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A review of CPT based axial pile design in the Netherlands

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Abstract

Because of the deltaic nature of the Netherlands, deep soft soil deposits are widespread. Due to the population density exploitation of underground space is vital for commercial developments and transport networks. Piles are used as primary support elements in deep excavations, cut and cover tunnels, quay walls, flood defences and to provide uplift resistance to the base of tunnels and basements. This paper examines empirical correlations linking the Cone Penetration Test (CPT) end resistance q_c and the resistance of deep foundations in sand. It is found that correlations between q_c and pile end resistance are independent of pile diameter. However, the impact of installation method, residual load, plugging and sand creep should be considered. 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 and pile plugging during installation are recommended.

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Keywords: Piles; Sand; CPT based methods; Design codes

1 Introduction

Whilst conventional bearing capacity and earth pressure approaches are widely used to design deep foundation in sand, many design codes (e.g. [API, 2007](#); [DNV, 2007](#)) are moving towards Cone Penetration Test (CPT) based design methods. This is a result of 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 piles (e.g. [Frank, 2017](#)). However, these values are rarely consistent across borders, 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 behaviour.

Because of the deltaic nature of the Netherlands, deep soft soil deposits are widespread. Consequently, in large cities such as Amsterdam and Rotterdam, more than 99% of structures are supported on pile foundations. Due to the population density exploitation of underground space is vital for commercial developments and road and rail tunnels. Piles are used as primary support elements in deep excavations, cut and cover tunnels and quay walls and to provide uplift resistance to the base of tunnels and basements. Changes to the pile design standard [NEN \(2016\)](#) in the Netherlands have resulted in the installation of larger piles, and increases in both construction time and associated installation risks. In addition, environmental revisions

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of the new code require reductions in the use of non-renewable resources, noise, and vibration. The combined effect is an estimated increase in construction costs of 10%–20% for all deep foundations projects in the Netherlands. The change in design standards arose for a number of reasons and is at least in part due to inertia in moving from historical practice and to a gap in the knowledge surrounding some key aspects of deep foundation behavior.

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

2 Dutch pile design practice

In the current Dutch code, a CPT based design method links the shaft and base resistance components directly to the cone end resistance, q_c measured during the CPT test using constant reduction factors α_s and α_p for the unit shaft, τ_f and base, q_b resistance respectively:

$$\tau_f = \alpha_s q_c, \quad (1)$$

$$q_b = \alpha_p q_c. \quad (2)$$

A range of constant α_s and α_p values for common pile types are given in Table 1. However, recent research suggests that the stress condition around a pile is affected by issues such as cyclic degradation of shear stress (Gavin & O'Kelly, 2007; Tsuha et al., 2012; White and Lehane, 2004), residual stresses established during pile installation (Altaee, Fellenius, & Evgin, 1992; Paik, Salgado, Lee, & Kim, 2003), soil ageing (Axelsson, 2000; Bowman & Soga, 2003; Mitchell, 2008; Schmertmann, 1991), soil plugging (Brucy, Meunier, & Nauroy, 1991; Paik et al., 2003) and the effect of installation method (Basu, Loukidis, Prezzi, & Salgado, 2011; Igoe, Gavin, & O'Kelly, 2013; Yetginer, Bolton, & White, 2006) amongst others and these should be reflected directly in reduction factors. In addition the derivation of reduction factors (α_p and α_s) is affected by local practice, e.g. the method used to determine the average or design q_c value for base resistance and the adoption of limiting values. In the Netherlands when calculating the shaft resistance of piles, the cone resistance is limited to between 12 MPa and 15 MPa, depending on the thickness of the bearing layers. The lower value is used when the

layer considered is less than 1 m thick. A comparison of the measured and design profiles of CPT q_c used to determine shaft resistance at a typical site in the Netherlands is shown in Fig. 1 where it is clear that the limiting value has a large impact in typical sand profiles. For the calculation of base resistance the q_c value is evaluated using the Koppejan technique wherein q_c is evaluated over a zone of 0.7 to 4D below the pile tip and 8D above the pile tip, where D is the pile diameter. The derived unit base resistance ($q_b = \alpha_p q_c$) is limited to a maximum value of 15 MPa.

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 performed on driven pre-cast concrete piles, See Fig. 2, it was found that the pile base capacity derived using the pre-2017 α_p value of 1, over-predicted the base capacity of piles as the penetration into the bearing layer increased. As a result a 30% reduction to the pile base reduction factor for all pile types was introduced from January 1st 2017. Van Tol, Stoevelaar, Bezuiken, Jansen, and Hannik (2015) noted that a number of hidden safety factors might pertain to pile capacity in the Netherlands and affect the interpretation of α factors, including pile ageing, residual stress and limiting values of resistance. The impact of these uncertainties on pile design in practice will be considered in this paper.

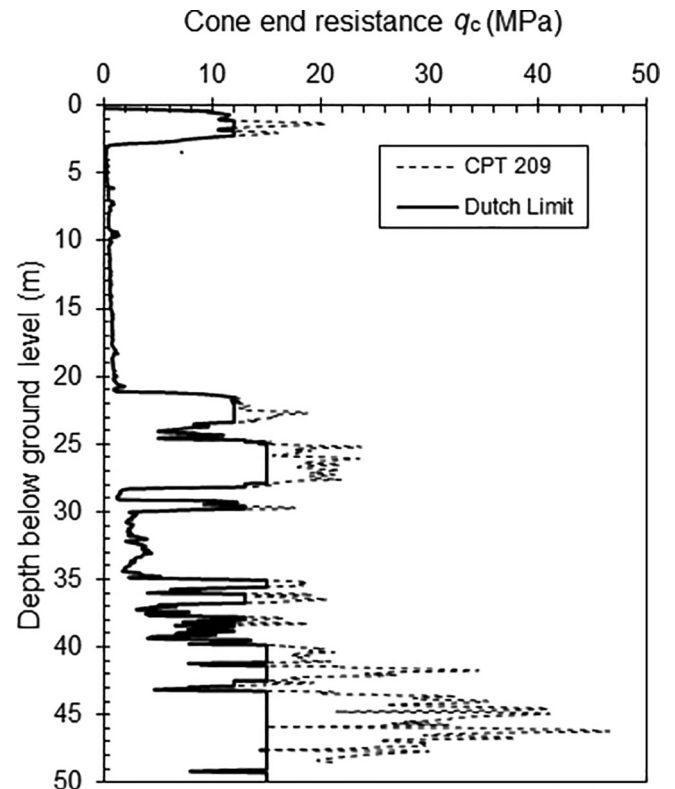


Fig. 1. Example of effect of limiting q_c values on a typical CPT profile.

Table 1
 α_p and α_s factors from the 2017 Dutch Standard.

Pile type	α_p	α_s	α_t
Pre-cast concrete closed-end	0.7	0.01	0.007
Steel tube closed-end	0.7	0.01	0.007
Steel tube open-end	0.7	0.006	0.004
Screw injection pile	0.63	0.009	0.009
Continuous flight auger	0.56	0.006	0.0045
Bored pile	0.35	0.006	0.0045

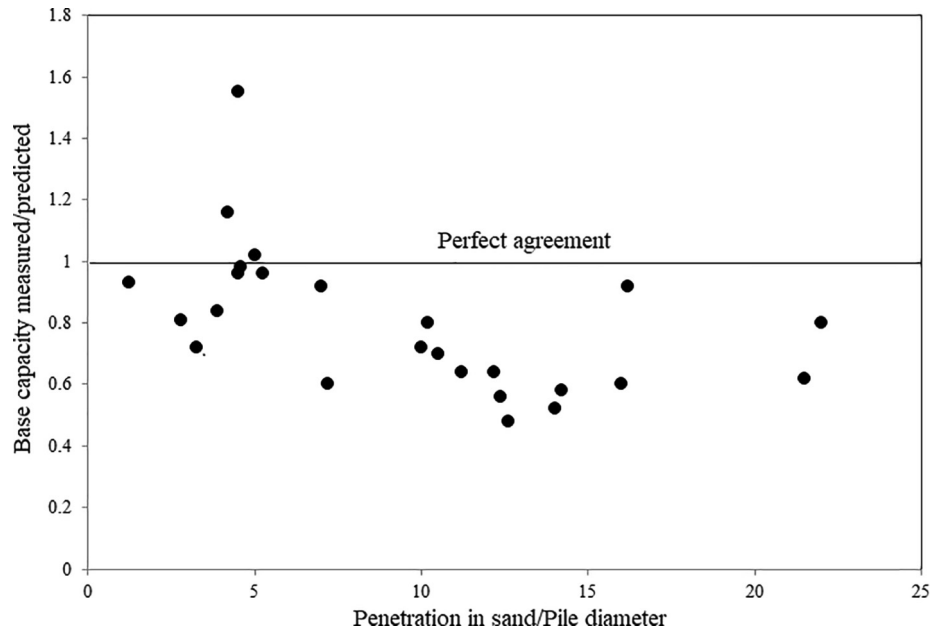


Fig. 2. Comparison of measured and calculated pile base capacity for Dutch method with $\alpha_p = 1.0$ (after Stoevelaar et al. 2011).

