Base Capacity of Open-Ended Steel Pipe Piles in Sand

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Abstract: This paper presents a new method for estimating the base capacity of open-ended steel pipe piles in sand, a difficult problem involving great uncertainty in pile foundation design. The method, referred to as the Hong Kong University (HKU) method, is based on the cone penetration test (CPT), and takes into consideration the mechanisms of annulus and plug resistance mobilization. In this method the annulus resistance is properly linked to the ratio of the pile length to the diameter—a key factor reflecting the influence of pile embedment—whereas the plug resistance is related to the plug length ratio, which reflects the degree of soil plugging in a practical yet rational way. The cone tip resistance is averaged over a zone in the vicinity of the pile base by taking into account the failure mechanism of the piles in sand, the condition of pile embedment (i.e., full or partial embedment), and the effect of soil compressibility. The predictive performance of the new method is assessed against a number of well-documented field tests including two fully instrumented large-diameter offshore piles, and through comparisons with major CPT-based methods in current engineering practice. The assessment indicates that the HKU method has attractive capabilities and advantages that render it a promising option. **DOI: 10.1061/(ASCE)GT.1943-5606.0000667.** © *2012 American Society of Civil Engineers.*

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Introduction

Steel pipe piles have been used increasingly as deep foundations for offshore and onshore structures. For example, more than 5,000 steel pipe piles were used in the construction of the Hangzhou Bay Bridge in China, the then-longest cross-sea bridge in the world. Steel pipe piles are usually open ended and, in most situations, driven to the foundation on competent strata such as dense sand. Determination of the base capacity of open-ended pipe piles is a difficult problem involving great uncertainty. The difficulty can be largely attributed to the complicated behavior of soil plugging. A column of soil tends to form as soil enters the pile from the pile tip during pile installation. Most of the earlier design methods did not differentiate between open- and closed-ended piles. Given an increasing demand for large-diameter open-ended pipe piles in offshore engineering, considerable effort has been made in recent years to investigate the loading behavior and bearing capacity of pipe piles in sand (e.g., Paikowsky and Whitman 1990; Jardine and Chow 1996; De Nicola and Randolph 1997; Lehane and Gavin 2001; Paik and Salgado 2003), leading to improved understanding and design methods. Nevertheless, current design methods remain largely empirical (Randolph 2003), relying heavily on the correlations derived from pile load tests and in situ penetration tests, and particularly on cone penetration tests (CPTs).

More recently, the American Petroleum Institute (API) issued an updated edition of practice for fixed offshore platforms (API 2006), in which four CPT-based design methods were included in the commentary, namely the Fugro, Imperial College pile (ICP), Norwegian Geotechnical Institute (NGI), and the University of Western Australia (UWA) methods. Reviews of the four methods have been documented in various forms in Lehane et al. (2005) and Schneider et al. (2008), showing that the UWA method (Lehane et al. 2005) and the ICP method (Jardine et al. 2005) have more advantages than the NGI method (Clausen et al. 2005) and the Fugro method (Kolk et al. 2005).

In this paper, the ICP and UWA methods are discussed with particular attention to their capability of accounting for the effect of soil plugging on pile base capacity, a key issue in the design of open-ended pipe piles, and the need for further improvement is identified. An improved approach, referred to as the Hong Kong University (HKU) method, is then presented along with the theoretical considerations and experimental observations behind it. The new method, which is also CPT based in order to take advantage of the widespread use of CPT data in pile foundation design, takes into consideration several important factors that have been largely ignored in current methods. The predictive performance of the new method is carefully assessed using well-documented field tests and through comparisons with the two major methods. This study is aimed at removing to some extent the heavy empiricism embedded in the current methods, while at the same time incorporating factors that can help capture the involved mechanisms properly. It represents one of the steps toward developing more cost-effective and rational methods for design of open-ended steel pipe piles.

Major Design Methods

ICP Method

The ICP method, formerly known as the Marine Technology Directorate (MTD) method (Jardine and Chow 1996), was developed from a database of pile load tests and CPT data, and targeted for both open- and closed-ended piles. To estimate the base capacity of pipe piles in sand, this method first requires determination of the plugging mode. With the aid of the empirical relationships given in Eq. (1),

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a pipe pile is determined as unplugged as long as either of the two following conditions is fulfilled:

$$d \geq 2.0(D_r - 0.3) \quad \text{or} \quad d \geq 0.03q_{c,a} \tag{1}$$

where d = inner diameter of the pile (m); D_r = relative density of the soil near the pile tip (as a decimal fraction); and $q_{c,a}$ = averaged CPT tip resistance over a specified range in the vicinity of the pile base (MPa). If none of the conditions in Eq. (1) are fulfilled, a rigid basal plug is assumed to form, and the pile is classified as fully plugged.

The ultimate unit base resistance of the pile, q_b, corresponding to $0.1D$ pile head displacement [where D is pile outer diameter (m)], is then calculated for unplugged and plugged conditions, respectively, as follows:

$$\begin{cases} \text{unplugged:} \; q_b/q_{c,a} = 1 - (d/D)^2 \\ \text{plugged:} \; q_b/q_{c,a} = \max\left[0.14 - 0.25 \log D, \, 0.15, \, 1 - (d/D)^2\right] \end{cases} \tag{2}$$

Note that for an unplugged pile, the ICP method assumes that the base capacity is provided only by the annular area, with a unit resistance of $q_{c,a}$. However, for a fully plugged pile the unit base resistance is taken as half of the base resistance of an identical closed-ended pile (Jardine et al. 2005) and is subjected to two lower limits—the base resistance of an identical unplugged pile and 15% of $q_{c,a}$.

It is evident that the ICP method treats the internal diameter of the pile (d) and the relative density of the sand at the pile base (D_r) as the main factors governing soil plugging and base capacity. For open-ended piles installed in sand, the degree of soil plugging is also closely related to the embedded lengths of the piles. There is adequate evidence that piles having large values of embedment are more likely to be plugged than piles of short embedment (Paikowsky and Whitman 1990; De Nicola and Randolph 1999). This important factor is not explicitly incorporated in the ICP method.

Moreover, the ICP method assumes that there are only two extreme cases of plugging; i.e., fully plugged and fully coring. However, there is evidence of the existence of a partially plugged mode (Paikowsky and Whitman 1990; O'Neill and Raines 1991). In this mode the plug of soil moves for a distance less than the base displacement as the pile penetrates. Additionally, for the unplugged mode the ICP method tends to give conservative predictions because it simply excludes the contribution of plug resistance. This underestimation can become significant in some situations where large friction is mobilized along the interface between the soil column and the inner wall of the pile, which is the case for many offshore piles. Given the aforementioned observations, a major concern here lies in how to account for the effect of soil plugging in a more rational manner such that the base capacity can be determined with increased reliability.

UWA Method

The UWA method was developed largely from the ICP method by incorporating several modifications. In this method the base capacity of an open-ended pipe pile, corresponding to a base displacement of $0.1D$, is calculated from a single empirical correlation that was calibrated from a database of 13 pile load tests (Xu et al. 2008) as follows:

$$q_b/q_{c,a} = 0.6 - 0.45(d/D)^2 \text{IFR} \tag{3}$$

where the incremental filling ratio (IFR) of the soil plug = ratio between the increment of soil plug length and the increment of pile penetration depth (Paikowsky et al. 1989; Paik and Salgado 2003) (see Fig. 1). Note that the IFR in the UWA method is taken as an averaged value over the last $3D$ of pile penetration. In calculating $q_{c,a}$ in Eq. (3), the Dutch method (de Kuiter and Beringen 1979) is used for averaging the CPT tip resistance over a zone extending from $0.7D$ to $4D$ below the pile base to $8D$ above the pile base. However, in the ICP method the averaged zone extends from $1.5D$ below the pile base to $1.5D$ above the base.

Compared with the ICP method, the UWA method does not require determination of the plugging mode beforehand. It employs the parameter IFR to allow for the degree of plugging. While this improvement is a step forward, the averaged IFR value over the final $3D$ penetration cannot be determined easily during pile installation, particularly in the offshore environment. Moreover, as will be shown subsequently, the Dutch method adopted for averaging the CPT tip resistance does not work well in some situations. One more point worth noting is that, while recognizing the existence of the partially plugged mode, the UWA method does not offer explicit estimates of individual contributions from the annulus and plug to the base capacity. Rather, it seeks to make, as with the ICP method, an overall estimate of the base capacity using a single empirical correlation.

