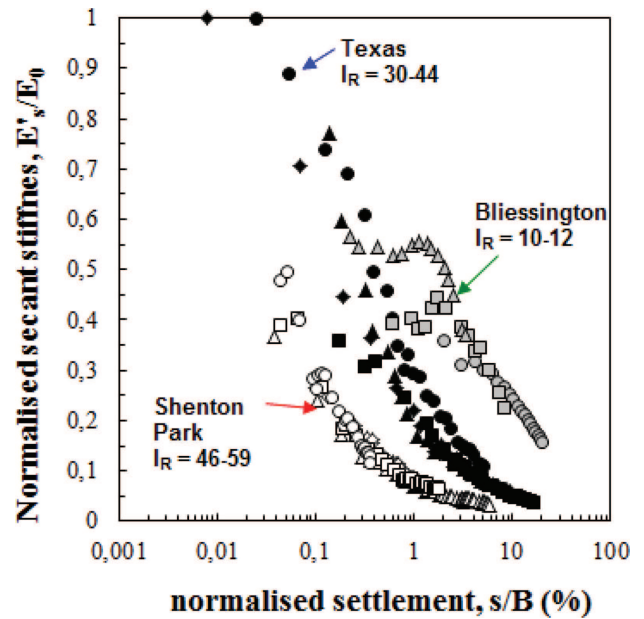


Figure 5. Variation of normalised secant stiffness with normalised settlement.

Park, Texas and Blessington respectively. The stiffness degradation rate appears to be related to the rigidity index,  $I_R$  (ratio of small strain shear modulus,  $G_0$  to soil strength,  $q_c$ ) with high  $I_R$  values resulting in rapid stiffness degradation.

The relatively low applied pressure imposed by most regular structures ensures that the factor of safety against ultimate failure of these foundations is high. As a result of this, the focus of designers is often to accurately predict settlement under working stresses. Settlement prediction models that require an estimate of the secant stiffness  $E'_s$  of sand are in common use. A number of direct correlations between  $E'_s$  and  $q_c$  have been proposed:

$$E'_s = \beta q_c \quad (3)$$

Das (1983) compiled  $\beta$  values recommended by 13 sources that ranged from 1 to 3. However, Lehane et al. (2008) and others state that at relatively small strain levels there is a weak dependence between  $E'_s$  and  $q_c$ , with the effects of stress level and ageing being dominant. Given that many foundations are designed for a maximum settlement often of the order of 25 mm, the resulting normalised settlement beneath foundations of width between 1 m and 10 m, results in  $s/B\%$  in the range 0.25–2.5%. The variation of  $\beta$  values mobilised from the field tests described above, and shown in Figure 6, shows a very large range of operational  $\beta$  values at these normalised settlement levels, confirming that constant  $\beta$  values should be used with caution. It is worth noting that at the settlement levels typical for full-scale foundations,  $\beta$  values are typically higher than 3.

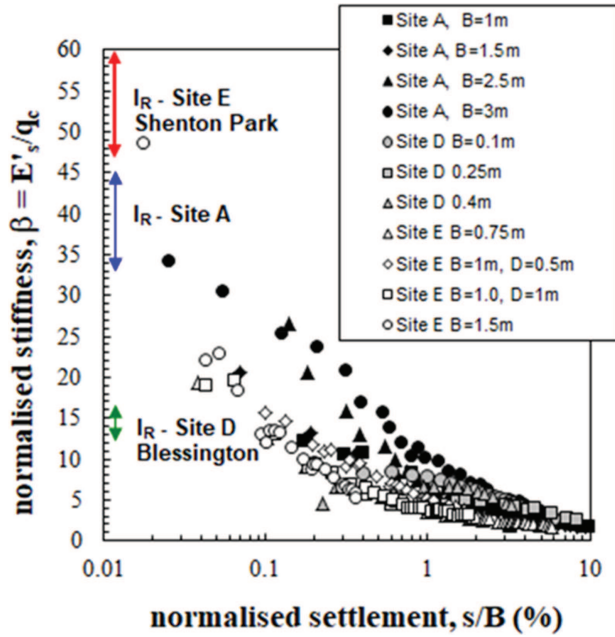


Figure 6. Variation of  $\beta$  ( $= E'_s/q_c$ ) with normalised settlement.

### 2.3 Summary

Due to the difficulty of sampling and testing the strength and stiffness properties of sand, direct correlation between the ultimate bearing capacity and settlement response of footings on sand and in-situ testing such as CPT offers a viable alternative that appears to give consistent results.

Data from field tests presented by Gavin et al (2009) suggest that the normalised bearing pressure normalised by CPT  $q_c$ , mobilised at a footing settlement of 10% of the footing diameter,  $\alpha$  is 0.2. This agrees reasonably well with the findings from a larger database study by Mayne et al. (2012) that considered a wider range of sand types and foundation size and concluded that a constant  $\alpha$  value of 0.18 was appropriate. Both studies propose simple non-linear elastic models that allow the complete pressure-settlement response of the foundation to derived thus providing important insights into the operation behaviour of footings, and particularly the relatively large stiffness response at typical operation pressure levels.

## 3 BASE RESISTANCE OF PILES

### 3.1 Background

In many design guides and codes the end bearing resistance ( $q_b$ ) mobilised by a deep footing in sand is calculated using a reduced form of Equation 1:

$$q_b = N_q \sigma'_v \quad (4)$$

Table 1. Recommendations for bearing capacity factors and limiting values for offshore piles (API 2007).

Density	$N_q$	$q_b$ limit (MPa)
Loose	–	
Medium	20	5
Dense	40	10
Very Dense	50	12

where  $N_q$  is a bearing capacity factor and  $\sigma'_v$  is the vertical effective stress at the pile base.  $N_q$  factors typically vary with  $\phi'$  or sand density as shown in Table 1. Another common feature is the adoption of limiting upper-bound values. Randolph and Gouverneur (2011) note that these limiting values are included as field data suggests that  $N_q$  reduces with increasing stress level.

The  $N_q$  values recommended for bored and cast in-place piles are typically assumed to be 50% of those derived for driven piles.

### 3.2 Closed-ended driven piles

Because of the similarities between the penetration mechanisms and the geometry of closed-ended piles and the CPT, a number of empirical correlations between the CPT end resistance  $q_c$ , and  $q_b$  have been proposed. Recent design methods for driven closed-ended or full-displacement piles in sand have been shown to have a relatively high reliability (Chow 1997, Gavin 1999 and Lehane et al. 2005) and have been widely accepted in industry. These techniques generally estimate the base resistance at relatively large pile base settlement ( $s_b$ ), typically at 10% of the pile diameter,  $q_{b0.1}$ , using an empirical reduction factor  $\alpha_p$ :

$$q_{b0.1} = \alpha_p q_c \quad (5)$$

Based on a database study Jardine et al. (2005) suggest that  $\alpha_p$  reduced from 0.63 to 0.43 as the pile diameter increased from 200 mm to 500 mm, Lehane et al. (2005) found that an  $\alpha_p$  value of 0.6 gave the best-fit to a database of instrumented pile load tests with diameters ranging from 0.2 m to 0.68 m. Randolph (2003) and White and Bolton (2005) argued that once appropriate averaging techniques were adopted to derive design  $q_c$  values and the effects of residual loads were accounted for, a constant  $\alpha_p$  factor can be adopted which is independent of pile diameter and tended towards  $q_b = q_c$  at large pile base displacements.

### 3.3 Open-ended driven piles

For partial displacement (open-ended) pipe piles, model and full-scale pile tests reported by Lehane

and Gavin (2001) and Foye et al. (2009) show that direct correlations between  $\alpha_p$  (based on the average pressure mobilised over the entire pile base area) and  $q_c$  which are independent of pile diameter or sand state can be determined once the degree of plugging during pile installation are included. The plugging behaviour is best quantified through the incremental filling ratio (IFR), which compares the rate of soil intrusion during pile installation with IFR = 1 for a fully coring pile (which causes minimal disruption) and IFR = 0 for a pile with a fully formed plug, which prevents soil intrusion and results in what is effectively a closed-ended pile.