3 Linking end bearing resistance and CPT q_c

3.1 Background

Because of the similarities between the penetration mechanisms and the geometry of piles and the CPT, a number of empirical correlations between the CPT end resistance q_c and q_b have been proposed. Recent researches in the field for various pile types are summarized here.

3.2 Closed-end driven piles

CPT based design methods generally estimate the base resistance at relatively large pile base settlements typically at 10% of the pile diameter using an empirical reduction factor α_p (e.g. Bustamante & Gianselli, 1982). Based on a database study Jardine, Chow, Overy, and Standing (2005) developed the IC-05 design method that suggests α_p reducing with pile diameter, D :

$$q_{b0.1}/q_{cav} = [1 - 0.5 \log(D/D_{CPT})], \quad (3)$$

where $q_{b0.1}$ is the base resistance mobilized at a pile tip displacement of 10%, $D_{CPT} = 36$ mm and a minimum α_p value of 0.3 is adopted for large diameter piles, and q_{cav} is determined by averaging q_c in the zone $\pm 1.5D$ around the pile tip.

Lehane, Scheider, and Xu (2005) found that a α_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 once the Dutch method for averaging q_c was adopted in the approach known as the University of Western Australia, UWA-05 method.

$$q_{b0.1}/q_{cav} = 0.6. \quad (4)$$

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 α_p factor can be adopted which is independent of pile diameter and it tended towards $q_b = q_c$ at large pile base displacements (typically multiples of the pile diameter).

Gavin, Reale, and van der Wal (2019) reported load tests performed on 450 mm square, pre-cast piles driven 35 m into a dense sand layer at the Port of Rotterdam in the Netherlands. The CPT profile at the test site in Fig. 3 shows a complex sequence with dense sand fill, over soft clay extending to about 20 m below ground level, bgl. Beneath the soft clay layer two dense sand layers are inter-bedded with a stiff clay. Traditionally structures are founded in the lower sand layer. It is noteworthy that most CPT tests were limited to $q_c = 30$ MPa and the significant variability particularly in CPT 501 below 35 m is related to the test procedure rather than inherent variability of the soil. The pre-cast piles were driven about 1 m into the lower sand layer. Three piles instrumented using optical fiber strain sensors were seen to develop significant residual loads after installation. The base pressure mobilized during static load tests is shown in Fig. 4. The initial stiffness response in all tests was very similar. Whilst there is a dearth of data for pile base displacement above 20 mm, the base pressure, mobilized at 50 mm is assumed as $q_{b0.1} = 14.4$ MPa, with a residual stress of 4.1 MPa included.

Considering the back-figured α_p values in Table 2, ignoring residual loads gives $\alpha_p = 1.01$ when the design CPT is calculated according to the Dutch method and $\alpha_p = 0.57$ when averaging over $\pm 1.5D$. The real base resistance mobilized includes the residual load and therefore the α_p value of

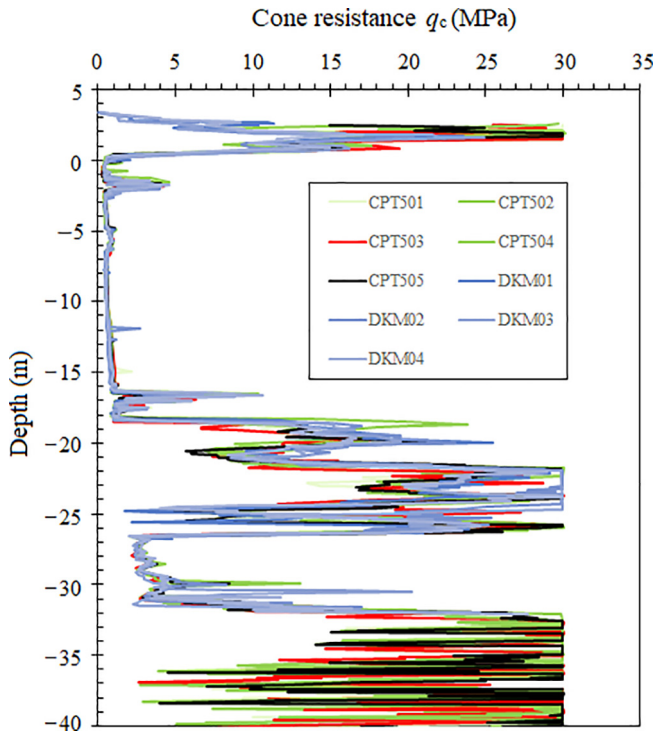


Fig. 3. CPT profile at the Port of Rotterdam.

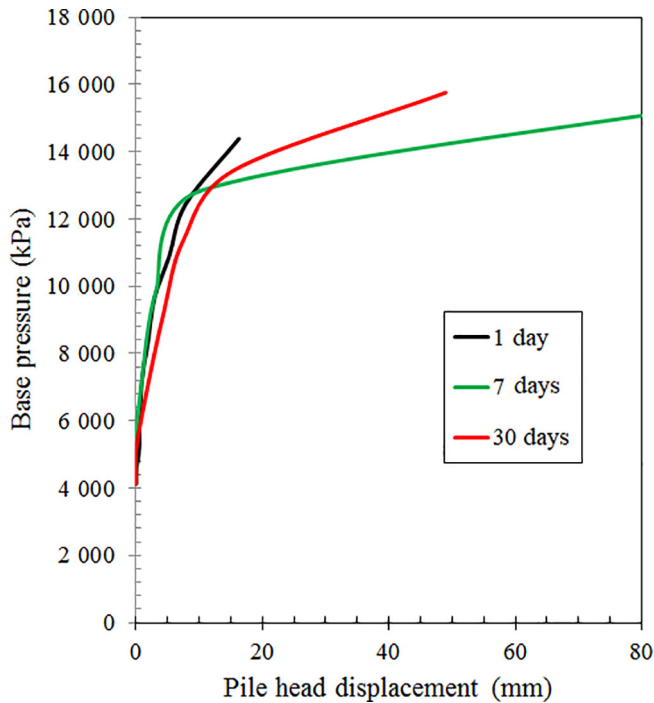


Fig. 4. Base pressure mobilized on three 450 mm square driven pre-cast concrete piles at the Port of Rotterdam.

1.41 determined using the Dutch CPT averaging method is double that value given in the 2017 Dutch code value and more than double the value recommended in the UWA method, Eq. (4). The former value is in keeping with the values reported by Stovelaar et al. (2011) in Fig. 2 for piles

with shallow embedment depth in the founding layer. In this case it appears that the primary reason for this large conservatism is caused by the CPT averaging technique adopted with the Dutch method which extends $8D$ above the pile base resulting in a significant influence of the stiff clay layer in the design q_c value.

3.3 Open-end piles

Steel Open-end tubular piles are used extensively in the Netherlands. For pile base resistance a constant α_p value of 0.7 is adopted in design. Model tests performed in the laboratory (Lehane & Gavin, 2001), field (Igoe, Gavin, & O'Kelly, 2011) and full-scale pile tests (Foye, Abou-Jaoude, Prezzi, & Salgado, 2009) indicate that the base resistance of open-ended piles comprises two components, plug stress developed by the soil plug inside the pile and annular stress developed beneath the pile toe. Tests on twin-walled instrumented piles show once these two components are normalized by an appropriate average CPT q_c values, that α_p is independent of pile diameter and sand state. However, the plug stress component is directly related to the degree of plugging experienced during pile penetration. The degree of plugging is affected by the pile diameter, sand density and installation technique adopted. The internal soil plug development during installation can be described by the Incremental Filling Ratio (IFR) whilst the Effective Area Ratio ($A_{r,eff}$) has proven to be a useful term for considering the impact of plugging on pile base resistance:

$$IFR = \Delta L_{plug} / \Delta L, \quad (5a)$$

$$A_{r,eff} = 1 - IFR(D_1/D)^2, \quad (5b)$$

where ΔL_{plug} is the change in the plug length for a given penetration increment ΔL and D_1 is the internal pile diameter. Thus $IFR = 1$ when a pile is fully coring, and $IFR = 0$ when the pile is fully plugged. $A_{r,eff}$ is the ratio of volume of soil displaced to gross pile volume and is equal to unity for a closed-end pile or fully plugged pile ($IFR = 0$) reducing to typical values of 0.1–0.2 for fully coring piles ($IFR = 1$). In base resistance calculations that explicitly consider plugging the final IFR value or final filling ratio, FFR is often used, See Gavin and Lehane (2005).