New Approach: The HKU Method

Physically, an open-ended pile should derive its base capacity from two components, the pile annulus and the soil plug, as schematically shown in Fig. 1. Depending on the degree of plugging, the two components of resistance can behave quite differently under axial loading.

With respect to the unit resistance beneath the pile annulus, it should be comparable to that of a closed-ended pile at plunging, especially for long piles associated with high-stress levels. As for the plug resistance, it can largely differ in stiffness and the load-transfer mechanism from the annulus resistance. The upper portion of the soil plug (see Fig. 1) is likely to be heavily disturbed owing to pile penetration, leading to insignificant side resistance mobilized over this range (O'Neill and Raines 1991; Paik and Salgado 2003). Thus, it is acceptable to neglect this small side resistance and approximately treat this part of the soil as a surcharge load acting on the lower portion of the soil plug. On the other hand, the bearing capacity of the soil beneath the soil plug should, initially, be greater than the sum of the plug weight and the friction between the soil and the inner wall of the pile. The height of the soil plug then tends to increase until a limiting equilibrium is achieved and a fully plugged mode is formed. In view of the previous observation, for practical purposes, it is both necessary and desirable to develop an improved method that allows determination of the individual resistance of the annulus and the plug from considerations of the mechanics involved. This is the goal of the HKU method.

Annulus Capacity

The base resistance of a displacement pile in sand is governed by the packing density, stress level, stiffness, and compressibility of the sand in the vicinity of the pile base (Yang et al. 2005). It has long been recognized that the deformation beneath a pile base resembles the expansion of a spherical cavity (e.g., Vesic 1972). From the viewpoint of cavity expansion modeling, the shape and size of a pile base are linked with the initial radius of the cavity, and the limit cavity pressure or, correspondingly, the base capacity is not affected by this initial radius (Yu 2004). This implies that the annulus capacity is similar to the base capacity of a closed-ended pile. Indeed, observations from model pile tests (e.g., Lehane and Gavin 2001) are in support of this theoretical consideration. Along this line, the

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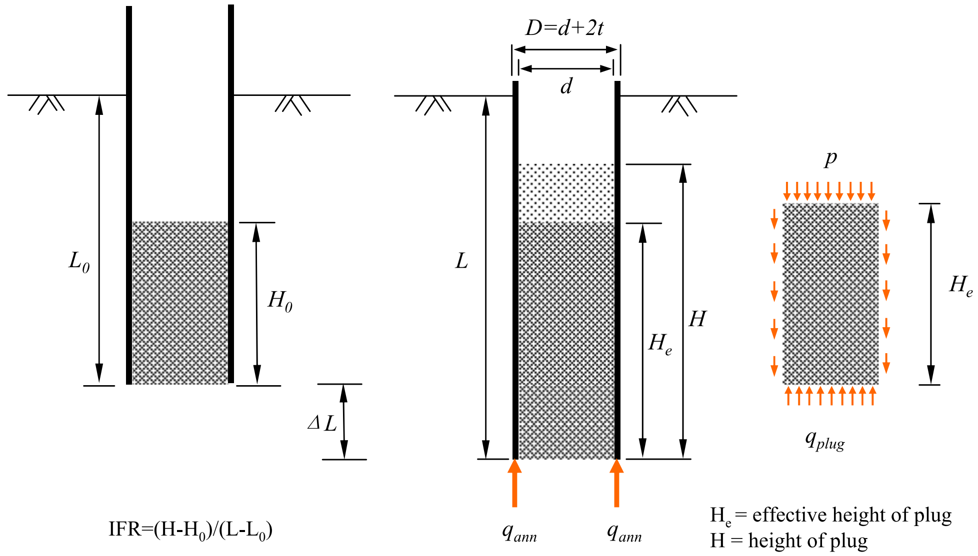

Fig. 1. Schematic illustration of soil plug formation and the load transfer mechanism

correlations available for base capacity of closed-ended piles can be transferred to the annulus capacity for open-ended piles.

In the ICP and UWA methods, the base resistance of a closed-ended pile is determined, respectively, as

$$\begin{cases} \text{ICP method}: q_b = (0.28 - 0.5 \log D)q_{c,a} \geq 0.3q_{c,a} \\ \text{UWA method}: q_b = 0.6q_{c,a} \end{cases} \tag{4}$$

where D = pile diameter (m). The two expressions in Eq. (4) show that while the ICP method suggests the base resistance, normalized by the cone tip resistance, decreases with increasing pile diameter, the UWA method suggests the normalized base resistance is a constant (0.6) independent of the pile diameter. This inconsistency is obviously not logical and leads to confusion for practitioners. Also note that both empirical correlations in Eq. (4) do not explicitly include the influence of pile embedment or the associated stress level.

The state-dependent analysis of Yang and Mu (2008) suggests the need to incorporate the embedded length in the study of base capacity for piles in sand. This need is also supported by observations from centrifugal chamber tests that simulate prototype stress levels (De Nicola and Randolph 1997). By analyzing the centrifuge tests of De Nicola and Randolph (1997) for pipe piles, the dependence of annulus resistance on pile length can be established as

$$\begin{cases} q_{\text{ann}} = (1.06 - 0.03L)q_{c,a}, & L < 20 \text{ m} \\ q_{\text{ann}} = 0.46q_{c,a}, & L \geq 20 \text{ m} \end{cases} \tag{5}$$

where q_{ann} = unit annulus resistance (MPa) and L = pile length (m). In deriving the aforementioned relationships, the annulus resistance is taken as that corresponding to 0.1D base displacement and the mean effective bulk density of the sand is taken to be 10 kN/m^3.

As stated previously, the ratio of the pile length to the diameter (L/D) is a parameter reflecting the condition of partial embedment, which is a notable case in CPT-based evaluation of pile base capacity (White and Bolton 2005). Therefore, it is advisable to further improve Eq. (5) such that this L/D ratio, or pile slenderness, can be properly incorporated. With this aim, the centrifuge model tests of De Nicola and Randolph (1997) are reinterpreted in terms of annulus resistance and L/D, as shown in Fig. 2. Remarkably, the annulus resistance, normalized by the corresponding CPT tip resistance, has a fairly good correlation with L/D values, showing that the normalized annulus resistance decreases linearly with an increase in L/D. In addition, Fig. 2 suggests that the normalized annulus resistance is not sensitive to the relative density of sand when the former is plotted against pile slenderness. A possible explanation for this observation is that the effect of relative density has been inexplicitly accounted for by the CPT tip resistance and pile length.

Given the data points in Fig. 2, the following expression is proposed to relate the annulus resistance with the L/D value:

$$q_{\text{ann}} = [1.063 - 0.045(L/D)]q_{c,a} \tag{6}$$

As the trend line will yield negative values of the annulus resistance for large L/D values, the ratio between q_{ann} and $q_{c,a}$ needs to be imposed by a lower bound. Keeping in mind that Eq. (5) has suggested a limiting value of 0.46 for $q_{\text{ann}}/q_{c,a}$ for long piles ($L \geq 20$ m), it is natural and logical to rewrite Eq. (6) as follows:

$$q_{\text{ann}} = [1.063 - 0.045(L/D)]q_{c,a} \geq 0.46q_{c,a} \tag{7}$$

Eq. (7) provides a useful explicit relationship between the normalized annulus resistance and the combination of pile embedment and diameter.

Recently, Paik et al. (2003) reported field tests on a closed-ended pipe pile and an open-ended pipe pile driven into a gravelly sand deposit. The two piles had the same outer diameter (0.356 m) and a similar embedment of about 7 m. For purposes of comparison, the measured base resistance of the closed-ended pile and the annulus resistance of the open-ended pile are superposed on the plot in Fig. 2. The lower bound in Eq. (7) appears to be reasonable for the closed-ended pile; however, it is conservative for the open-ended pile. As there is currently a lack of high-quality field test data, it would be wise not to raise the lower bound until sufficient field test data become available in the future.