However, in reality, plug development during installation is rarely recorded and the final plug length ratio, PLR = soil plug length/pile embedment depth is only known. In the field, plugging appears to be controlled by pile diameter, sand density and installation method. The majority of large diameter ( $D > 500$  mm), piles driven offshore appears to have PLR values close to one. Significant plugging seems to occur only in the case of smaller diameter piles, or piles installed by jacking.

Gavin and Lehane (2005) present data shown in Figure 7 of plug stress mobilised during jacking strokes (closed-symbols) and static load tests (open-symbols) for pipe piles in jacked into loose sand. The normalised plug stress varies linearly with IFR from a minimum value in the range 0.15 to 0.2 that is in keeping with values suggested by Lehane and Randolph (2002) from numerical analyses of fully-coring piles. As IFR reduced,  $q_{\text{plug}}/q_c$  increased linearly. The trend was shown to be consistent over a range of installation methods (driving and jacking) and sand state (Gavin and Lehane 2003).

Lehane and Gavin (2001) report measurements of the annular stress mobilised during installation

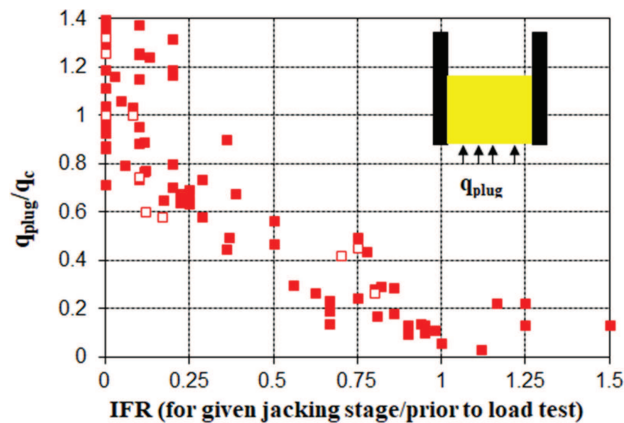


Figure 7. Effect of Incremental Filling Ratio on the plug stress mobilised by open-ended piles in sand (after Gavin and Lehane 2005).

of open-ended piles that were independent of IFR (with  $q_{\text{ann}} = q_c$ ). Lehane et al. (2005) propose that the average base stress (combining the plug and annular components) mobilised for an open ended pile be calculated using Equation 6:

$$q_{b0.1} = 0.15 + 0.45 A_{r,\text{eff}} \quad (6)$$

where  $A_{r,\text{eff}} = 1 - \text{FFR} (D_i/D)^2$ , and FFR is the IFR value at the end of driving.

Equation 6 suggests that  $\alpha_p$  increases from  $\approx 0.15$  for a fully coring pile (IFR = 1) to 0.6 for a fully-plugged pile.

### 3.4 Partial displacement screw injection piles

Funderingstechnieken Verstraeten bv. performed three static compression load tests on screw injection piles installed in dense sand at a site in Terhausen, Netherlands. The piles which had shaft diameters of 0.46 m, base diameters of 0.56 m and embedded lengths of between 20.2 m and 20.3 m were instrumented with strain gauges along their length. The purpose of the test was to assess recent changes in pile capacity factors introduced in the Dutch standard NEN-EN 9997-1 introduced on January 1st 2017 that reduced the  $\alpha_p$  values for all piles by 30%. The result was that for screw injection piles the value decreased from 0.9 to 0.63.

Estimates of the pile capacity at the test site were made using the pre-2017 design value and the piles were load tested to this capacity in an attempt to validate the old design approach. The CPT profiles at the test site are shown in Figure 8. The estimated pile capacity ranged from 5750 to 6100 kN. The load test on pile 1 was terminated an applied load of 5874 kN when the pile head settlement reached 23 mm (i.e. less than 5% of the pile diameter). The other piles were loaded up to failure at ultimate loads of 6096 kN (Pile 3) and 6312 kN (Pile 5) causing pile displacements of 60 mm.

The base pressure-settlement response of the piles is illustrated in Figure 9, showing that the initial stiffness response of the three piles was similar. Pile 1 mobilised the highest resistance, 13.7 MPa despite not reaching failure. Pile 3 mobilised an ultimate base resistance of 12 MPa and Pile 5 had the lowest resistance of 10 MPa.

The back-figured  $\alpha_p$  value depends on the design average  $q_c$  value adopted. A comparison of values derived from the Dutch averaging method, where  $q_c$  is averaged over a distance  $-8B$  to  $+4B$  and assuming  $q_c$  averaged over a distance  $\pm 1.5B$  above the pile base are shown in Table 2. Adopting the Dutch averaging technique the measured  $\alpha_p$  is 1.12, 80% higher than the current value (of 0.63). Using the  $\pm 1.5B$  suggests that the ultimate end bearing resistance of these piles approximates  $q_c$ .

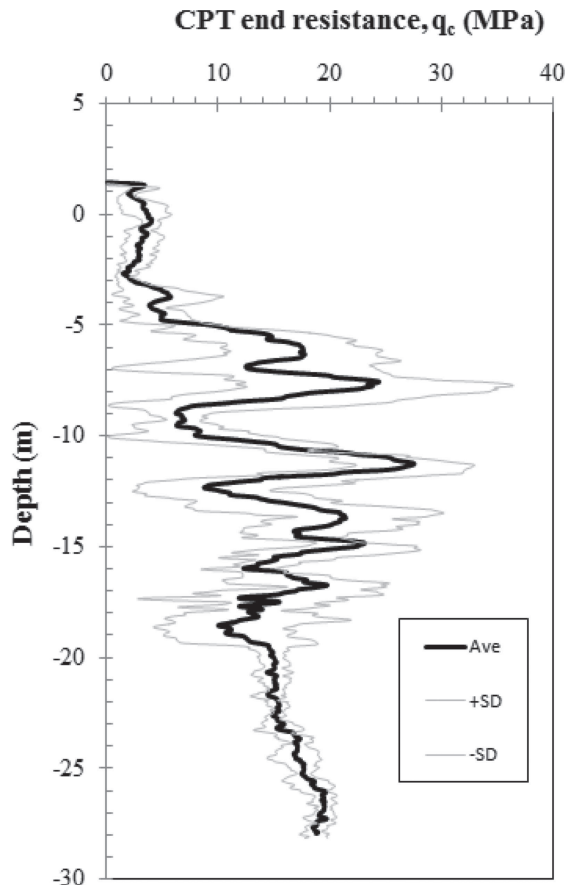


Figure 8. CPT profile at Terhausen test site.

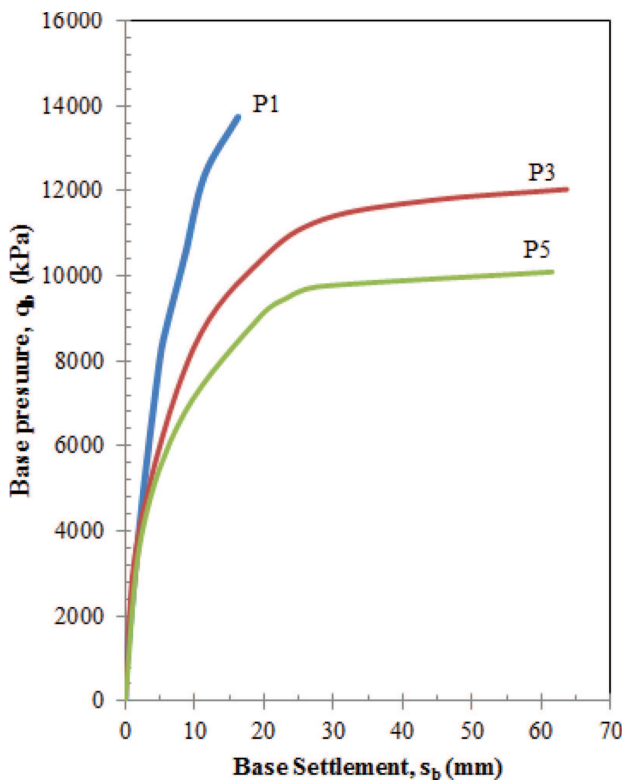


Figure 9. Base resistance mobilised by grout injection piles at Terhausen test site.