Because open-end piles are seen to develop lower end bearing resistance than closed-end piles in similar sand deposits the IC-05 method assumes that $q_{b0.1}$ is 50% lower than that for a closed-end pile.

$$q_{b0.1}/q_{cav} = [0.5 - 0.25 \times \log(D/D_{CPT})], \quad (6)$$

whilst the UWA-05 accounts for plugging explicitly;

$$q_{b0.1}/q_c = 0.15 + 0.45A_{r,eff}. \quad (7)$$

Paik et al. (2003) reported a field test performed on a twin-walled, 356 mm diameter instrumented open-end steel pipe pile driven at Pigeon River, Lagrange County, Indiana, USA. The site consisted of 3 m of loose sand

Table 2

Comparison of α_p factors with CPT averaging technique at Port of Rotterdam ($q_{b0.1} = 13.2$ MPa including residual load or 9.4 MPa ignoring residual load).

Design CPT q_c (MPa)	α_p ignoring residual load	α_p with residual load
Dutch = 10.18 MPa	1.01	1.41
$\pm 1.5D = 17.75$ MPa	0.57	0.81

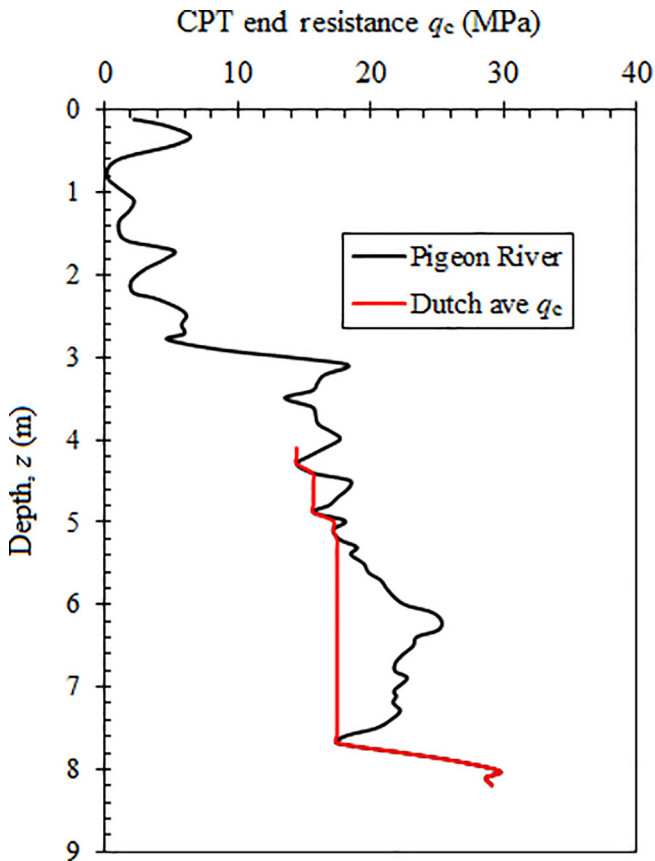


Fig. 5. CPT profile at Pigeon River Site (after Paik et al., 2003).

overlying dense sand with the CPT profile shown in Fig. 5. The soil plug length was recorded throughout driving. The pile remained partly coring during installation with an FFR value of 77.5%. The instrumentation on the pile included strain gauges that allowed the residual load developed during pile installation to be determined. A static load test revealed the pile developed $q_{b0.1}$ of 7.2 MPa before residual loads are accounted for, with the corrected end resistance being 8.9 MPa. Comparison of the predictive performance of the CPT methods in Table 3 reveals the following:

Table 3

Comparison of measured and predicted based resistance $^1 q_{b0.1} = 8.9$ MPa with residual load, 7.2 MPa and 2 ignoring residual load.

Method	Measured/Predicted ¹	Measured/Predicted ²
Dutch	0.71	0.57
IC-05	1.58	1.28
UWA-5	1.35	1.09

- (1) The Dutch method for averaging the q_c value at the base gives a value of 18 MPa and the application of α_p of 0.7 over-estimates the end bearing resistance significantly.
- (2) The IC-05 method used the average the q_c design CPT q_c value in the region $\pm 1.5D$ ($q_c = 22.6$ MPa), i.e. much higher than the Dutch approach. Despite this the method significantly under-predicts the base resistance. This is because the method assumes no plugging with Eq. (6) resulting in an α_p of 0.25.
- (3) The UWA-05 approach which accounts for plugging provided an under-prediction of the true base resistance (an under-prediction of 35%). This under-prediction may be in part because of the adoption of the Dutch q_c value of 18 MPa. Using the q_c value in the region $\pm 1.5D$ with the UWA α_p value from Eq. (7) would reduce the under-prediction to 8%.

3.4 Partial displacement screw injection piles

Funderingstechnieken Verstraeten bv. and BMNED bv. performed three static compression load tests on screw injection piles installed in dense sand at a site in Ternaussen, 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. 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 Fig. 6. The estimated pile capacity ranged from 5750 kN to 6100 kN. The load test on Pile 1 was terminated at 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 Fig. 7, 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 α_p values derived from the Dutch averaging method and assuming q_c averaged over a distance $\pm 1.5D$ above the pile base are shown in Table 4. Adopting the Dutch averaging technique the measured α_p is 1.12, 80% higher than the current NEN value (of 0.63). Using

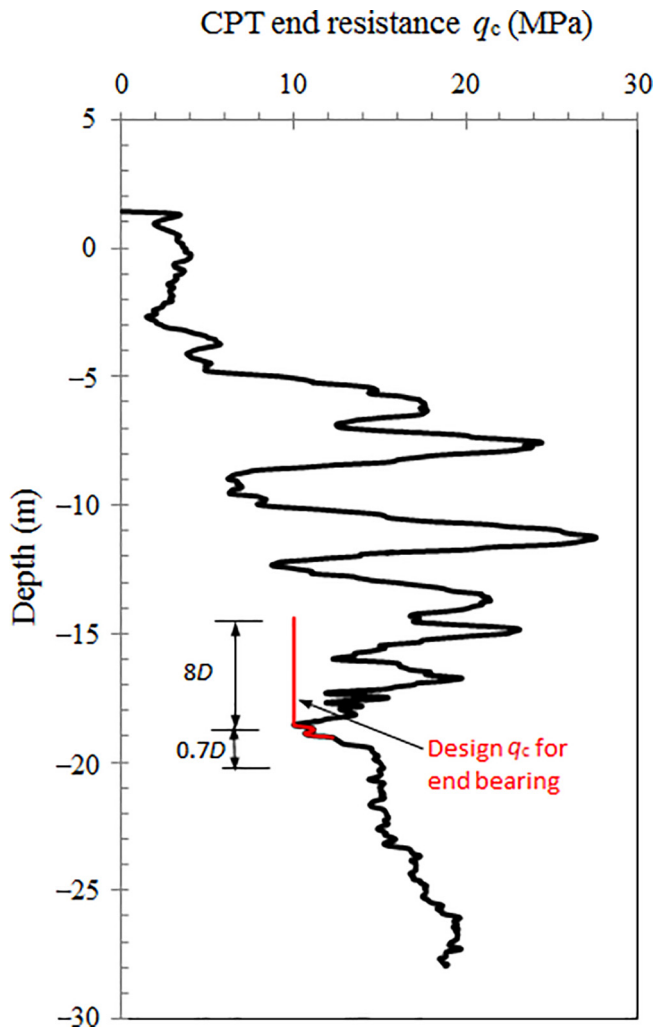


Fig. 6. CPT profile at Ternaussen test site.

the $\pm 1.5D$ method suggests that the ultimate end bearing resistance of these piles is 6% higher than the pre-2017 value of $\alpha_p = 0.9$.