Plug Capacity

The plug capacity is mainly mobilized from the friction along the inner pile wall, particularly along the lower part of the soil plug

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where soil arching is significant and a large lateral coefficient of earth pressure is achieved. This arching effect was well observed in field testing of a concrete pipe pile (Liu et al. 2012), as schematically shown in Fig. 3 where the CPT profile at the site is also included. Note that the standard CPT cone used in China has a projected cross-sectional area of 15 cm^2, which is 50% larger than what is widely used outside of China. The friction along the inner pile wall is closely related to the development of plug length during pile installation, and the arching effect is also responsible for the rotation of principal stresses in the soil adjacent to the inner wall.

As discussed previously, the IFR is a measure of the degree of plugging. The fully plugged and fully coring modes are represented by IFR = 0 and IFR = 100%, respectively. For the partially plugged mode, the IFR varies between the two limiting values. The value of the IFR depends on a number of factors (Paikowsky and Whitman 1990; De Nicola and Randolph 1997; Lee et al. 2003), including the relative density of the sand near the pile base, the pile inner diameter, and the pile embedment. The major effects of these factors can be summarized as follows:

1. Piles installed in dense sand tend to plug more than those in loose sand, indicating that the IFR tends to decrease with an increase in the relative density of the sand.
2. The IFR tends to increase as the inner diameter of the pile increases.
3. The IFR will vary inversely with pile length or penetration depth; this is because longer pipe piles are more likely to be fully plugged.

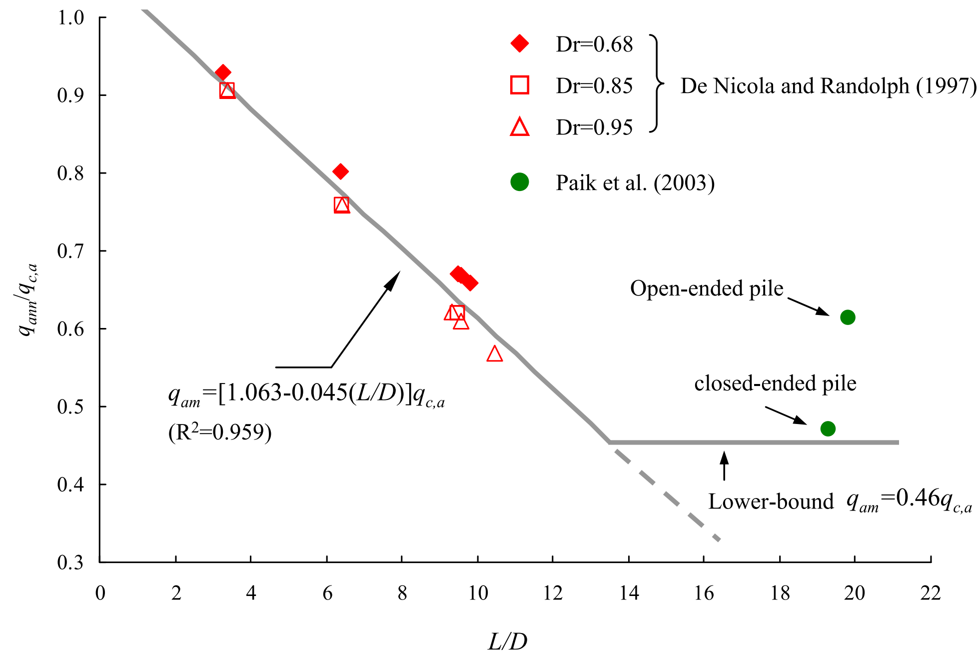

Fig. 2. Proposed relationship between normalized annulus resistance and pile slenderness

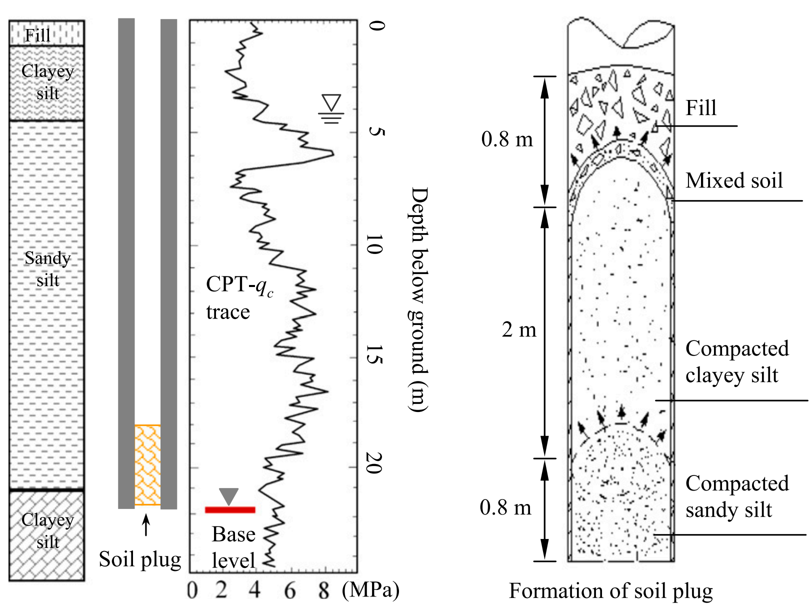

Fig. 3. Field observation of soil plug formation and soil arching (data from Liu et al. 2012)

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In real applications, particularly in the offshore environment, it is not easy to determine the IFR, which involves continuously measuring the soil plug length during pile installation and also recording the penetration depth of the pile. An alternative is the plug length ratio (PLR), defined as H/L, where H is the length of the plug measured at the end of pile installation (see Fig. 1). Because both the IFR and PLR reflect the degree of soil plugging, they should be related to each other in some manner. Indeed, the model tests of Paik and Salgado (2003) showed that the PLR and IFR have a fairly good correlation as

$$\mathrm{PLR} = 0.917\mathrm{IFR} + 0.202 \tag{8}$$

where IFR (in decimals) is measured at the final penetration depth.

For the fully plugged mode and fully coring mode, Eq. (8) yields PLR = 0.202 and 1.119, respectively. It may be questioned why a fully plugged pile has a PLR value being greater than zero. This is because in the initial stages of pile installation and prior to the formation of a fully plugged mode, a column of soil may enter the pipe. Also, note that the value of the PLR can be greater than unity for a fully coring pile, meaning that the top of the soil column inside the pipe is above the ground level—this case was reported by Kikuchi et al. (2007) in testing full-scale offshore piles. Of course, in estimating pile capacity for such cases, a reasonable approximation can be taken such that PLR = 1.

A key problem here is to find out how plug resistance is related to the index PLR. In exploring the relationship, a database consisting of three sets of tests is compiled and analyzed. Table 1 summarizes the details of these tests. A total of 48 sets of data are plotted in Fig. 4, where plug resistance is normalized by CPT tip resistance and then is expressed as a function of the PLR after installation. A data point derived from a field-scale load test by Paik et al. (2003) on a pipe pile ($L = 7.04$ m; $D = 356$ mm; $d = 292$ mm) is also included in the plot. Note that in the cases where only the IFR values are available, Eq. (8) has been used to derive the PLR values.