Table 2. Comparison of base capacity factors back-figured from Terhausen test site using different CPT averaging techniques.

Pile	$q_c$ Dutch (MPa)	$\alpha_{p,Dutch}$	$\pm 1.5B$	$\alpha_{p,Dutch}$
Pile 1	9.75	1.40	12	1.23
Pile 3	10.90	1.10	15	0.80
Pile 5	11.60	0.86	11.8	0.85
Average		1.12		0.96

### 3.5 Bored piles

The earliest correlation between the end bearing resistance of bored piles and  $q_c$  were suggested by (Meyerhof 1956; Meyerhof 1976; Meyerhof 1983). Based on theoretical and experimental studies on deep foundations, he introduced two reduction factors,  $C_1$  for scale effects and  $C_2$  to account for shallow penetration into dense strata to predict the ultimate end bearing resistance ( $q_{bu}$ ) of a bored pile:

$$q_{bu} = 0.3 q_c C_1 C_2 \quad (7)$$

The LCPC or French method (Bustamante and Gianeselli 1982) was developed from a database of 197 pile tests on driven and bored piles in a range of soil conditions. An  $\alpha_p$  value of 0.2 is recommended for bored piles in sand and gravel. The design  $q_c$  is the average from  $\pm 1.5B$  having excluded values that are in excess of  $\pm 30\%$  of the average. De Cock et al. (2003) compiled a review of pile design practice in Europe and found that  $\alpha_p$  commonly ranged between 0.15 and 0.2, and typically did not depend on the diameter or the length of the pile. Eurocode 7, Part 2, suggests that  $\alpha_p$  values, which although independent of footing width and depth, reduce from 0.2 for  $q_c$  values up to 15 MPa, to 0.16 for  $q_c = 20$  MPa. Lehane (2008) reported a database study of Continuous Flight Auger (CFA) piles where the pile length varied from 4 m to 10.5 m. The database contained both straight and expanded base piles and recorded  $\alpha_p$  values that increased from approximately 0.15 for 4 m long piles, to approximately 0.4 for a 10.5 m long pile.

Gavin et al. (2013) compiled a pile test database comprised of 20 static maintained load tests performed on non-displacement piles installed in sand where the piles were loaded to settlements in excess of 10% of the pile diameter. The diameter  $B$  of the piles ranged from 0.2 m to 1.5 m, while their length  $L$  ranged from 4 m to 26.5 m, with  $L/B$  in the range 4 to 37. They were founded in sand where the CPT end resistance,  $q_{c,1.5B}$  value ranged between 2 MPa and 40 MPa.

In the assessment of mobilised  $\alpha_p$  values, the design  $q_{c(1.5B)}$  was adopted. The variation of  $\alpha_p$  with pile diameter, length, relative density and CPT  $q_c$  value is shown in Figure 10. Although there is considerable scatter in the results, there is no suggestion that  $\alpha_p$  varied in a consistent manner with any of the parameters considered in the assessment, with an average  $\alpha$  value for the database piles of 0.24.

Tolooiyan and Gavin (2013) performed finite element analysis using PLAXIS to investigate the factors affecting  $\alpha_p$  for bored piles in sand. The sand was modelled using the Hardening Soil (HS) model described by Schanz et al. (1999) with

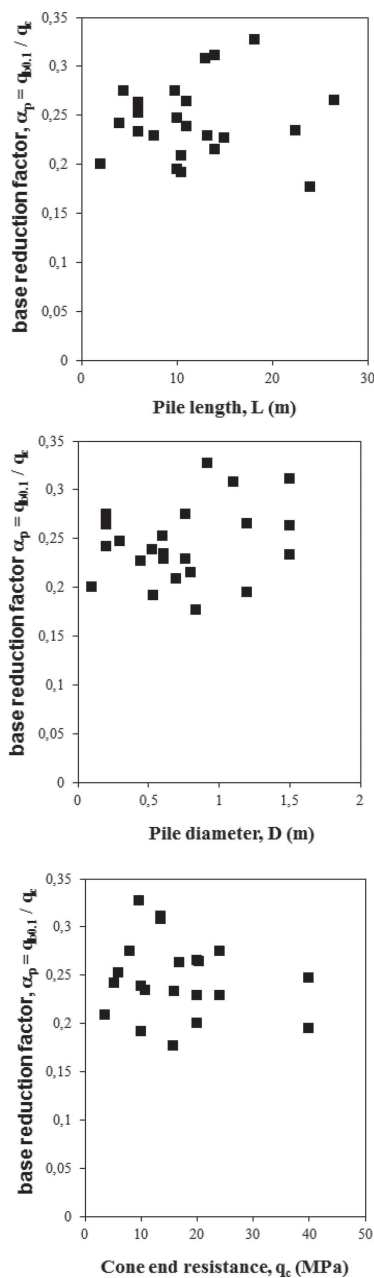


Figure 10.  $\alpha_p$  values back-figured from load test database (a) effect of pile length, diameter, and strength.

the HS model parameters calibrated from lab-test data. Cavity expansion analyses were performed using a procedure described by Xu and Lehane (2008) and Tolooiyan and Gavin (2011) in order to predict the CPT  $q_c$  profile for Blessington sand. The FE generated values are compared in Figure 11 with a four typical  $q_c$  profile measured at Blessington. It is clear that the FE model provided a reasonably good albeit lower-bound estimate of the measured CPT  $q_c$  profile.

The effect of pile width, B on the bearing pressure mobilised by piles in Blessington sand was considered by analysing a pile of fixed length of 6 m with a diameter ranging from 0.2 m to 0.8 m. The piles were loaded until the settlement reached 10% of the pile diameter, see Figure 12a. The maximum end bearing resistance ( $\approx 5,500$  kPa) of all piles was similar. However, the settlement required to achieve this resistance increased in proportion to the pile diameter. The normalised pressure-settlement response is shown in Figure 12b. This reveals that the normalised stiffness response for the piles was very similar, with  $\alpha = 0.31$ .

Considering the effect of pile length in Figure 13, a tendency for a slight reduction in  $\alpha_p$

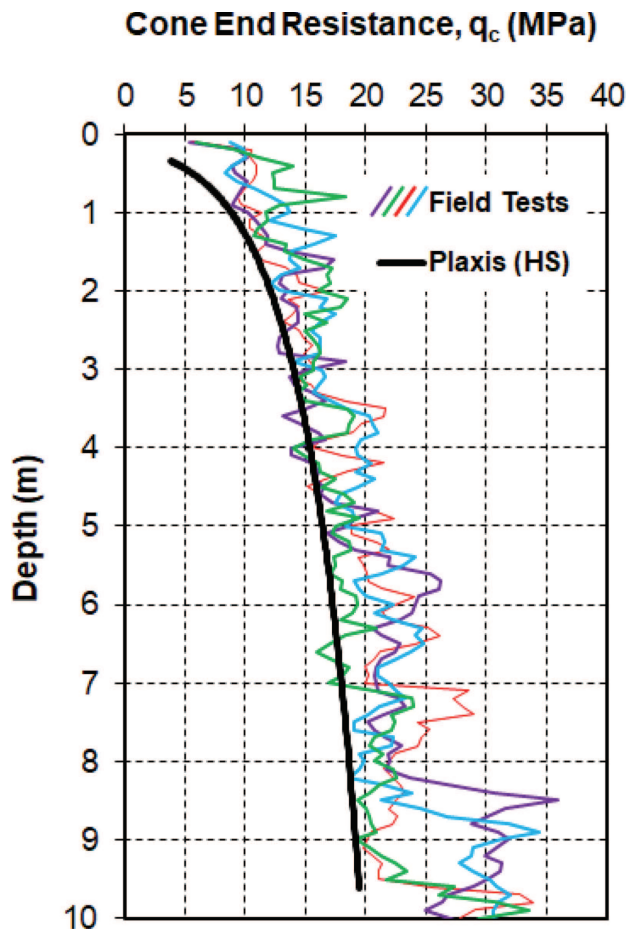


Figure 11. Measured and predicted CPT profile at Blessington.