3.5 Low-displacement piles

The Dutch standard recommends α_p value for low displacement piles that vary with construction type. For piles formed using the Continuous Flight Auger (CFA) process, $\alpha_p = 0.56$ whilst for traditional bored piles a lower $\alpha_p = 0.35$ is recommended. For low-displacement piles the Dutch method calculates the base pressure a normalized pile base displacement of 20% of the pile diameter, most other design approaches maintain the 10% displacement criteria adopted for driven piles. The IC-05 and UWA-05 methods were not developed for non-displacement piles, however, Gavin, Cadogan, Casey, and Tolooiyan (2013) showed that α_p values approximate to values applicable to pile with IFR = 100%, $\alpha_p = 0.20$. Eurocode 7, Part 2, suggests that α_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.

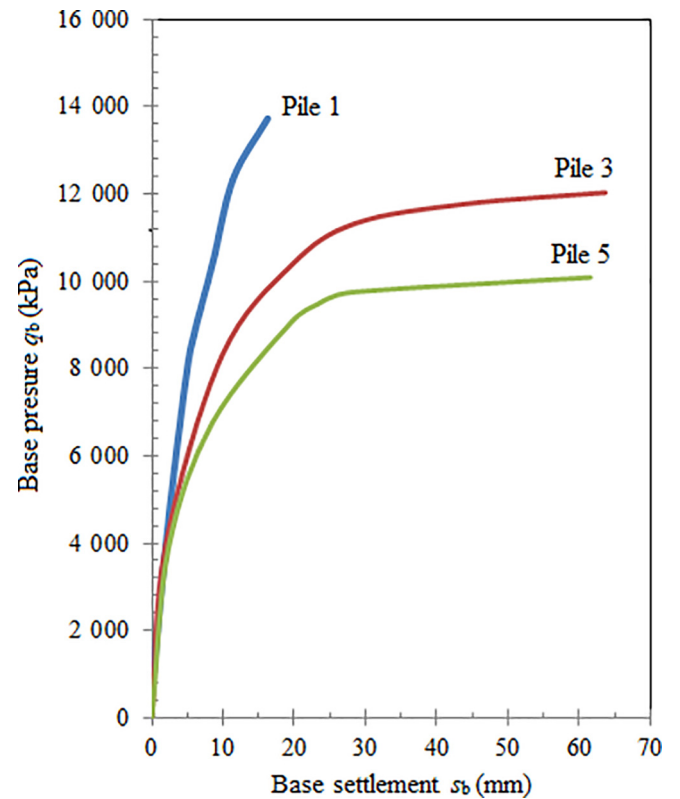


Fig. 7. Base resistance mobilised by grout injection piles at Ternaussen test site.

Table 4

Comparison of α_p factors back-figured from Ternaussen test site using different CPT averaging techniques.

Pile No.	q_c Dutch (MPa)	α_p Dutch (MPa)	$q_c \pm 1.5D$	$\alpha_p \pm 1.5D$
Pile 1	9.75	1.40	12	1.23
Pile 2	10.90	1.10	15	0.80
Pile 3	11.60	0.86	11.8	0.85
Average		1.12		0.96

Given the range of recommendations in the literature 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 D 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/D in the range of 4–37. They were founded in sand where the CPT q_c value ranged between 2 MPa and 40 MPa. In the assessment of mobilised α_p values, the design $q_c \pm 1.5D$ was adopted. The variation of α_p with CPT q_c and pile diameter is plotted in Fig. 8, there was no suggestion that α_p varied in a consistent manner with any of the parameters considered in the assessment, with an average α_p value for the database piles of 0.24.

Tolooiyan and Gavin (2013) performed finite element analysis using PLAXIS to investigate the factors affecting α_p for bored piles in sand. The sand was modelled using the Hardening Soil (HS) model described by Schanz,

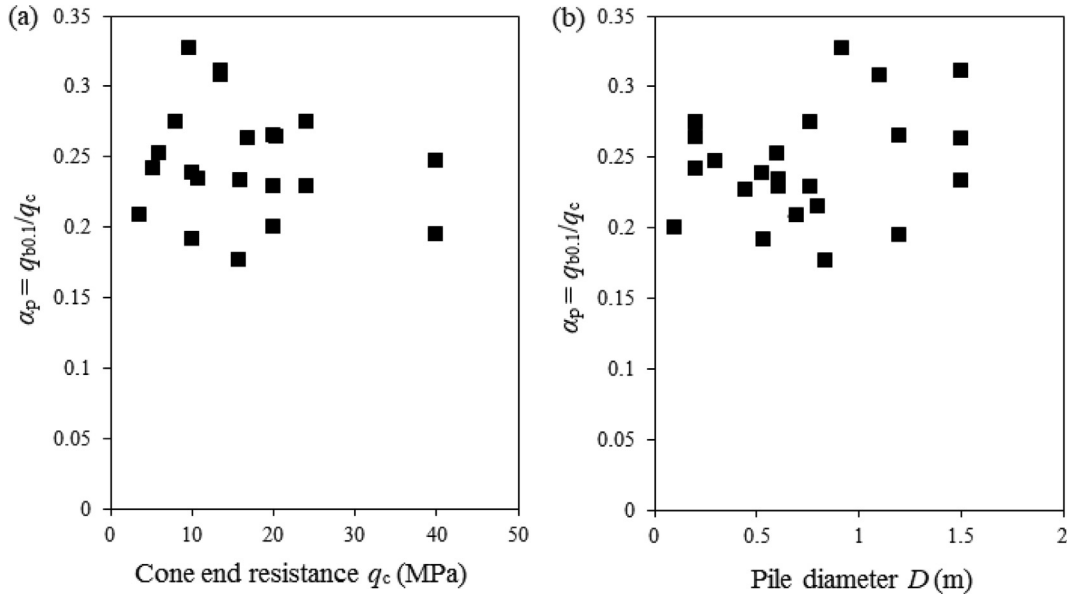


Fig. 8. α_p values backfigured from load test database: (a) effect of strength, and (b) pile diameter.

Vermeer, and Bonnier (1999) with 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 correctly model the CPT q_c profile for Blessington sand. The finite element model was used to investigate the effect of pile diameter D on the bearing pressure mobilised by piles. A series of pile tests on a pile of fixed length of 6 m with a diameter ranging from 0.2 m to 0.8 m were performed, see Fig. 9(a). The end bearing resistance mobilized at a pile base settlement of 10% of the pile diameter (≈ 5500 kPa) of all piles was similar and the piles had not reached their ultimate resistance. However, the settlement required to achieve this resistance increased in proportion to the pile diameter. The normalised pressure-settlement response from these tests is shown in Fig. 9(b). This reveals that the response for the piles was very similar, with $\alpha_p = 0.31$, albeit at a normalised pile base displacement of 10% and noting in this case q_c was determined over a distance of $\pm 1D$. From the shape of the mobilization curves higher α_p value would have been mobilized at larger normalized displacements.

The influence of sand state and CPT averaging technique on α_p was considered by comparing the results for Blessington sand with three other sand deposits. These were Tanta sand from Egypt which has an in situ relative density $D_r = 75\%$, Monterey sand from the United States with $D_r = 65\%$ (Wu et al., 2004), and Hokksund sand from Norway with $D_r = 50\%$. When the bearing pressure mobilised at a pile base settlement of 10% was normalised by the CPT q_c value averaged over $\pm 1D$, α_p values in the range of 0.32–0.33 were determined. There were variations in the rate of mobilization at low normalized settlements with the resistance developing more slowly as the relative density increased. The influence of CPT averaging technique

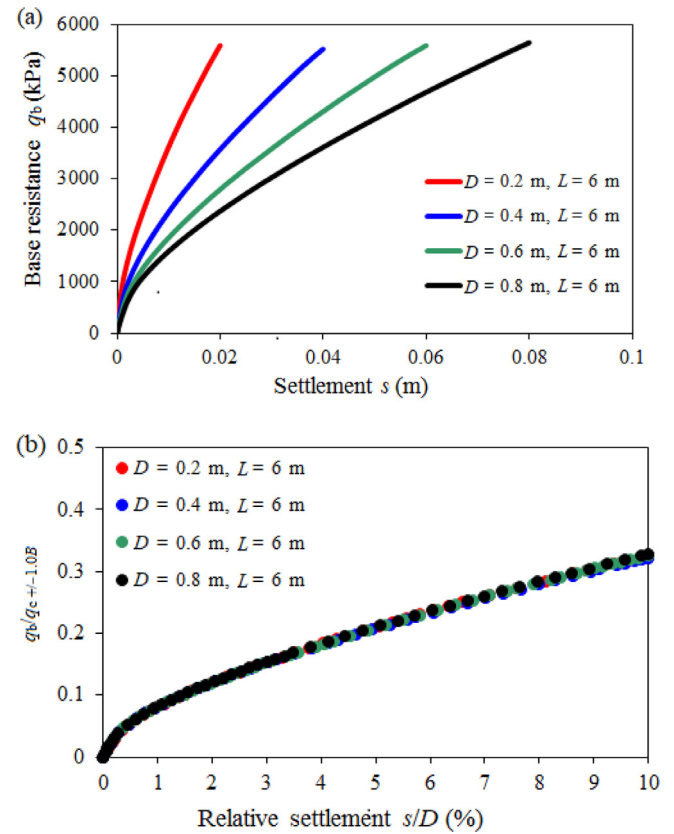


Fig. 9. Effect of pile diameter on the base resistance of bored piles in Blessington sand: (a) base pressure versus settlement, and (b) normalized response.