A trend line for the test data in Fig. 4 can be proposed in the form of

$$q_{\mathrm{plug}} = a\exp(b\mathrm{PLR})q_{c,a} \tag{9a}$$

Table 1. Details of Model Pile Tests Used for Analysis of Plug Resistance

Reference	Description
De Nicola and Randolph (1997)	Pile geometry: $L = 5.2$–16.7 m, $D = 1.6$ m, and $d = 1.49$ m in prototype Soil property: silica flour; $D_r = 68$, 85, and 95% Test method: centrifuge chamber tests; installed by driving and jacking Remarks: 14 sets of data used (12 by driving and two by jacking); PLR available
Lehane and Gavin (2001)	Pile geometry: $L = 1$ and 1.55 m; $D = 40$ and 114 mm; $d = 37.6$–97.4 mm Soil property: siliceous sand; $D_r = 30 \pm 2\%$ Test method: chamber tests; all installed by jacking Remarks: 10 sets of data used; PLR inferred from IFR by Eq. (8)
Lee et al. (2003)	Pile geometry: $L = 0.25$–0.76 m; $D =4$ 2.7 mm; $d = 29.9$ and 36.5 mm Soil property: siliceous sand; $D_r = 23$, 56, and 90% Test method: calibrated chamber tests; all installed by driving Remarks: 24 sets of data used; PLR inferred from IFR by Eq. (8)

where q_{plug} = unit base resistance of the soil plug. Parameters a and b are here determined as 1.063 and −1.933, respectively, and Eq. (9a) is rewritten as

$$q_{\mathrm{plug}} = 1.063\exp(-1.933\mathrm{PLR})q_{c,a} \tag{9b}$$

Eq. (9b) shows that the normalized plug resistance takes a maximum value (1.063) for PLR = 0. Note that PLR = 0 represents an extreme case of the fully plugged mode in which no soil comes into the pipe throughout the process of installation and loading. In this extreme case an open-ended pipe pile will behave similarly to a closed-ended pile. Keeping this in mind, and to be consistent with the previous proposal in Eq. (7) for the annulus resistance, parameter a in the exponential function in Eq. (9a) is first fixed at 1.063, and another parameter b (−1.933) is then determined by a best-fit procedure. The trend line thus determined has a coefficient of determination of about 0.67. If parameter a is not fixed, the generated best fit has almost the same coefficient of determination as the one given in Eq. (9b); however, the beauty of the consistency between Eqs. (9b) and (7) for the special case of PLR = 0 is lost.

Furthermore, for a fully plugged pile with nonzero PLR values (which is common in real applications), say IFR = 0 and PLR = 0.202 according to Eq. (8), the proposal in Eq. (9b) yields a plug capacity equal to 68% of the base capacity of a closed-ended pile. This is quite a sound prediction because it reflects the compressibility of the soil plug compared with the real closed pile base. Also, the index PLR in Eqs. (9a) and (9b) can help allow for the influence of soil properties and pile embedment on plug capacity because its value is affected by these properties.

In recent years, large-diameter and thin-walled tubular piles have received increasing applications. These piles usually have higher values of the PLR. For instance, the observations of Lu et al. (1999) show that the PLR values of steel pipe piles with a diameter of 610 mm range from 0.625 to 0.795, being much larger than those of small-diameter, thick-walled concrete pipe piles. Given the proposal in Eqs. (9a) and (9b), these observations indicate that small-diameter piles can develop larger unit plug resistance, which is in good agreement with the findings of the numerical study of Liyanapathirana et al. (1998).

As far as the method of pile installation is concerned, it should be noted that jacked piles are more likely to plug than identical driven piles, as observed in laboratory experiments (e.g., De Nicola and Randolph 1997). A similar observation was also found at the field scale for a number of concrete pipe piles installed by jacking and driving (Qin 2008). In this connection, the influence of the installation method on plug capacity can preliminarily be accounted for through the index PLR in Eqs. (9a) and (9b). In other words, the proposed relationship in Eqs. (9a) and (9b) can, to a first approximation, apply to jacked piles. When more high-quality data become available for jacked pipe piles, Eqs. (9a) and (9b) can be further refined or improved.

Influence Zone for End Bearing

In CPT-based design methods, averaging is often taken to derive $q_{c,a}$ for calculation of pile base capacity. The influence zone specifies the range in which the CPT-q_c trace should be taken in calculating the average value. Table 2 summarizes several proposals for the size of the influence zone, where A and B represent the range of the zone above and below the pile base, respectively (see Fig. 5). The averaging techniques adopted in the ICP and UWA methods are briefly described subsequently, along with that adopted in the CPT-based methods currently used in China, JGJ 94-2008 (CABR 2008) and TBJ37 (CMR 1993):

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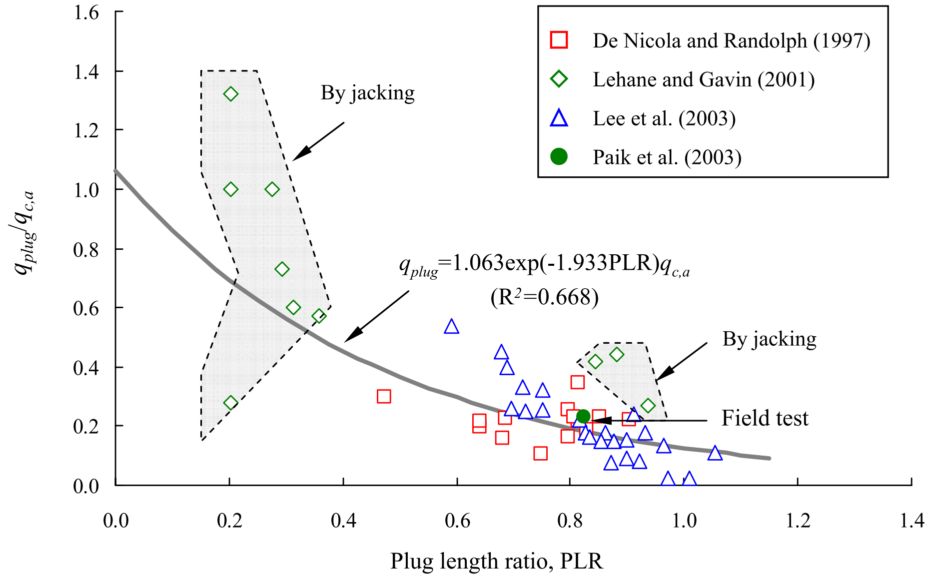

Fig. 4. Proposed relationship between the normalized plug resistance and PLR

Table 2. Various Proposals for Influence Zones for End-Bearing Analysis

	Method				Yang (2006)	
Influence zone	ICP	UWA	JGJ94	TBJ37	Sand with low compressibility	Sand with high compressibility
A	1.5*D*	8*D*	4*D*	4*D*	(1.5–2.5)*D*	(0.5–1.5)*D*
B	1.5*D*	(0.7–4)*D*	1*D*	4*D*	(3.5–5.5)*D*	(1.5–3)*D*

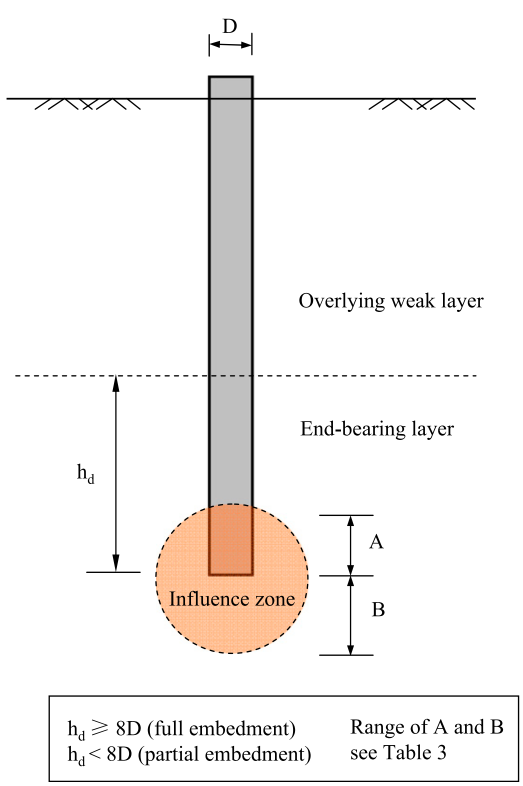

Fig. 5. Influence zone for averaging the cone tip resistance near the pile base (HKU method)

1. The ICP method simply takes an average over the range of $A + B$. When the variation of q_c within the influence zone is remarkable, a q_c value below the mean is recommended.
2. The UWA method takes an average over the range of *B* to get Value 1, finds the minimum within the range of *B*, and averages it with Value 1 to get Value 2. It then takes an average of the envelope of minimums recorded over the range of *A* to get Value 3 and, finally, uses the mean of Values 2 and 3.
3. The JGJ 94-2008 method takes an average over the range of *A* and *B* to get Values 1 and 2, respectively, and uses the mean of Values 1 and 2.
4. The TBJ37 method takes an average over the ranges of *A* and *B* to get Values *1* and 2, respectively, and then uses the mean of Values 1 and 2 provided Value 1<Value 2; otherwise it uses Value 2.