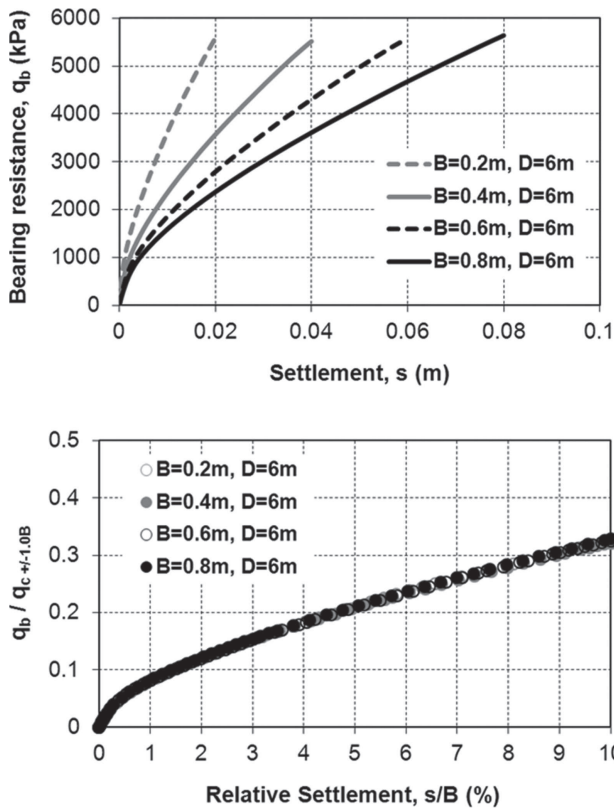


Figure 12. Bearing pressure mobilised by piles installed in Blessington (a) effect of pile width; (b) normalised pressure-settlement response.

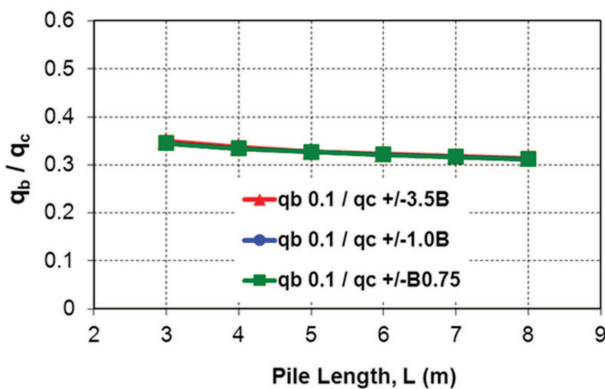


Figure 13. Effect of pile length  $L$  on normalised bearing pressure.

with depth is noted with the value decreasing from 0.34 to 0.31 as the pile length increased from 3 m to 8 m. The influence of CPT averaging technique (from  $\pm 0.75B$  to  $\pm 3.5B$ ) technique is also considered. However, for Blessington sand the effect is negligible.

In order to investigate the effect of sand density on the mobilised  $\alpha$  factor, sensitivity analyses were performed using three sand deposits. These were Tanta sand from Egypt which has an in-situ relative

density  $D_r = 75\%$  (reported by El Sawwaf (2005); El Sawwaf (2009)), Monterey sand from the United States with  $D_r = 65\%$  (Wu et al. (2004) and Yang et al. (2008)), and Hokksund sand from Norway with  $D_r = 50\%$  (Tefera et al. 2006). Synthetic CPT  $q_c$  profiles were derived for these deposits using the procedure as for Blessington sand. The predicted CPT  $q_c$  profiles are illustrated in Figure 14.

The sensitivity analyses were performed using a fixed pile diameter ( $B = 0.4$  m) and a pile length which varied from 4 m to 8 m. The pressure-settlement values predicted are shown in Figure 15. The highest  $q_b$  values were measured in Tanta sand and the lowest were in Hokksund sand. The  $q_b$  values at all sites increased as the pile length increased from 4 m to 8 m. The increase was in the range 56% to 77%, with the highest increase being in Tanta sand.

When the bearing pressure mobilised at a pile base settlement of 10% was normalised by the CPT  $q_c$  value averaged over  $\pm 1B$ ,  $\alpha_p$  values in the range 0.32 to 0.33 were determined. There were clear variations in the rate of mobilization at low settlement levels across the sites, with the normalised resistance developing more slowly as the relative density increased.

The influence of CPT averaging technique and pile length on  $\alpha_p$  is considered in Figure 16. Consistent

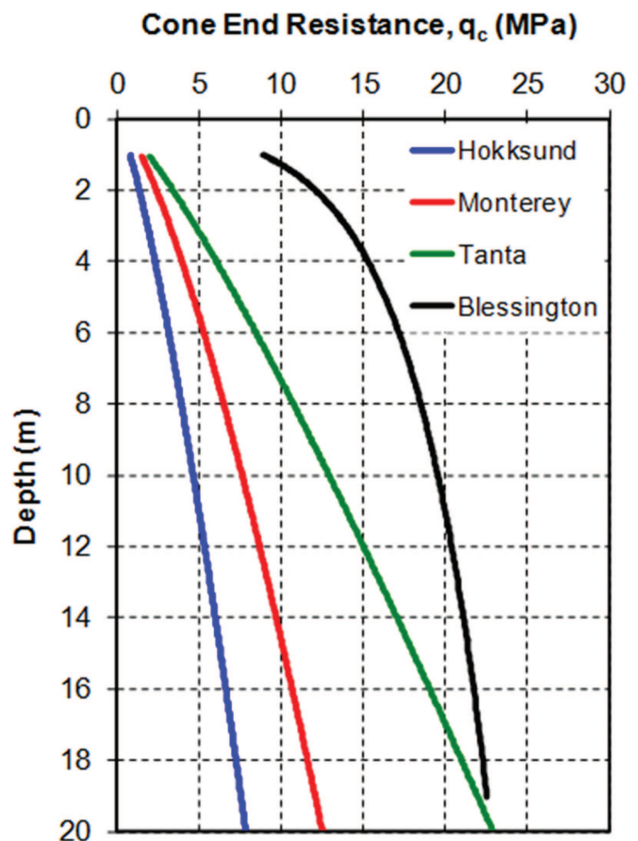


Figure 14. FE derived CPT  $q_c$  profile for Hokksund, Monterey and Tanta sand.