and pile length on α_p developed in Hokksund Sand is considered in Fig. 10. Consistent trends were found for all sites where α_p was reasonably constant when q_c was averaged over equal distances above and below the pile base, see

Fig. 10(a). When q_c was averaged with a bias for values either above or below the pile base, see Fig. 10(b), higher variability and a more pronounced length bias were observed with the highest α_p values being inferred when using the Dutch average technique. On the basis of these numerical analyses in four sand types, it appeared that an approximate constant α_p factor of 0.3 produced reasonable lower-bound estimates of the end bearing resistance of bored piles in sand. This value is 50% higher than values typically used in practice and 25% higher than the α_p value of 0.24 inferred from the database study, although it is 17% lower than the value given in the Dutch standard.

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 α_p . Gavin et al. (2009, 2013) reported load tests performed on instrumented continuous flight auger, CFA piles installed in medium-dense overlying dense sand in Killarney, South-West Ireland. The soil conditions at the site comprised 2–3 m of made ground over glacial sand deposits to depth. The sand deposit was a medium-dense layer overlying a very dense layer. The depth to the very dense sand layer was quite variable, and other CPT tests per-

formed within a few metres of the test piles showed the very dense sand layer at much shallower depths. Two piles were installed to end bear in a dense sand deposit and load tested, a 450 mm diameter, 15 m long pile and an 800 mm diameter, 14 m long pile. The static load test procedure involved a maintained load test (MLT) followed by a fast-loading, constant rate of penetration (CRP) test. The piles were instrumented with strain gauges to allow the base and shaft resistance to be separated. The results of the MLT portion of the load test are shown in Fig. 11(a) 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 α_p factor approached

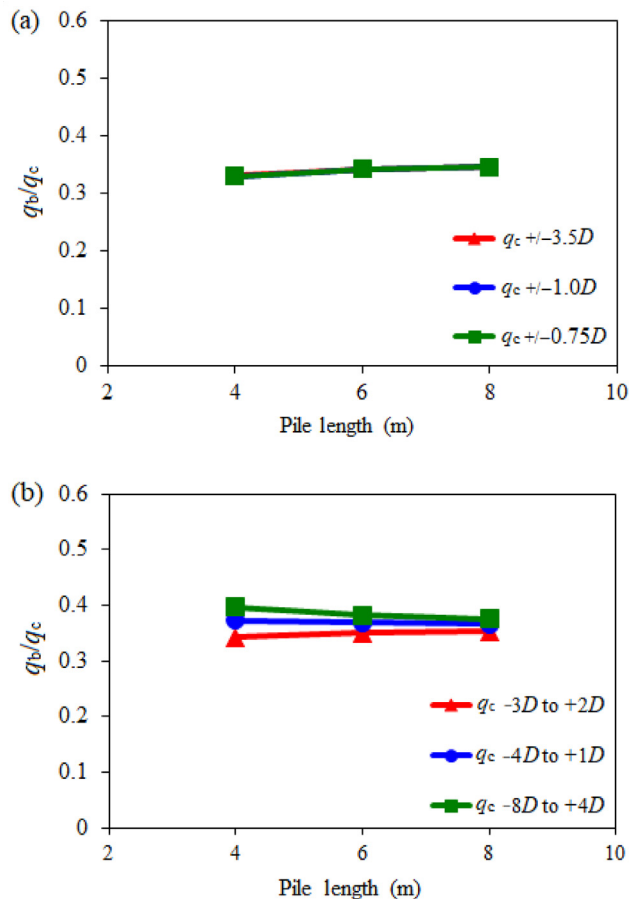


Fig. 10. Effect of CPT averaging technique and pile diameter on α_p values in Hokksund Sand: (a) averaging with equal weight above and below pile tip, and (b) averaging with bias.

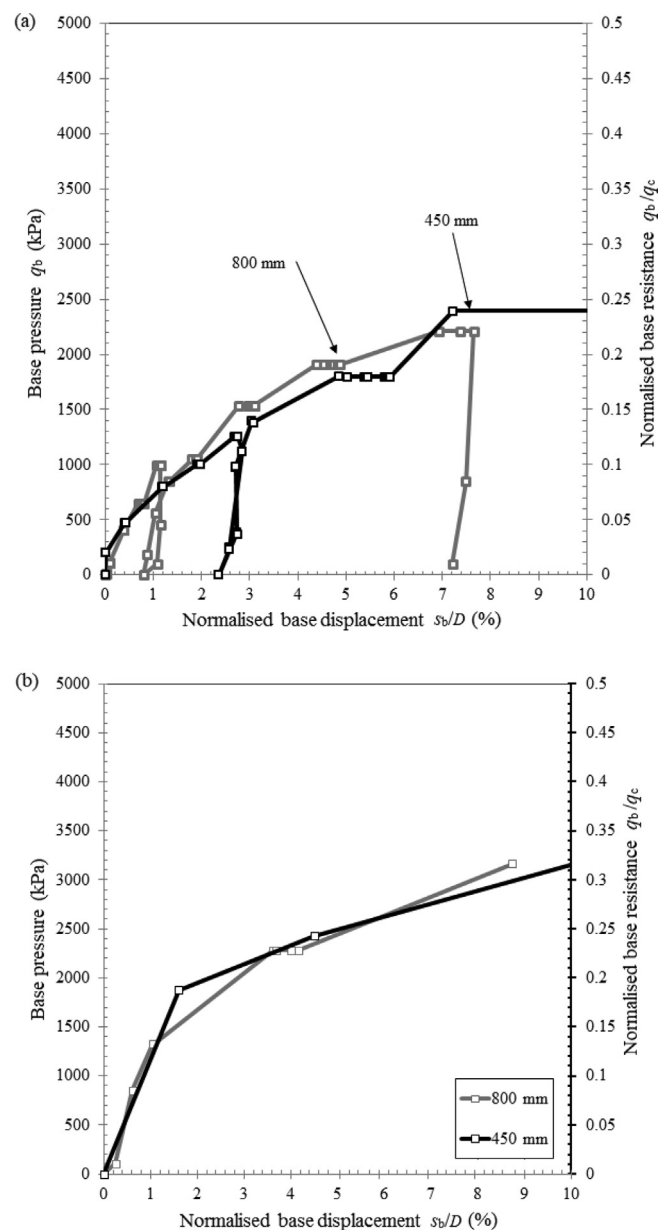


Fig. 11. Effect of loading rate or test method on α_p mobilised by CFA piles in sand: (a) Static load test and (b) CRP test.

0.24. When the piles were reloaded in the CRP test (see Fig. 11(b)), significantly higher base resistance was mobilised. Whilst the loading history would affect the initial pressure-settlement response, the α_p factors mobilised in the fast loading tests (where creep effects were minimised) exceeded 0.31. Thus the CRP load test gave results similar to those seen in the numerical models. 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 Linking shaft resistance and CPT q_c

Whilst many design guidelines, particularly in the offshore sector continue to use traditional effective stress approaches for estimating the shaft and base resistance of piles, due to uncertainties regarding input parameters such as effective friction angle ϕ' , over-consolidation ratio, OCR and interface friction angle δ_f , and the effect of installation method, the use of CPT based design methods is increasing. In the Netherlands as with other design codes the α_s values in Table 1 are the lowest for bored piles and the highest for displacement piles. Limiting values are shaft resistance are included which is in keeping with Belgian practice, summarized by Huybrechts, De Vos, Bottiau, and Maertens (2016). They report that α_s depends on q_c , pile type and roughness with a limiting maximum shaft resistance of 150 kPa is imposed for $q_c > 20$ MPa.