The proper averaging of q_c around the pile base is still an unresolved issue. However, it plays an important role in CPT-based pile design (Yang 2006; Salgado 2008). There are several reasons that necessitate a serious examination of the influence zone, including (1) the contrast of the size of a CPT cone and that of a pile base; (2) the contrast of the displacement required for mobilizing the CPT tip resistance and that for mobilizing the pile base resistance; and (3) the contrast of soil heterogeneities involved in loading a CPT cone and a pile base. The UWA and JG 94-2008 methods follow a similar concept that the base capacity is influenced more by the soil above the pile base than by the soil below the base. A possible consideration underlying this practice has been discussed by Yang (2006) from the perspective of the failure patterns of piles in sand. This practice is possibly reasonable or at least conservative in the situation where piles are partially embedded into the end-bearing layer such that the piles can still feel the effect of the overlying softer

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layer. A real pile can be affected more by the softer layer than the CPT cone if the pile tip penetrates within 8D below the soft layer (White and Bolton 2005).

However, in many cases of practical interest piles are usually driven into stiff strata for a sufficiently large distance and the overlying softer strata, if any, have little effect. For this full embedment mode, punching or local shear failure rather than the general shear failure will be dominant. An axially loaded pile is analogous to a spherical cavity expansion, such that the influence zone is linked with the plastic zone in the cavity expansion modeling (Yang 2006). In recognition of the importance of state-dependent sand properties, Yang (2006) has revealed that the size of the influence zone depends on a number of factors including the relative density and stress level of the sand at the pile base and the compressibility of the sand (see Table 2). The effect of compressibility deserves particular attention in offshore applications where highly crushable sand is involved.

Given the previous considerations, the HKU method recommends a set of influence zones for various conditions of pile embedment and soil compressibility (Table 3). Under the condition of partial embedment, the customary practice that the influence zone above the pile base is not smaller than that below the pile base is retained in cases where the variation of q_c is significant; when the variation of q_c is insignificant, the use of the $\pm 1.5D$ range as in the ICP method is adopted. Under the condition of full embedment, the influence zone proposed by Yang (2006) is adopted.

The averaging technique for calculation of $q_{c,a}$ in the HKU method generally involves two steps:

1. Take an average of the q_c trace within the range of A or B defined previously. The averaged values are denoted by M_A and M_B, respectively. The M_A and M_B are determined by the geometric mean as

$$M_A \text{ or } M_B = \sqrt[n]{q_{c1}q_{c2}\ldots q_{ci}\ldots q_{cn}} \tag{10}$$

   where q_{ci} = ith CPT-q_c number recorded over the range of A or B. The geometric mean rather than the arithmetic mean is suggested here because it can reduce the uncertainty associated with dramatic variations in CPT profiles.
2. If $M_A \leq M_B$ is satisfied, let $q_{c,a} = 0.5 \times (M_A + M_B)$; otherwise, let $q_{c,a} = M_B$. To allow for spatial variability of CPT-q_c profiles in practice, it is recommended, if applicable, that an average q_c profile is developed from several CPT logs at the site before applying the averaging technique.

Overall Base Capacity

Given the annulus and plug resistance, the overall base capacity of an open-ended pile (Q_b) can be determined by

Table 3. Influence Zones for End-Bearing Analysis Recommended by the HKU Method

Case	Soil condition	Range above pile base: A	Range below pile base: B
Case 1. Partial embedment: $h_d < 8D$	Extreme variation in q_c	8D	1D
	Other situations	1.5D	1.5D
Case 2. Full embedment: $h_d \geq 8D$	Embedded in sand of low compressibility	2D	4.5D
	Embedded in sand of high compressibility	1D	2.5D

Note: h_d = penetration depth in the end-bearing layer (Fig. 5).

$$Q_b = \frac{\pi}{4}\left[d^2 q_{\text{plug}} + (D^2 - d^2)q_{\text{ann}}\right] \tag{11}$$

where q_{ann} and q_{plug} are calculated from Eq. (7) and from Eqs. (9a) and (9b), respectively. As a common practice, the base capacity calculated here corresponds to a pile base displacement of 10% of the pile diameter D.

Case Studies

There has been a lack of high-quality test data for piles in sand; particularly, there is a dearth of test data for open-ended pipe piles with adjacent CPT profiles. Table 4 lists nine field tests on open-ended steel pipe piles in sand, for which relevant CPT and IFR or PLR data are available in the public literature. In particular, the database here includes two fully instrumented large-diameter steel pipe piles tested in Tokyo Port Bay (Kikuchi et al. 2007), which provide a valuable opportunity to examine the performance of the new and existing methods when applied to real offshore piles.

Note that for Test Piles P1–P6 reported by Xu et al. (2008), only the profiles of the IFR are given. The values of the PLR for these piles can be derived using the following equation:

$$\text{PLR} = \frac{1}{L}\int_0^L \text{IFR}\, dz \tag{12}$$

When PLR values are not available from pile trial tests or there is no past experience on similar sites and piles for reference, a preliminary estimate of the PLR value can be made by

$$\text{PLR} = \left(\frac{d}{100}\right)^{0.15} \tag{13}$$

Here, d = inner diameter of the pile (mm). The aforementioned empirical relationship is developed from analysis of the database in Table 4, which is found to offer a fairly good fit to the test data (Fig. 6). For large-diameter pipe piles in which the PLR values probably go beyond unity, imposing an upper bound (PLR = 1) is suggested. It should be mentioned that while it appears to be an attractive proposal for practical use, Eq. (13) may require improvement when new quality data are available. For real applications, the recommended practice is to conduct reliable measurements of the PLR values through trial piles.

Before examining the performance of the three methods, Fig. 7(a) shows an example of the influence zones determined by the three methods for the test pile of Paik et al. (2003). The soil profile of the site is relatively uniform, with only one notable layer interface at about 3 m below the ground, where the CPT tip resistance shows a dramatic increase. It is evident that the condition of full embedment is fulfilled, and the influence zone is determined by the HKU method to be 2D above the pile base and 4.5D below the base. Using the HKU method, the geometric averages within A and B are determined from the CPT-q_c trace as M_A = 22.35 MPa and M_B = 22.74 MPa, and because $M_A < M_B$, $q_{c,a}$ is taken as the mean of M_A and M_B; i.e., $q_{c,a}$ = 22.55 MPa. By comparison, the values of the averaged cone tip resistance $q_{c,a}$ determined using the ICP and UWA methods are 21.66 and 17.91 MPa, respectively. Table 5 summarizes the calculated $q_{c,a}$ values using various methods for all test piles. Generally, the $q_{c,a}$ values determined by the HKU method show a balanced agreement with the $q_{c,a}$ values determined by the UWA and ICP methods.

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Table 4. Details of Test Piles Used in the Case Studies

Reference	Test pile	L (m)	D (mm)	d (mm)	IFR	PLR	Property of end-bearing soil
Jardine et al. (2005)	—	47	763	691	0.89	1.0	Dense sand; $D_r \approx 0.87$
Paik et al. (2003)	—	7.04	356	292	0.8	0.824	Dense gravelly sand, $D_r \approx 0.8$
Xu et al. (2008)	P1	4	88.9	83.7	0.69	0.76	Dry-to-moist yellow siliceous sand
	P2	4	42.4	37.2	0.5	0.44	
	P4	4	88.9	78.9	0.77	0.76	
	P5	4	114.3	107.9	0.85	0.88	
	P6	4	88.9	82.5	0.77	0.75	
Kikuchi et al. (2007)	TP4	73.5	1,500	1,444	1.0	1.0	Sandy gravel
	TP5	86	1,500	1,444	1.0	1.0	Sand

Note: d = pile inner diameter; D = pile outer diameter; D_r = relative density; IFR = incremental filling ratio over the last 3D penetration; L = pile length; and PLR = plug length ratio at the end of installation.