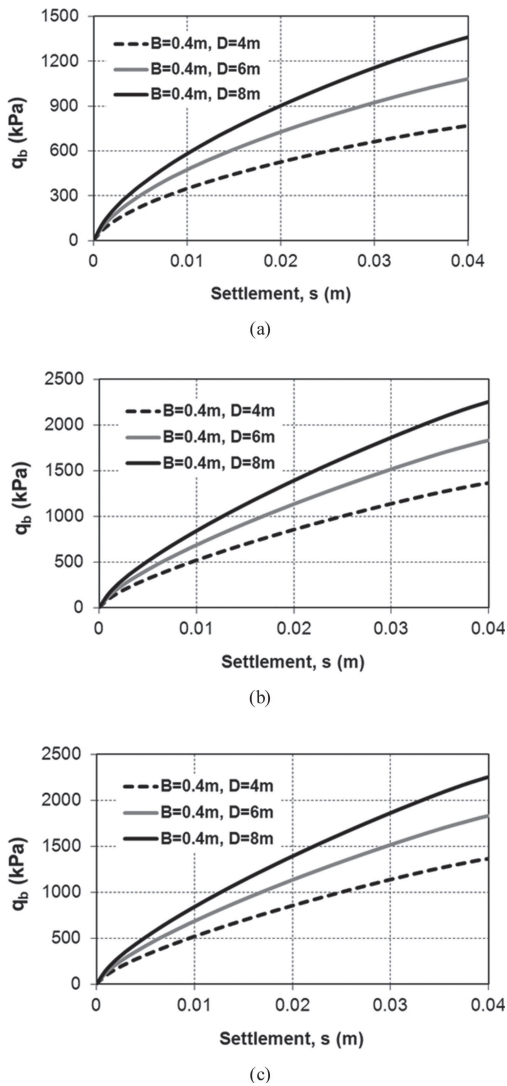


Figure 15. Pressure-settlement values predicted for: (a) Hokksund; (b) Monterey; (c) Tanta sand.

trends were found for all sites where  $\alpha_p$  was reasonably constant when  $q_c$  was averaged over equal distances above and below the pile base, see Figure 16a. When  $q_c$  was averaged with a bias for values either above or below the pile base, See Figure 16b, higher variability and a more pronounced length bias was observed with the highest  $\alpha_p$  values being inferred when using the Dutch average technique.

On the basis of these analyses, it appears that an approximate constant  $\alpha_p$  factor of 0.3 would produce reasonable estimates of the end bearing resistance of bored piles in sand. This value is 50% higher than values typically used in practice and 20% higher than the  $\alpha_p$  value of 0.24 inferred from the database study. To investigate one possible reason for the difference between the field response and FE analyses, it is of interest to consider the effect of loading rate on the mobilisation of  $\alpha_p$ . Gavin et al. (2013) describe load tests performed

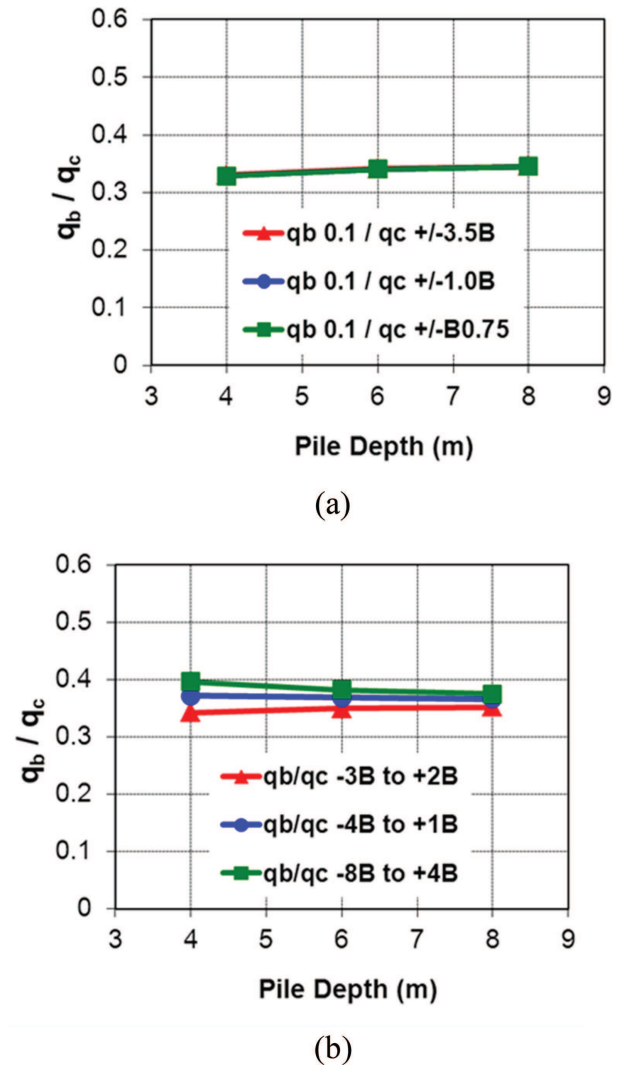


Figure 16. Effect of CPT averaging technique and pile length on  $\alpha_p$  values in Hokksund Sand.

on instrumented continuous flight auger piles installed in dense sand in Killarney, South-West Ireland. Load tests were performed on two piles, a 450 mm diameter, 15 m long pile and an 800 mm diameter, 14 m long pile. The load test procedure involved a maintained load test (MLT) followed by a fast-loading, constant rate of penetration (CRP) test. The results of the MLT portion of the load test are shown in Figure 17a where it is clearly evident that when the applied base pressure exceeded 1500 kPa, the piles experienced creep during load increments. When the normalised base displacement reached 10% of the pile diameter, the  $\alpha_p$  factor approached 0.24.

When the piles were reloaded in the CRP test (See Figure 17b), significantly higher base resistance was mobilised. Whilst the loading history would affect the initial pressure-settlement response, the  $\alpha_p$  factors mobilised in the fast loading tests (where

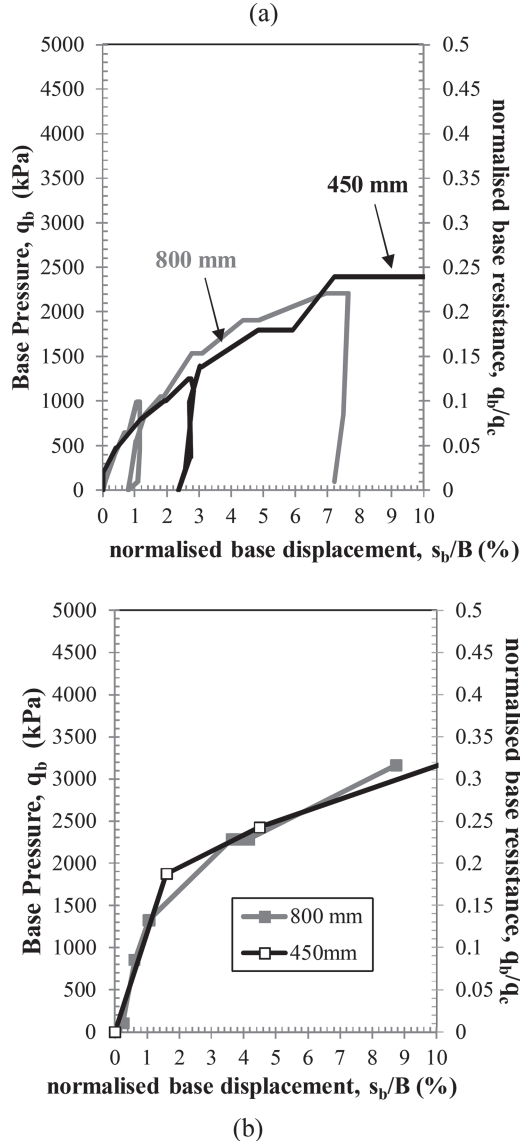


Figure 17. Effect of loading rate or test method on  $\alpha_p$  mobilized by CFA piles in sand.

creep effects were minimised) exceeded 0.31. The soil models used in the numerical analyses did not model sand creep which clearly affected the MLT result and thus the mobilised base resistance.

## 4 SHAFT RESISTANCE OF PILES

### 4.1 Background

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

$$\tau_f = K \sigma'_v \tan \delta_f \quad (8)$$

where  $K$  is the earth pressure coefficient,  $\sigma'_v$  is the in-situ vertical effective stress, and  $\delta_f$  is the soil-

pile interface friction angle. As with all earth pressure approaches, the difficulty with the application of Equation 8 is in the choice of an appropriate  $K$  value for design. For bored piles, Reese and O'Neill (1999) suggest  $K/K_0$  (where  $K_0$  is the coefficient of earth pressure at rest), varies with the pile construction method, varying from 0.67 for a pile excavated using slurry, to 1.0 for a pile formed in a dry excavation.  $K_0$  is notoriously difficult to measure, but may be estimated using the method proposed by Kulhawy and Mayne (1990):

$$\begin{aligned} K_0 &= 1 - \sin \phi' - \text{Normally Consolidated} \\ K_0 &= (1 - \sin \phi') \text{OCR}^{\sin \phi'} - \text{Over Consolidated} \end{aligned} \quad (9)$$

where  $\phi'$  is the friction angle and  $OCR$  is the Over-Consolidation Ratio.