4.1 Closed-end driven piles

Based on field tests using a highly instrumented pile Jardine et al. (2005) and Lehane et al. (2005) show that the local shaft resistance is given by:

$$\tau_f = (\sigma'_{rc} + \Delta\sigma'_{hd}) \tan \delta_f, \quad (8)$$

where: σ'_{rc} 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 σ'_{rc} recorded by an instrumented pile installed at two sites and found that σ'_{rc} values at a given location on the pile were almost directly proportional to the q_c value at that level, the distance from the level to the pile base (h) normalised by the pile radius R or diameter D and the in situ stress level. 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).

$$\sigma'_{rc} = 0.029q_c \left(\frac{h}{R} \right)^{-0.38} \left(\frac{\sigma'_{vo}}{p_{ref}} \right)^{0.13}, \quad (9)$$

where σ'_{vo} is the in situ vertical effective stress and the coefficient p_{ref} is 100 kPa, and a minimum h/R value of 8 is adopted. Lehane et al. (2005) proposed a similar approach known as the UWA method (Lehane et al., 2005), where:

$$\sigma'_{rc} = 0.03q_c \left(\frac{h}{D} \right)^{-0.5}, \quad (10)$$

where a minimum h/D value of 2 is adopted in this study.

Eqs. (9) and (10) suggest that α_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:

$$\Delta\sigma'_{hd} = \frac{4G\delta_h}{D}, \quad (11)$$

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

4.2 Open-ended pipe piles

The IC-05 and UWA-05 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-end displacement piles by Chow (1997), Gavin (1999), Schneider (2007) and others. Given the prevalence of driven open-end pipe piles, particularly in the marine and offshore sectors, 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) noted that whilst the two approaches give very similar predictions for shaft capacity of closed-end piles, differences in how they address the issue of plugging can result in significantly different estimates for the shaft resistance of closed-end piles.

In the IC-05 approach it is assumed that piles remain fully coring during installation (IFR = 100%) and Eq. (9) is modified using a modified pile radius, R^* . This increases the rate of degradation of shaft resistance with distance from the pile base:

$$R^* = \sqrt{D^2 - D_1^2}, \quad (12)$$

where D_1 is the internal diameter of a pipe pile.

In the UWA plugging is included explicitly using the effective area $A_{r,eff}$:

$$\sigma'_{rc} = 0.03q_c \left[\max \left(\frac{h}{D}, 2 \right) \right]^{-0.5} A_{r,eff}^{0.3}. \quad (13)$$

Gavin and Igoe (2019) reported load tests on two instrumented piles, one tested four days after installation and the second after an ageing period of 146 days. The open-end steel piles were driven into dense sand, see Fig. 12(a) for the CPT profiles. The piles had an external diameter, D of 340 mm and a wall thickness, t of 14 mm and were installed to penetrations of between 6.5 m and 7 m below ground level. During installation driving was paused at intervals of 0.25 m to record soil plug length. The Incremental Filling Ratio (IFR) value for both piles is shown in Fig. 12(b). The pile was almost fully coring

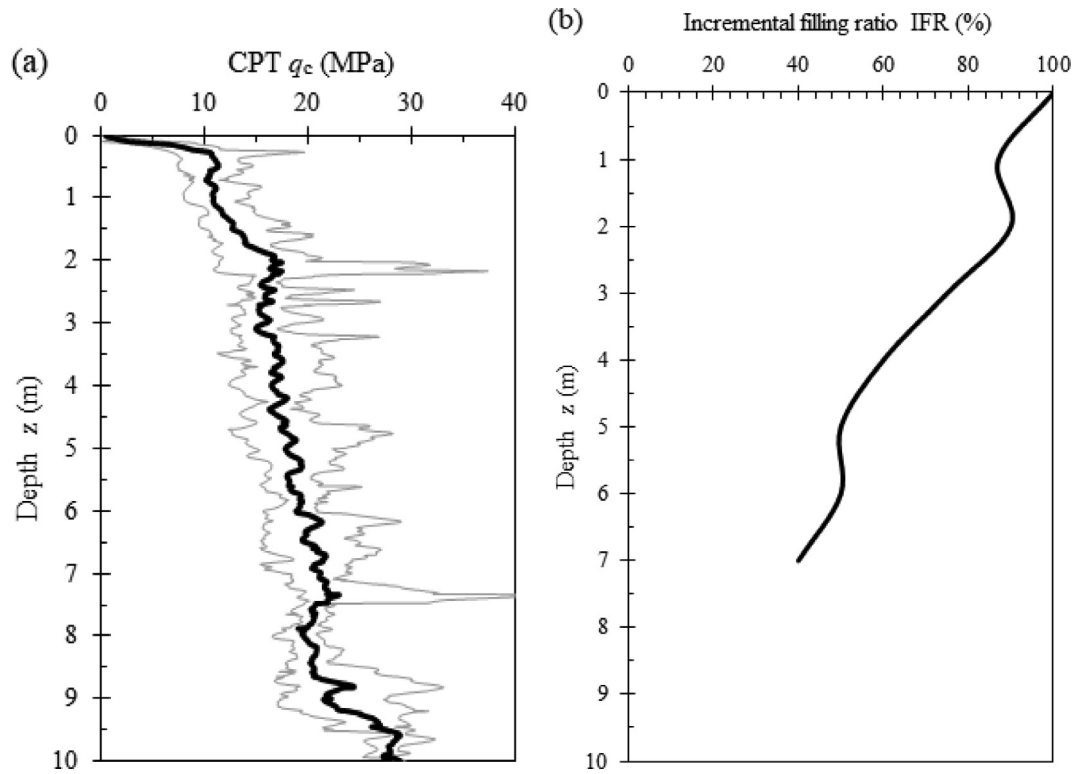


Fig. 12. (a) CPT cone resistance at Blessington showing average, maximum and minimum profiles; (b) Incremental Filling Ratio, IFR profile.

(IFR > 85%) for the first 2 m of penetration. Below this depth IFR reduces with depth with a final IFR value at 7 m of 40%. Gavin and Igoe (2019) noted that the decrease of IFR was seen to cause a strong increase in driving resistance, horizontal stress on the pile and residual load.

The instrumentation included strain gauges at multiple levels to allow the distribution of shear stress to be measured during tension load tests. The load tests revealed that the axial tension capacity of the pile increased significantly with time from 440 kN four days after installation to approximately 1000 kN after 146 days. This corresponds to an increase in local average shear stress τ_{av} from 63 kPa to 140 kPa. The distribution of shear stress along the piles is shown in Fig. 13, which reveals:

- (1) Despite the relatively uniform soil strength over the penetration depth of the pile, see Fig. 12(a) that the load transfer and shear stress mobilized between ground level and 4 m bgl. was relatively low.
- (2) The majority of the resistance developed by the pile was in the region from 4 m to 7 m bgl, with shear stresses being much higher near the pile tip. This is most likely due to a combination of friction fatigue effects, soil plugging and surface effects.
- (3) During the ageing period the shear stress in the region between 3 m and 7 m bgl. increased significantly whilst those closer to ground level did not appear to change.

Estimates of the shear stress profile were made using the Dutch, IC-05 and UWA-05 methods.

- (1) For the 4 day test (Fig. 13(a)), the Dutch method with a constant α_s value provides a reasonable, albeit upper bound value of the measured shear stress over the first 4 m of the pile penetration. Because of the limiting value of q_c of 15 MPa, the maximum shear stress according to the approach is 60 kPa. It is clear that this under-estimates the pile resistance over the lower 3 m near the pile tip.
- (2) The IC-05 and UWA-05 methods provide comparable approximations for the shear stress profile over the first 4 m of pile penetration. Both approaches match the measured profile better than the Dutch method near the pile tip. The UWA-05 approach that incorporates the plugging effect provides the closest match to the measured stresses in this region.
- (3) In the 146 day test Fig. 13(b), all methods under-estimate the shaft resistance developed below 2.25 m bgl.

4.3 Driven cast-in-place piles

Flynn and McCabe (2016) described instrumented pile load tests performed on three driven cast-in-place piles installed at a site near Coventry, in the United Kingdom.