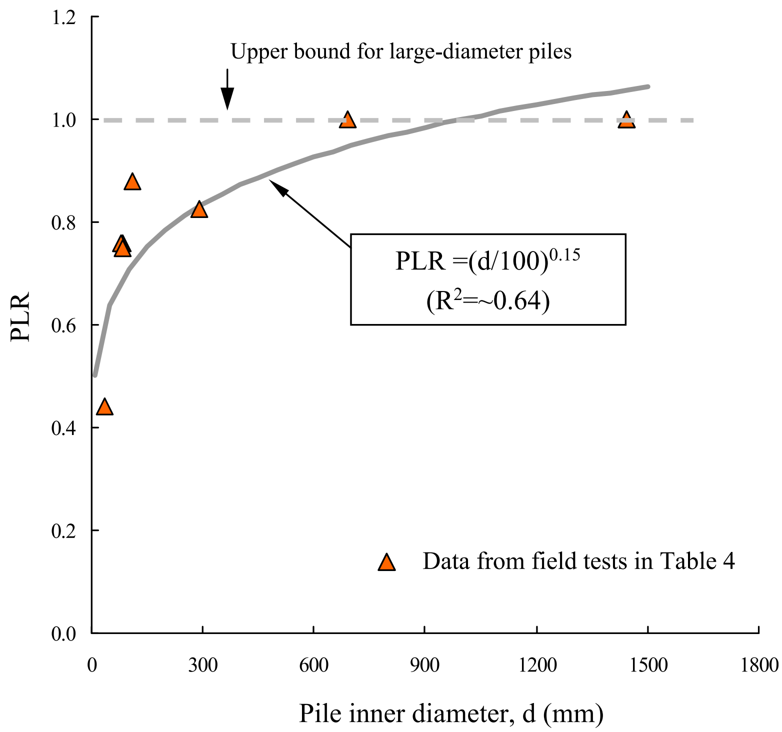

Fig. 6. Proposal for preliminary evaluation of PLR

For Pile TP4, the $q_{c,a}$ value determined by the HKU method, 43.03 MPa, is markedly lower than that determined by the ICP and UWA methods (84.4 and 57.20 MPa, respectively). A careful examination of the profile of CPT-q_c for this case [Fig. 7(b)] reveals that the significant difference is due mainly to the CPT-q_c trace having a large reduction in soils underneath the pile base. Given this fact, the $q_{c,a}$ value determined using the HKU method is considered more reliable and rational.

The values of base capacity predicted by the three methods are summarized in Table 6, together with the measured values and the statistics of their ratios. While the ICP method yields satisfactory predictions for the test piles of Jardine et al. (2005) and Paik et al. (2003), it significantly overpredicts the capacity for most of the test piles reported by Xu et al. (2008), and largely underpredicts the capacity of the offshore piles of Kikuchi et al. (2007). By comparison, the HKU and UWA methods both show an improved predictive performance. The performance of the three methods can also be viewed in Fig. 8, which shows the calculated base resistance against the measured ones for all test piles.

A further examination of the performance of the three methods is given in Fig. 9, where the ratios between the calculated and measured base resistances are plotted as a function of pile outer diameter, and in Fig. 10 where the calculated-to-measured ratios are plotted as a function of pile length. Note that while its size is limited, the database here covers a reasonably wide range of pile diameter (from about 40 to 1,500 mm) and a wide range of pile length (from 4 to 86 m). It is evident from Figs. 9 and 10 that the HKU method performed the best for such a wide range of pile-dimensions. By combining the pile diameter and length, Fig. 11 compares the calculated-to-measured ratios generated from the three methods with respect to the pile slenderness ratio, L/D. The HKU method performs consistently well over the wide range of L/D (approximately from 20 to 100), giving the most accurate predictions, with the mean value of the calculated-to-measured ratio being 1.02 and the coefficient of variation (COV) being 0.18.

It is of particular interest to examine the performance of the new method in predicting the base capacity of the two offshore large-diameter pipe piles, TP4 and TP5. The CPT-q_c profiles

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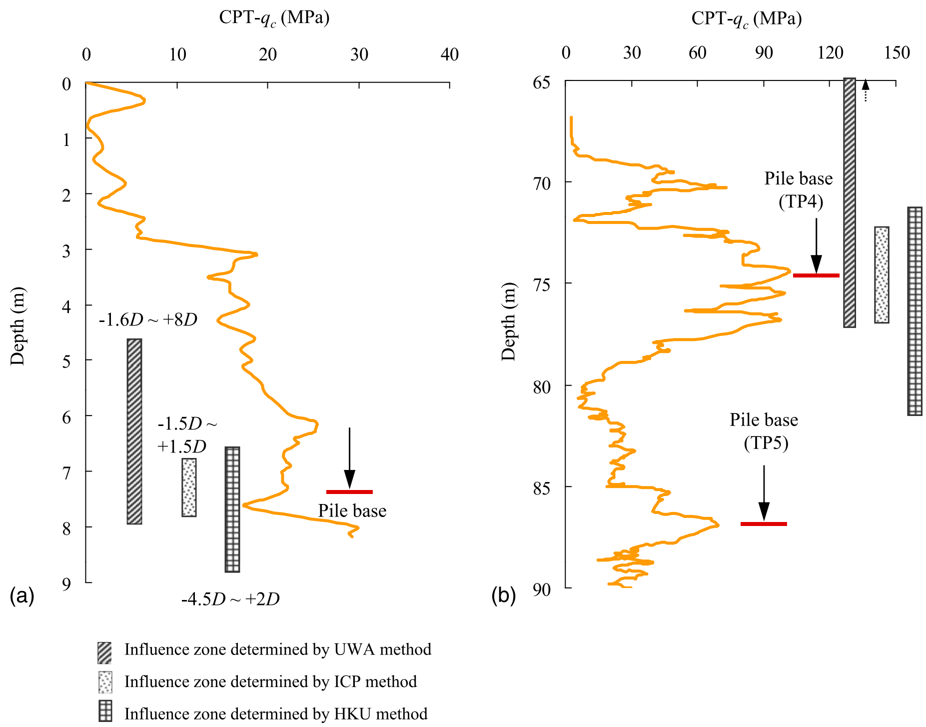

Fig. 7. Comparison of influence zones determined by various methods for (a) test pile of Paik et al. (2003) and (b) test piles of Kikuchi et al. (2007)

Table 5. Averaged CPT Tip Resistance ($q_{c,a}$) from Various Methods

Method	Jardine et al. (2005)	Paik et al. (2003)	Xu et al. (2008)					Kikuchi et al. (2007)	
			P1	P2	P4	P5	P6	TP4	TP5
ICP	66.40	21.66	10.80	11.58	10.86	11.72	9.90	84.40	47.90
UWA	53.10	17.91	10.56	11.30	10.53	11.30	9.58	57.20	36.00
HKU	59.52	22.55	11.15	11.13	11.15	11.12	11.15	43.03	37.00

Note: The unit of $q_{c,a}$ is MPa.

Table 6. Measured and Calculated Base Resistances

Reference	Test pile	Measured $q_{b,m}$	ICP method			UWA method			HKU method		
			$q_{b,c}$	$q_{b,c}/q_{b,m}$	$q_{b,m}/q_{b,c}$	$q_{b,c}$	$q_{b,c}/q_{b,m}$	$q_{b,m}/q_{b,c}$	$q_{b,c}$	$q_{b,c}/q_{b,m}$	$q_{b,m}/q_{b,c}$
Jardine et al. (2005)	—	10.72	11.33	1.06	0.95	14.44	1.35	0.74	12.41	1.16	0.86
Paik et al. (2003)	—	7.19	7.08	0.98	1.02	6.41	0.89	1.12	6.67	0.93	1.08
Xu et al. (2008)	P1	4.08	4.34	1.06	0.94	3.43	0.84	1.19	3.00	0.74	1.36
	P2	3.96	5.58	1.41	0.71	4.82	1.22	0.82	5.07	1.28	0.78
	P4	3.83	4.36	1.14	0.88	3.44	0.90	1.11	3.24	0.85	1.18
	P5	2.05	4.39	2.14	0.47	2.93	1.43	0.70	2.48	1.21	0.83
	P6	2.79	3.98	1.43	0.70	2.86	1.04	0.96	3.11	1.11	0.90
Kikuchi et al. (2007)	TP4	8.88	6.18	0.70	1.44	10.47	1.18	0.85	7.58	0.85	1.17
	TP5	6.37	3.51	0.55	1.81	6.59	1.03	0.97	6.52	1.02	0.98
		Mean	—	1.16	0.99	—	1.10	0.94	—	1.02	1.02
		COV	—	0.4	0.41	—	0.19	0.19	—	0.18	0.19

Note: $q_{b,c}$ = calculated unit base resistance (MPa) and $q_{b,m}$ = measured unit base resistance (MPa).

adjacent to the two piles are referred to in Fig. 7(b). The two piles have been used by Xu et al. (2008) in the evaluation of the UWA method compared with the ICP, Fugro, and NGI methods. Here, Fig. 12 compares the performance of the HKU method with the other four methods. It is evident that among all of them, the HKU method yields the best predictions for both piles.