For displacement piles,  $K$  will be changed during pile installation due to large stress changes as the pile base moves through the penetration depth and friction fatigue occurs during cyclic installation. Values in the range  $K = 1-2$  are often used in practice. Due to uncertainties regarding input parameters such as  $\phi'$ ,  $OCR$  and  $\delta_p$ , and the effect of installation method, the use of CPT based design methods cone Penetration Test (CPT) are increasing:

$$\tau_f = \alpha_s q_c \quad (10)$$

A range of  $\alpha_s$  values that depend on pile type are recommended in many design codes, with values typically being lowest for bored piles and highest for displacement piles. For example, in the Netherlands NEN 9997-1-2016 recommends  $\alpha_s$  value that range from 0.06 for bored or driven open-tube piles, to 0.01 driven concrete or closed-ended steel tubes and up to a maximum of 0.014 for a driven cast-in-place piles. In Belgian practice, summarized by Huybrechts et al. (2016),  $\alpha_s$  depends on  $q_c$ , pile type and roughness. A limiting maximum shaft resistance of 150 kPa is imposed for  $q_c$  values  $> 20$  MPa.

Intensive research on displacement piles reported in Jardine et al. (2005) and Lehane et al. (2005) show that the local shaft resistance is given by:

$$\tau_f = (\sigma'_{hc} + \Delta \sigma'_{hd}) \tan \delta_f \quad (11)$$

where:  $\sigma'_{hc}$  is the fully equalized horizontal effective stress after pile installation and  $\Delta \sigma'_{hd}$  is a component derived by dilation during loading.

Chow (1997) examined profiles of  $\sigma'_{hc}$  recorded by an instrumented pile installed at two sites and found that  $\sigma'_{hc}$  values at a given location on the pile were almost directly proportional to the  $q_c$  value at that level and the distance from the level to the

pile base ( $h$ ) normalised by the pile diameter ( $D$ ). These findings were incorporated into the widely used design method for displacement piles known as the Imperial College (IC-05) design method (Jardine et al. 2005) and a similar approach known as the University of Western Australia (UWA) method (Lehane et al. 2005), where:

$$\sigma'_{hc} = 0.03 q_c (h/D)^{-0.5} \quad (12)$$

A minimum  $h/D$  value of 2 should be used in Equation 12.

Equation 12 suggests that  $\alpha_s$  is highest near the pile tip and reduces with increasing distance from the pile tip. Lehane (1992) suggests that the dilation induced increase in horizontal stress ( $\Delta\sigma'_{hd}$ ) could be predicted using cavity expansion theory:

$$\Delta\sigma'_{hd} = \frac{4 G \delta_h}{D} \quad (13)$$

where  $G$  is the shear modulus of the soil mass and  $\delta_h$  is the horizontal displacement of a soil particle at the pile-soil interface.

The IC-05 and UWA design approaches have been shown to provide more reliable estimates of the shaft capacity of piles and accurate predictions of the distribution of mobilised local shear stress on closed-ended displacement piles by Chow (1997), Gavin (1998), Schneider (2007) and others. Given the prevalence of driven open-tube piles, particularly in the offshore sector, both the IC-05 and UWA methods allow for a reduction of shaft stress due to the lower displacement resulting from installation of these piles. Gavin et al. (2011) note that whilst the two approaches give very similar predictions for shaft capacity of closed-ended piles, differences in how they address the issue of plugging can result in significantly different estimates for open-ended piles.

#### 4.2 Driven cast-in-place piles

Flynn and McCabe (2016) describe instrumented pile load tests performed on 3 driven cast-in-place piles installed at a site near Coventry, in the United Kingdom. The piles that were formed by driving a 0.32 m diameter hollow steel tube with a sacrificial circular steel plate at the base were instrumented with strain gauges at four levels. When they reached their final penetration depths of between 5.5 m and 7 m bgl. the steel tube was filled with concrete and the steel casing was withdrawn. The CPT profile at the site is shown in Figure 18. The ground conditions comprise made ground and stiff sandy clay to approximately 1.8 m bgl. underlain by medium dense to dense sand to a depth of ~ 6.5 m. Below this depth the sand becomes increasingly gravelly.

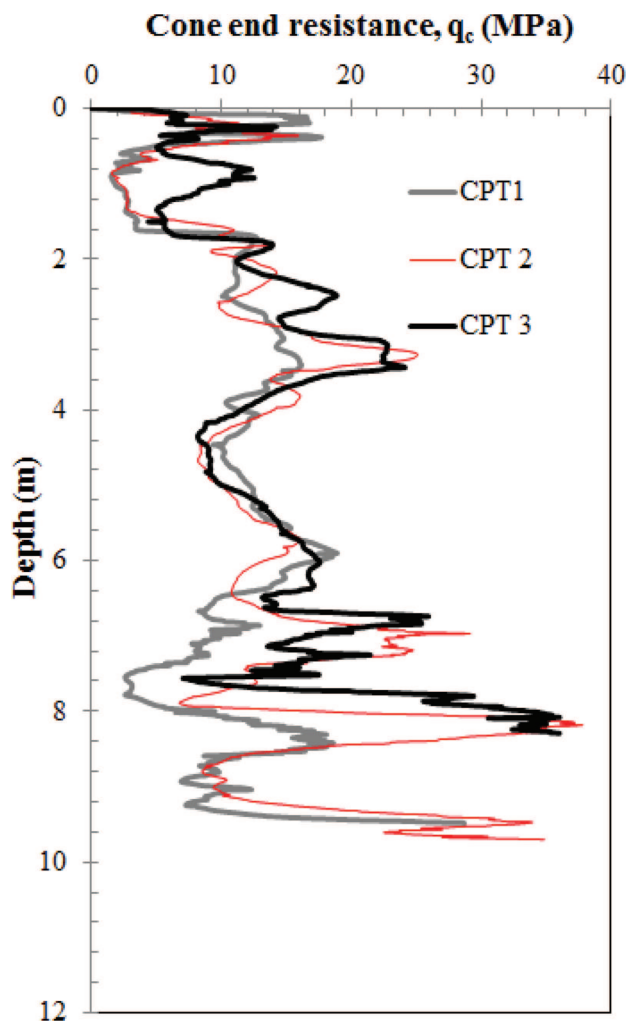


Figure 18. CPT profiles at Coventry.

The piles were load tested in compression between 19 and 23 days after installation.

The mobilisation of average shaft resistance ( $\tau_{av}$  = total shaft resistance/shaft area) during the load tests is shown in Figure 19. The three piles exhibited similar initial stiffness response with peak  $\alpha_s$  values in the range 0.094 to 0.14.

A noticeable feature of the response is that relatively large normalised displacement was required to mobilise the peak capacity (between 3% and 13% of the pile diameter) and in the case of Piles 1 and 3, the resistance was still increasing up to the end of the load test.