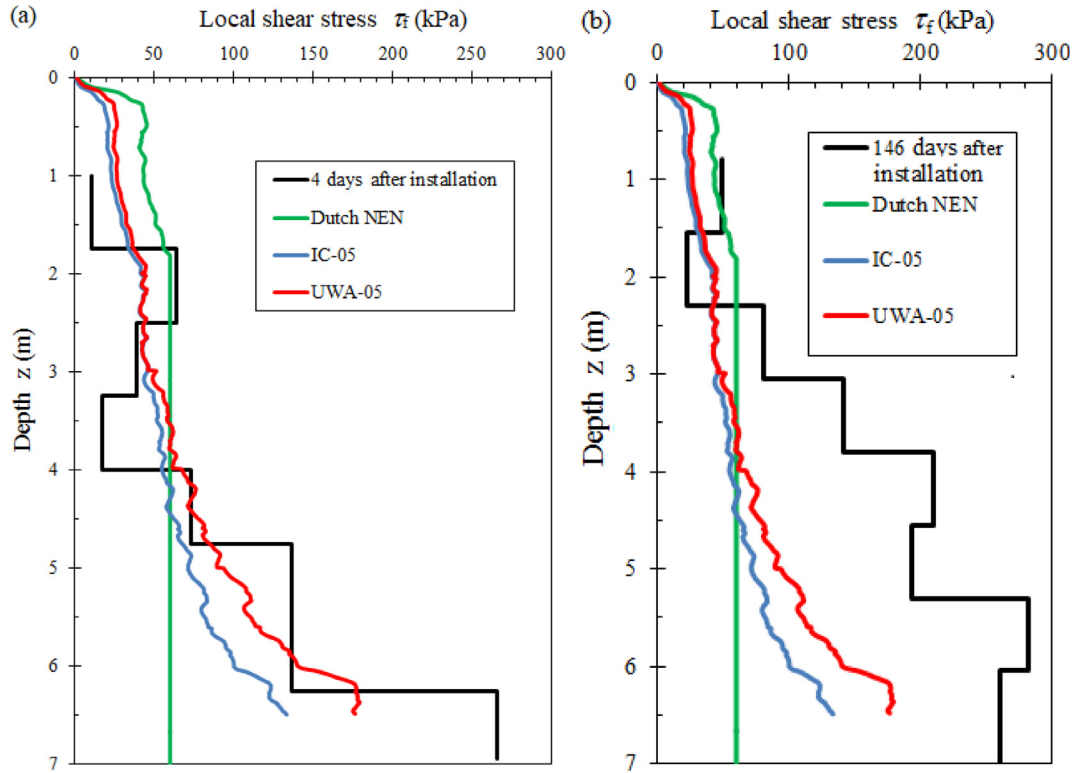


Fig. 13. Comparison of measured and predicted shear stress at Blessington: (a) 4 days load test, and (b) 146 days load test.

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. After the piles reached their final penetration at depths of between 5.5 m and 7 m bgl. the steel tube was filled with concrete and the steel casing was withdrawn. The ground conditions comprise made ground and stiff sandy clay to approximately 1.8 m bgl. underlain by medium dense to dense sand. The CPT profiles at the site are shown in Fig. 14. The piles were load tested in compression between 19 and 23 days after installation. The mobilisation of average shaft resistance (τ_{av} = total shaft resistance/shaft area) during the load tests is shown in Fig. 15. The three piles exhibited similar initial stiffness response with peak $\alpha_s = \tau_t/q_c$ values in the range of 0.094–0.14. 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.

4.4 Screw injection piles

The average shaft resistance mobilised by the screw injection piles installed in dense sand at a site in Ternaussen, Netherlands, described in Section 3.3 is shown in Fig. 16. 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

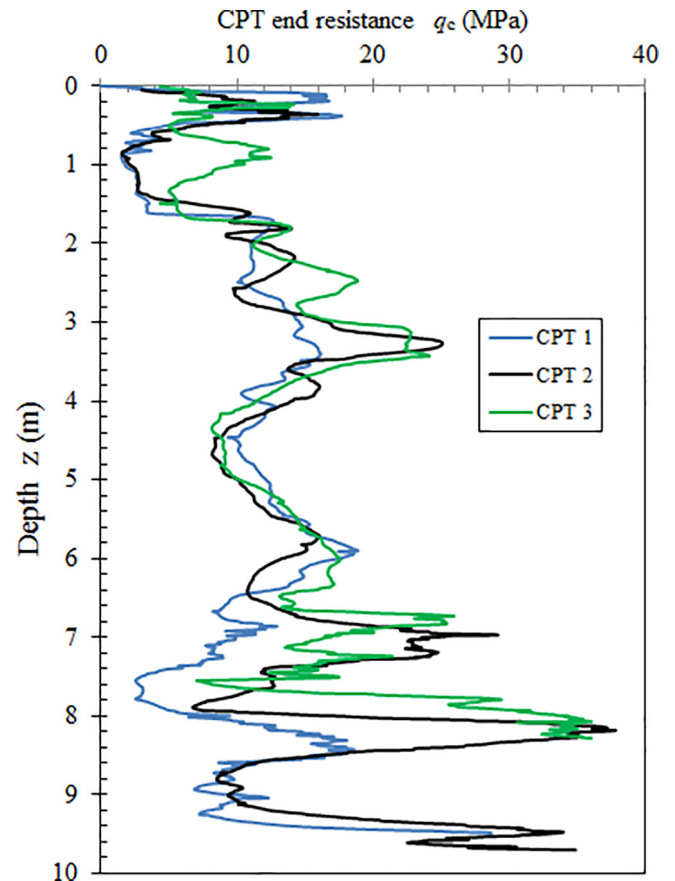


Fig. 14. CPT profiles at Coventry.

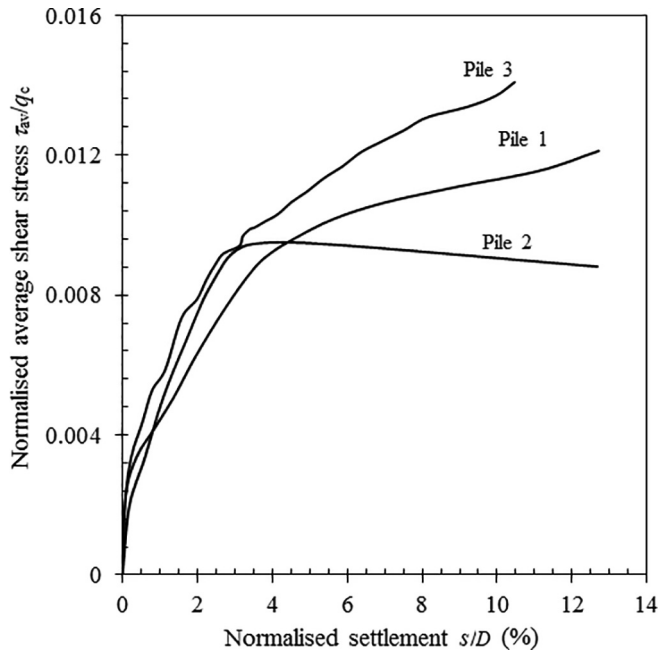


Fig. 15. Normalised shaft resistance mobilised by driven cast-in-place piles (after Flynn & McCabe, 2016).

the pile mobilised a much higher base resistance than the other test piles, see Fig. 7.

Notwithstanding this, it would seem that the ultimate shaft resistance of the piles was in the range of 110 kPa to 130 kPa. However, displacements in excess of 10% of the pile shaft diameter were required to mobilise this resistance. The back-figured α_s values show that the Dutch standard NEN-EN 9997-1 recommended α_s value of 0.09 would provide a reasonable estimate of the fully mobilised shaft resistance for these piles.

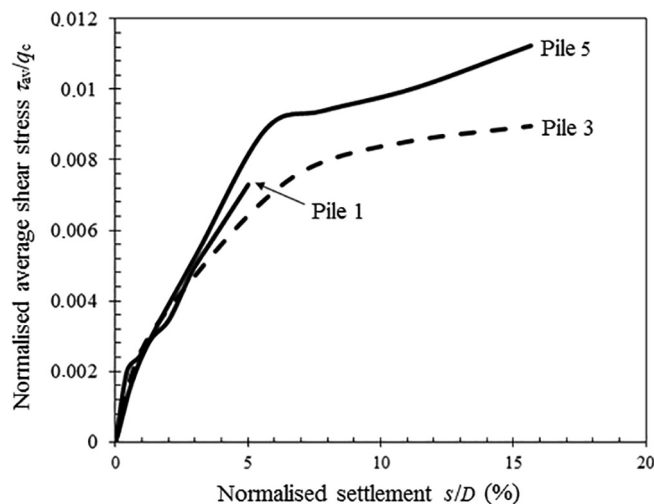


Fig. 16. Normalised shaft resistance mobilised by Screw Injection Piles at Ternaussen.