For each test pile, the HKU method provides not only the estimate for the overall base capacity but also individual values of the annulus and plug resistance (see Table 7). In this respect, the new method has an attractive capability of elaborating the load transfer mechanism for the base capacity of open-ended piles.

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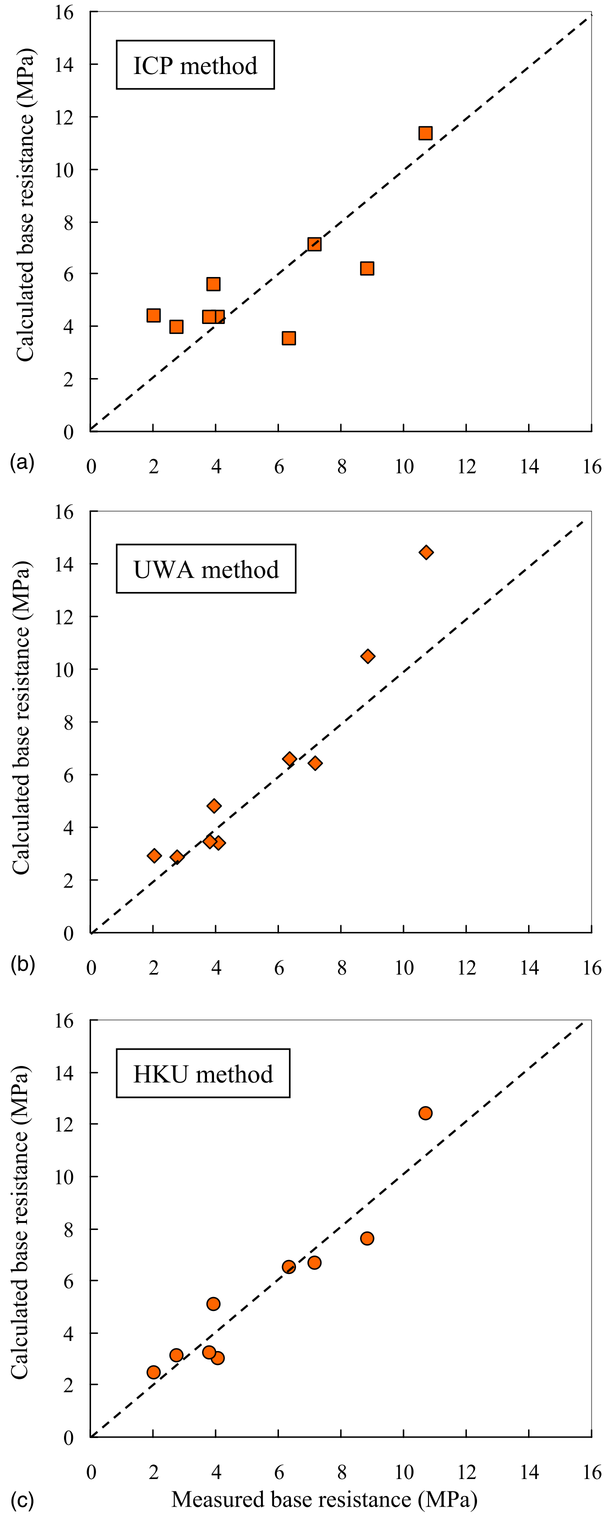

Fig. 8. Calculated versus measured base resistance: (a) ICP method; (b) UWA method; (c) HKU method

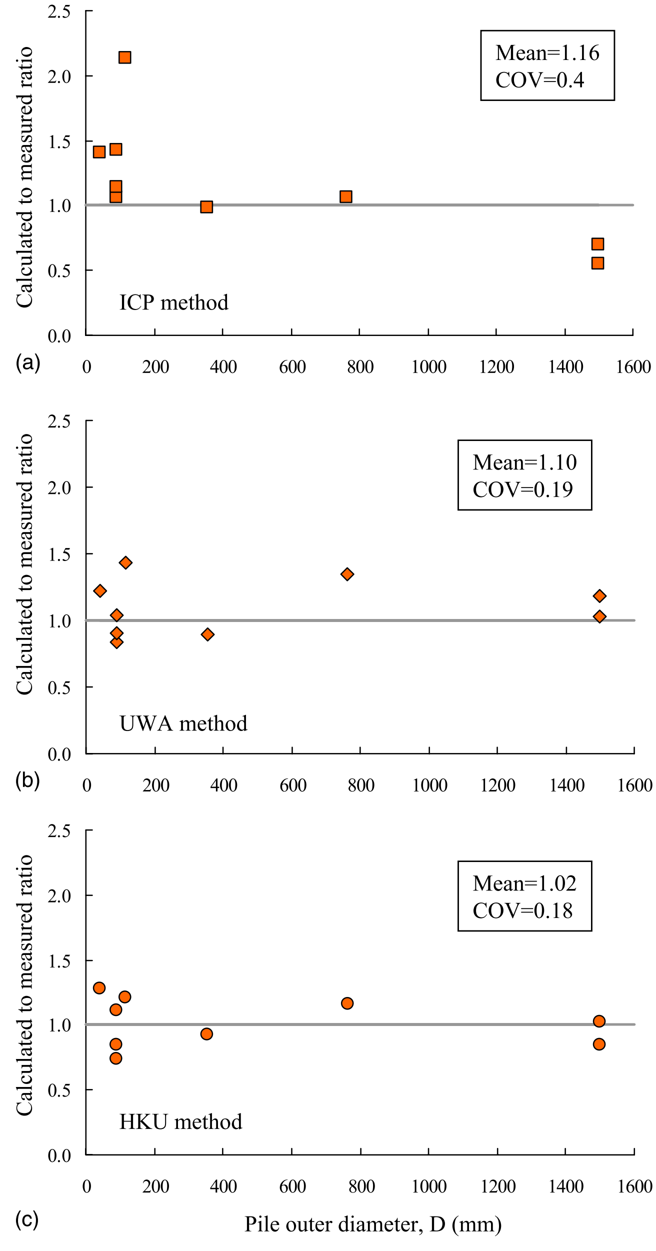

Fig. 9. Calculated-to-measured ratios of the base resistance as a function of pile outer diameter: (a) ICP method; (b) UWA method; (c) HKU method

Summary and Conclusions

This paper presents a new CPT-based approach, the HKU method, for estimating the base capacity of open-ended pipe piles in sand. The new method takes into consideration several important factors that have been largely ignored in current design methods, and offers both theoretical and practical advantages. These advantages are summarized as follows:

1. The HKU method decomposes the overall base capacity into the annulus resistance and the plug resistance from considerations of the mechanisms involved. The annulus resistance is properly linked with the ratio between the pile length and pile diameter, a key parameter reflecting the effect of pile embedment.
2. The HKU method accounts for the degree of soil plugging and its effect on plug resistance in a practical yet rational manner by incorporating the PLR at the end of pile installation into the calculation. Compared with the IFR, the PLR can be determined easily in practical applications.
3. The HKU method recommends a set of influence zones for averaging CPT tip resistance based on considerations of the effects of pile embedment, soil heterogeneity, and soil compressibility. In this respect, the method can produce more