#### 4.3 Continuous Flight Auger (CFA) Piles

Gavin et al. (2009) presents the results of instrumented load tests performed to study the development of shaft resistance on Continuous Flight Auger piles installed in sand. Ground conditions at the test site consist of approximately 2 m of mixed (sand, silt and clay) deposits overlying a deep deposit of sand. The CPT end resistance,  $q_c$

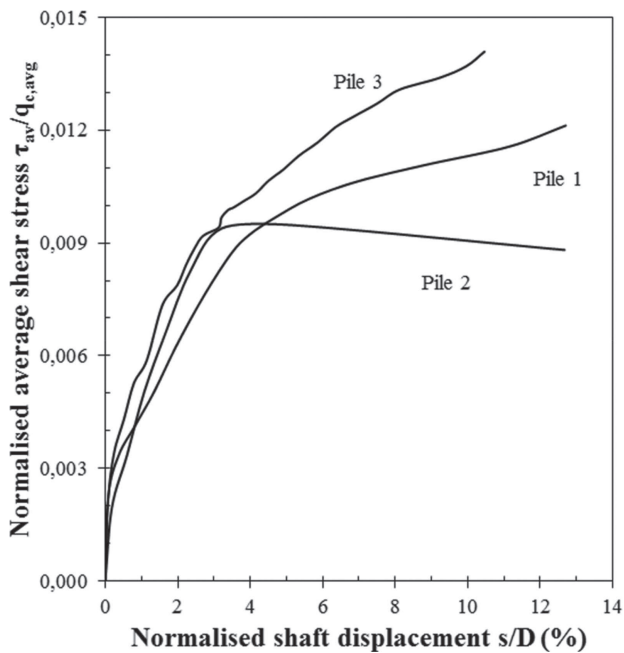


Figure 19. Normalised shaft resistance mobilised by driven cast-in-place piles (after Flynn and McCabe 2016).

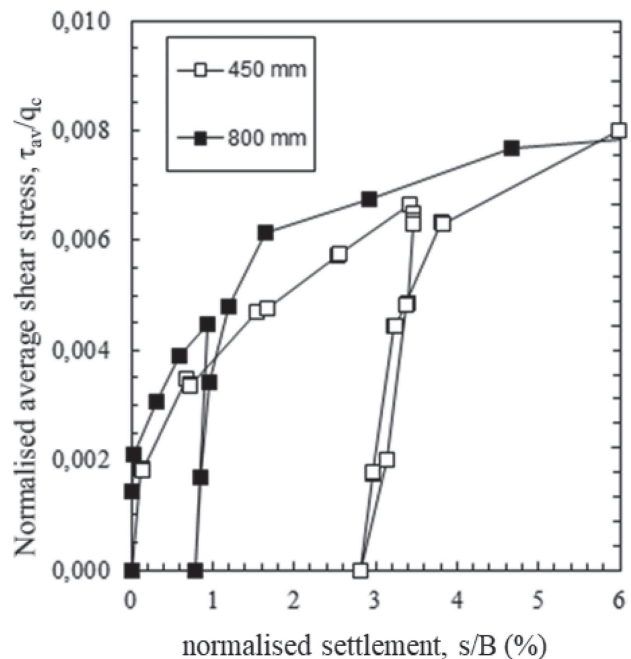


Figure 21. Normalised shaft resistance mobilised by Continuous Flight Auger Piles (after Gavin et al. 2013).

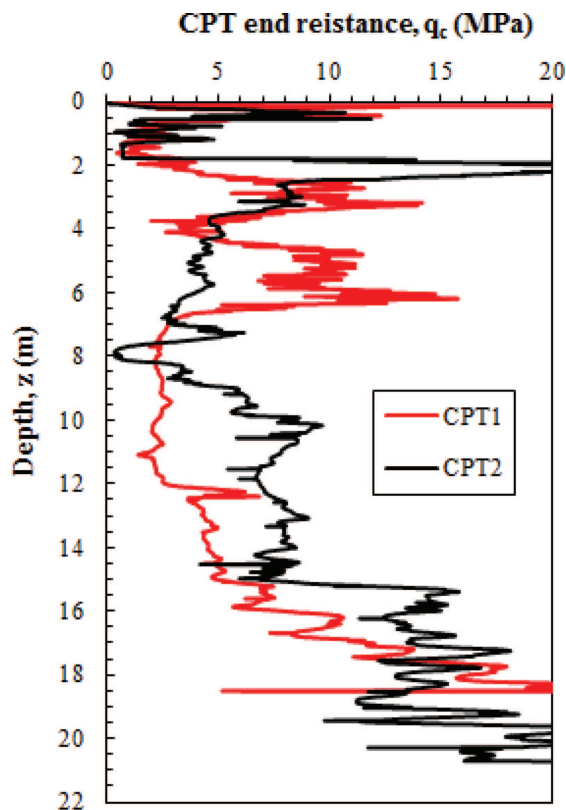


Figure 20. CPT profiles for CFA pile tests at Killarney (after Gavin et al. 2009).

measured in the vicinity of the test piles are shown in Figure 20.

Two instrumented test piles were installed at the site. The first pile was installed using an 800 mm

diameter auger to 14 m bgl., and the second using a 450 mm auger to 15 m bgl. The piles were instrumented with strain gauges at four levels in order to separate base and shaft resistance components and to determine the distribution of shaft resistance along the pile.

Static load tests in compression were performed and the average shaft resistance ( $\tau_{av}$ ) mobilised during the static load tests of between 35 and 36 kPa was almost identical on both the 450 mm and 800 mm diameter piles suggesting that scale effects were insignificant. The resulting  $\alpha_s$  ( $= \tau_{av}/q_{cav}$ ) value of 0.008, shown in Figure 21 are similar to those used for the design of displacement piles in sand. The relatively large displacement required to mobilise the peak resistance is again a feature of the pile tests.

#### 4.4 Screw injection piles

The average shaft resistance mobilised by the screw injection piles installed in dense sand at a site in Terhausen, Netherlands, described in Section 2.4 is shown in Figure 22a. The initial stiffness response of the piles which had shaft diameters of 0.46 m were remarkably similar. The load test on Pile 1 was stopped before it reached peak resistance as the pile mobilised a much higher base resistance than the other test piles, see Figure 9.

Notwithstanding this, it would seem that the ultimate shaft resistance of the piles was in the range 110 kPa to 130 kPa. However, displacements in excess of 10% of the pile shaft diam-

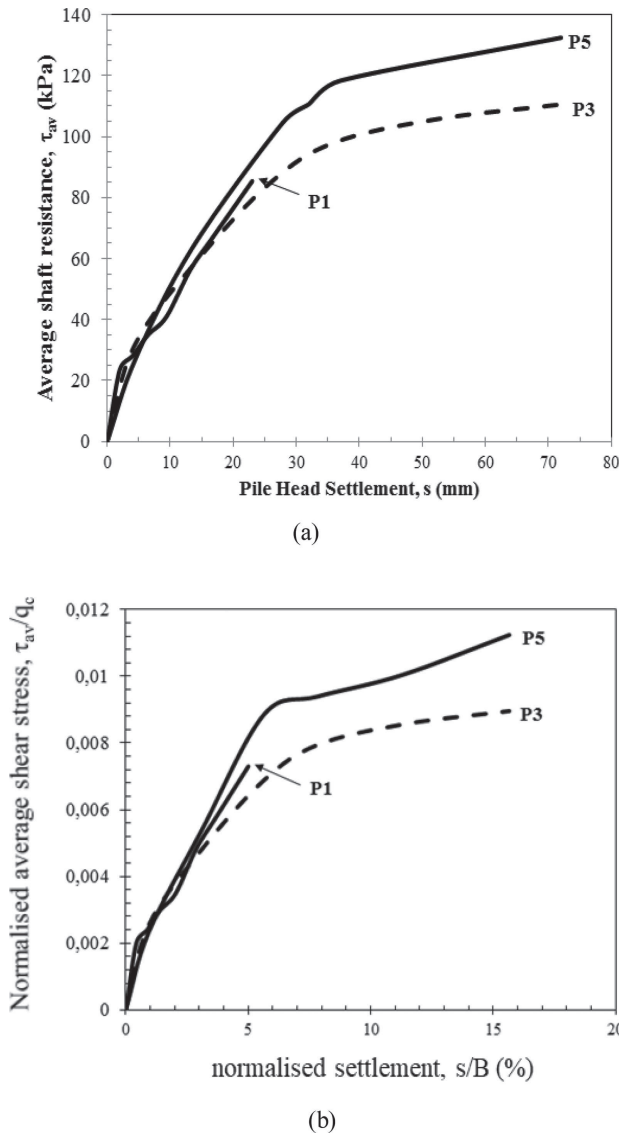


Figure 22. Shaft resistance (a) and normalised resistance mobilised (b) mobilised by Screw Injection Piles at Terhausen.

eter were required to mobilise this resistance. The back-figured  $\alpha_s$  values shown in Figure 22b show that the Dutch standard NEN-EN 9997-1 recommended  $\alpha_s$  value of 0.09 would provide a reasonable estimate of the fully mobilised shaft resistance for these piles.