4.5 Continuous flight auger (CFA) piles

Gavin, Cadogan, and Casey (2009) report the local and average shaft resistance measured during the load tests on two CFA piles installed in Killarney, Ireland described in Section 3.4. The average shaft resistance (τ_{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 α_s ($=\tau_{av}/q_c$) value of 0.008 shown in Fig. 17 is 33% higher than those given in the NEN, and are similar to those used for the design of displacement piles in sand, see Table 1. The relatively large displacement required to mobilise the peak resistance is again a feature of the pile tests.

4.6 Distribution of normalised shaft resistance on piles in sand

Whilst the α_s values mobilised by the test piles described above conformed broadly with the constant values proposed in the Dutch code and seemed to depend on pile type, with higher values generally pertaining to driven piles, there remains some questions over whether a constant α_s value is appropriate for displacement piles where friction fatigue effects may be important. Some insight into possible reasons for this can be determined by the instrumented load tests. The normalised local shear stress profile along the driven cast-in-place (DCIP) pile (Flynn & McCabe, 2016) is compared in Fig. 18 with the profile predicted using the UWA-05 approach with $\delta_f = \phi'_{cv}$ (ϕ'_{cv} is the critical state friction angle). It is clear that a method which includes for the effects of friction fatigue provides

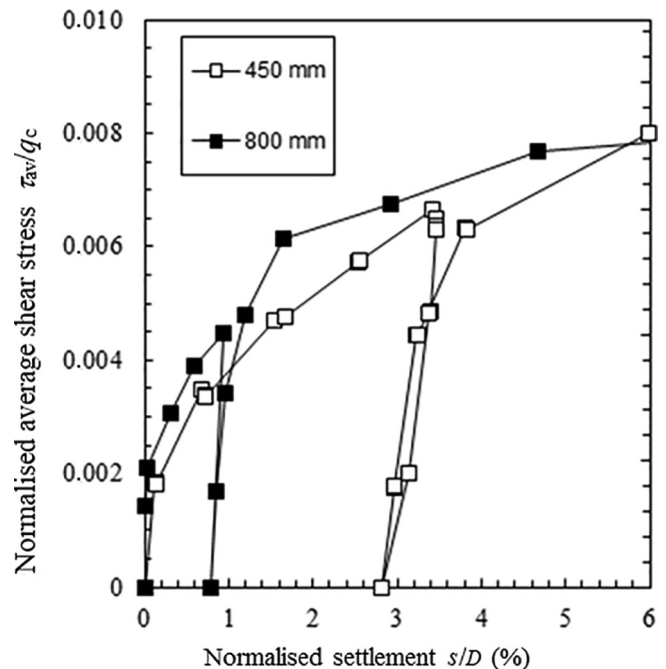


Fig. 17. Normalised shaft resistance mobilised by Continuous Flight Auger Piles (after Gavin et al., 2009).

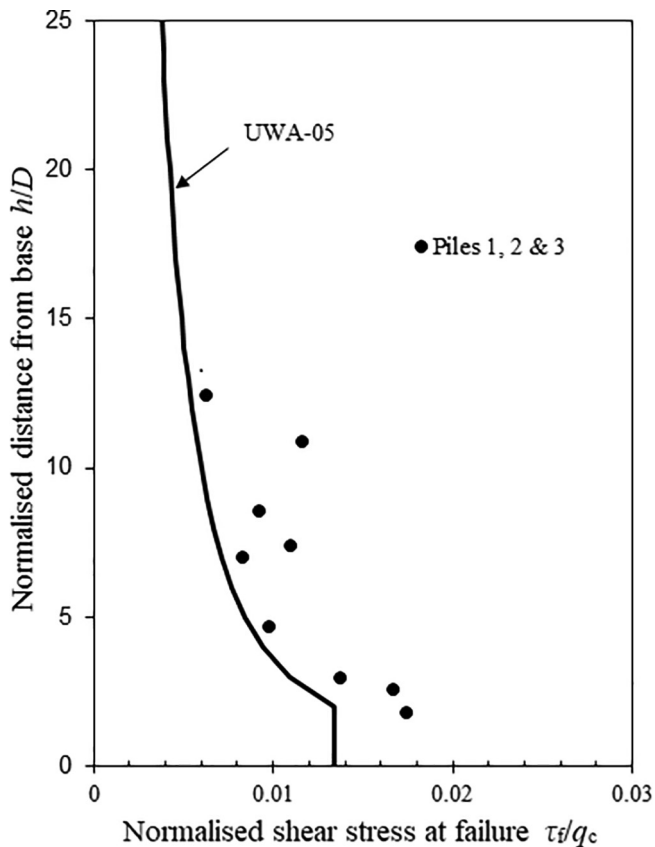


Fig. 18. Variation of α_s values with distance from the pile tip for DCIP pile at Coventry.

a very good match to the measured response and suggests that for piles where friction fatigue occurs during installation that α_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 Fig. 19. For these piles that do not experience friction fatigue during installation, a constant α_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 α_s values. Thus it is suggested that non-displacement piles with deep embedment lengths could mobilise higher average α_s values than piles driven in the same deposit.

5 Conclusions

Given that the CPT provides a continuous 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 and the accuracy of the Dutch design values are examined.

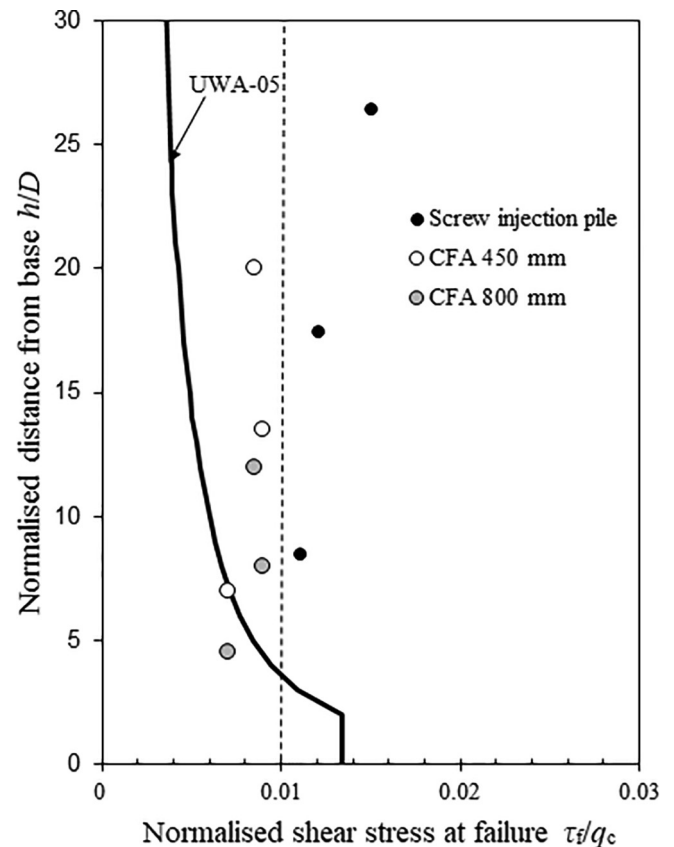


Fig. 19. Variation of α_s values with distance from the pile tip for low-displacement piles.

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 (2005) and Randolph (2003). 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. 10% of the pile diameter, the stiffness response depends on the pile installation method. For low displacement pile types α_p in the range of 0.15–0.56 are typically adopted in practice. For displacement piles, q_b/q_c of 0.6–0.7 is recommended for closed-end driven steel and concrete piles. From the result presented herein a particular feature of field tests on 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 not consider creep, showed α_p values around 50% higher than those measured in maintained load tests. In quick field-load tests where creep was minimised, α_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 α_p values can be adopted in design codes.

It would appear that due to the CPT averaging technique adopted and effect of residual loads the Dutch approach under-estimates the base resistance of closed-end piles. In contrast the approach over-estimates the resistance of open-end piles presented here. Both findings will in truth be dependent on the site conditions as in some cases errors are counter-balanced, i.e. the high α_p factor for open-end piles may be compensated by the Dutch q_c averaging technique which tends to return conservative values in very dense sand.

When considering shaft resistance, the Dutch code suggests constant α_s values, with recommended values for displacement piles being higher than non-displacement piles. The case histories presented herein strongly suggest that α_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 α_s than a non-displacement pile installed in the same soil deposit. Whilst ageing can benefit the shaft capacity of driven piles and therefore α_s values mobilized on displacement piles, Gavin and Igoe (2019) noted that important scale effects might need to be considered and further field testing to investigate such effects are necessary prior to adoption of ageing in design practice. In addition plugging effects for open-end piles which are ignored in the Dutch code should be explicitly considered.

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Conflict of interest

The authors are aware of no conflicts of interest.

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