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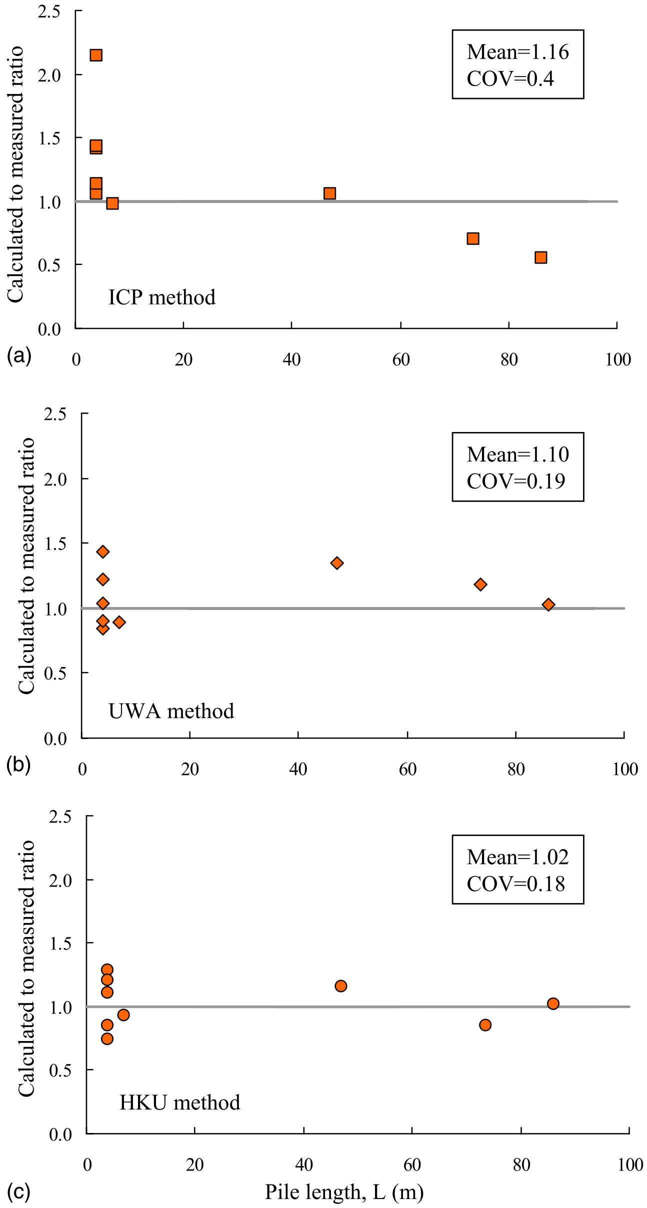

Fig. 10. Calculated-to-measured ratios of the base resistance as a function of pile length: (a) ICP method; (b) UWA method; (c) HKU method

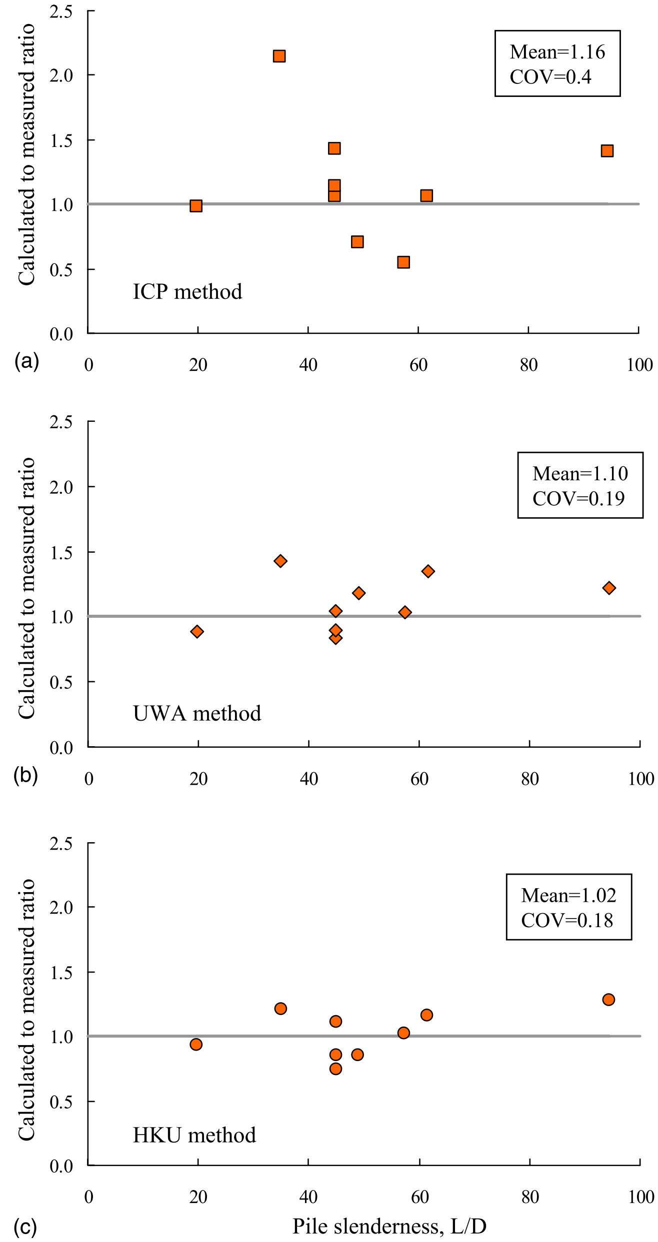

Fig. 11. Calculated-to-measured ratios of the base resistance as a function of pile slenderness: (a) ICP method; (b) UWA method; (c) HKU method

reliable estimates for a variety of site conditions in terms of safety and cost effectiveness.

Assessment of the proposed HKU method has been conducted against field-scale test piles and against the major design methods in current engineering practice. The assessment has consistently indicated that the HKU method is capable of producing satisfactory predictions over a wide range of pile lengths (L), pile diameters (D), and pile slenderness ratios (L/D). While several issues remain open to discussion and refinement, the HKU method offers increased rationality and accuracy and, hence, is a promising option in the design of open-ended pipe piles.

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Notation

The following symbols are used in this paper:

A = influence zone above pile base;
B = influence zone below pile base;
D = pile outer diameter;
D_r = relative density of sand;
d = pile inner diameter;
H = length of soil plug;
H_e = effective height of soil plug;
L = pile length;
M_A = geometric mean of CPT cone tip resistances over a range of A above pile base;
M_B = geometric mean of CPT cone tip resistances over a range of B below pile base;
Q_b = overall base capacity of pile;

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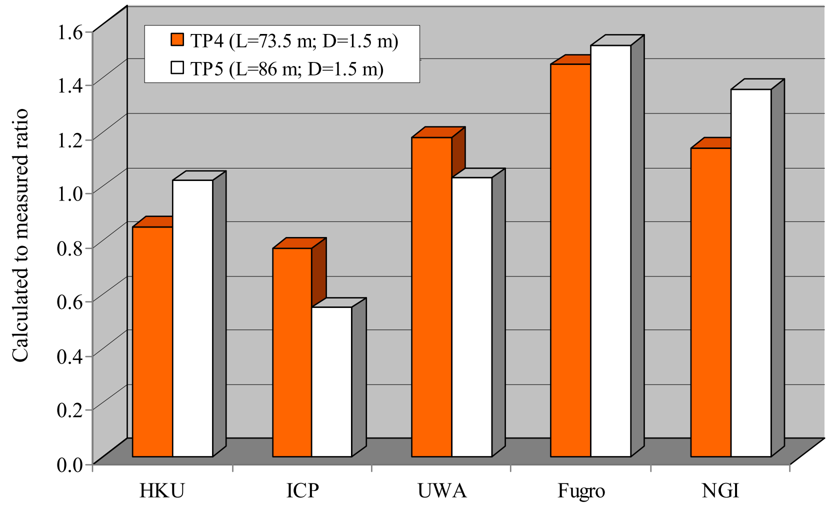

Fig. 12. Performance comparison of HKU method with the other four methods using instrumented large-diameter offshore piles

Table 7. Annulus and Plug Resistances Predicted by the HKU Method

Reference	Test pile	q_{ann} (MPa)	q_{plug} (MPa)
Jardine et al. (2005)	—	27.24	9.16
Paik et al. (2003)	—	10.37	4.88
Xu et al. (2008)	P1	5.13	2.73
	P2	5.12	5.05
	P4	5.13	2.73
	P5	5.12	2.16
	P6	5.13	2.78
Kikuchi et al. (2007)	TP4	19.79	6.62
	TP5	17.02	5.69

Note: The ICP and UWA methods do not offer predictions for the individual resistances.

q_{ann} = unit annulus resistance of pile;
q_b = unit base resistance of pile;
$q_{b,c}$ = calculated unit base resistance of pile;
$q_{b,m}$ = measured unit base resistance of pile;
q_c = CPT cone tip resistance;
$q_{c,a}$ = averaged CPT cone tip resistance; and
q_{plug} = unit plug resistance of pile.

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