#### 4.5 Distribution of normalised shaft resistance on piles in sand

Whilst the  $\alpha_s$  values mobilised by the test piles described above conformed broadly with the constant values proposed in many codes and seemed to depend on pile type, with higher values generally pertaining to driven piles, there remains some variation in  $\alpha_s$  values proposed by different codes. Some insight into possible reasons for this can be

determined by the instrumented load tests. The strain gauges on the piles allowed the local shear stress profiles to be determined.

The normalised local shear stress profile along the driven cast-in-place (DCIP) pile (Flynn and McCabe 2016) is compared in Figure 23 to the profile predicted using the UWA-05 approach with

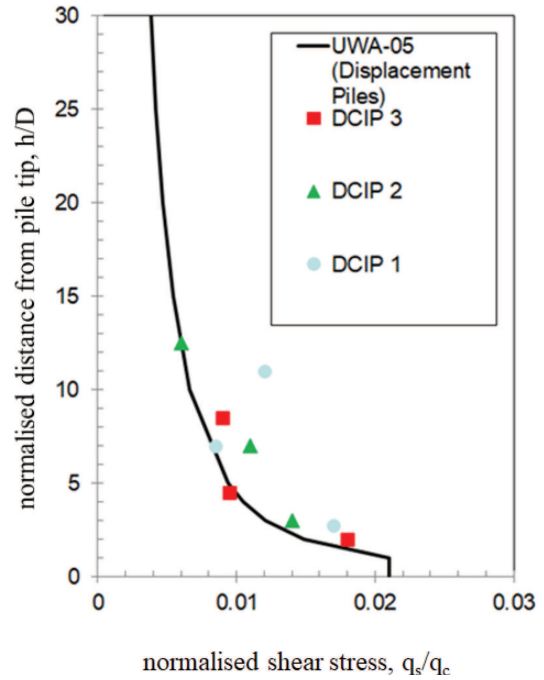


Figure 23. Comparison of distribution of shear stress on driven cast-in-place pile with prediction of UWA-05 method.

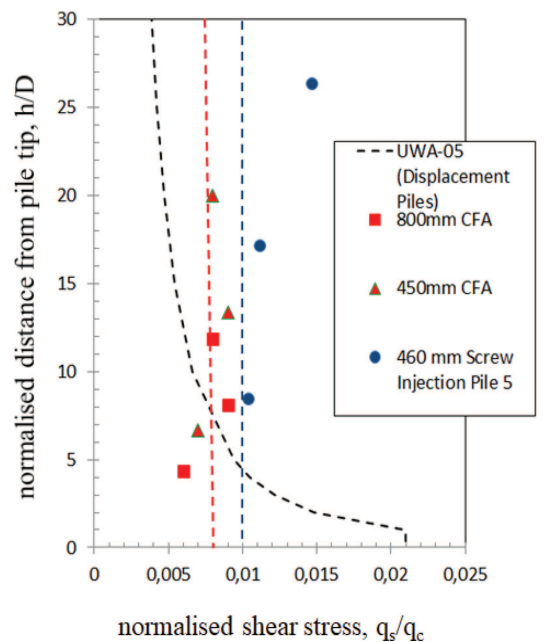


Figure 24. Comparison of distribution of normalised shear stress on CFA and Screw-Injection piles with prediction of UWA-05 method.

$\delta_f = \phi'_{cv}$ ). It is clear that a method which includes for the effects of friction fatigue provides a very good match to the measured response and suggests that for piles where friction fatigue occurs during installation that  $\alpha_s$  should vary with pile geometry.

The normalised local shear stress values for the CFA piles and a typical screw injection pile are shown in Figure 24. For these piles that do not experience friction fatigue during installation, a constant  $\alpha_s$  value is suitable to describe the shaft resistance. The value is lower than that applying to driven piles over a distance of 5D from the pile tip. Above this level the piles developed larger  $\alpha_s$  values. Thus suggesting that non-displacement piles with deep embedment lengths could mobilise higher average  $\alpha_s$  values than piles driven in the same deposit.

## 5 CONCLUSIONS

Whilst conventional earth pressure approaches remain in common use for the design of shallow and deep foundations in sand, researchers have highlighted a number of challenges related to the applicability of these methods. In all cases, obtaining high quality samples and determining operation frictional angles over the range of stress and strain relevant for design remains an issue. For over-consolidated deposits, accurately determining the earth pressure coefficient is essential and non-trivial. For shallow foundations, issues related to determining bearing capacity factors to address depth and width effects particularly for soil deposits where the strength does not increase linearly with depth have been highlighted. For pile design, the impact of installation effects; for example friction fatigue which causes complex changes to in-situ horizontal stress regime around a pile and plugging effects that results in large changes to the strength and stiffness of the sand below the pile base have been quantified in recent field tests.

Given that the CPT provides a continues indirect measurement of the strength and stiffness properties over the complete range of sand state, correlations between the cone end resistance  $q_c$  and foundation behaviour are in common use and have been embedded in the design codes in many countries. However, many of these codes give conflicting guidance on design. In this paper, results from lab and field experiments on instrumented piles and finite element analyses are used to explain the physical basis for the regional variations in CPT based approaches.

For the design of shallow foundations a number of workers have proposed  $\alpha$  values that vary with sand state and footing dimensions. Field tests performed by Briaud (2007), Gavin et al. (2009),

Mayne et al. (2012) and others suggest that unique site specific normalised resistance curves can be derived which are independent of footing geometry. A database of field tests collated in this paper suggests that at large normalised settlement levels ( $s/B = 5-10\%$ ), these normalised resistance curves converge and are independent of sand state. However, the initial stiffness response of the foundation is strongly dependent on the stress history and knowledge of both the strength ( $q_c$ ) and small-strain stiffness parameters are crucial for design.

Considering the base resistance mobilised by piles in sand, the CPT  $q_c$  value appears to be an ideal design tool. Whilst at very large pile displacements the base resistance,  $q_b$  tends to the  $q_c$  value, see White and Bolton (2001) and Randolph (2005). The exact displacement required depends on the pile installation method and in some cases could be several pile diameters. At displacement levels typically considered in practice as representing failure, e.g.  $s/B = 10\%$ , the stiffness response depends on the pile installation method. For low displacement pile types  $\alpha_p (= q_{b0.1}/q_c)$  in the range of 0.15–0.2 are typically adopted in practice. For displacement piles,  $q_b/q_c$  of 0.6 is recommended for closed-ended driven steel and concrete piles. A particular feature of field tests on shallow foundations and bored pile (i.e. where pre-stressing during installation did not occur) was that significant creep occurred during maintained load tests at high stress levels. Numerical analyses of these piles using soil models that do consider creep, showed  $\alpha_p$  values around 50% higher than those measured in maintained load tests. In quick load tests where creep was minimised,  $\alpha_p$  values similar to those in the finite element model were mobilised. Based on these results, if effects such as loading rate, the definition of failure and residual loads are considered then higher  $\alpha_p$  values can be adopted in design codes.

When considering shaft resistance, a number of codes suggest constant  $\alpha_s (= \tau_{av}/q_{cav})$  values, with recommended values for displacement piles typically being higher than non-displacement piles. The case histories presented herein strongly suggest that  $\alpha_s$  values for driven piles should incorporate a friction fatigue parameter. The absence of friction fatigue effects for non-displacement piles could result in some slender driven piles developing lower  $\alpha_s$  than a non-displacement pile installed in the same soil deposit.

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