Canadian Geotechnical Journal
 Revue canadienne de géotechnique

Statistics of Model Factors in Reliability-Based Design of Axially Loaded Driven Piles in Sand

Journal:	<i>Canadian Geotechnical Journal</i>
Manuscript ID	cgj-2017-0542.R1
Manuscript Type:	Article
Date Submitted by the Author:	22-Feb-2018
Complete List of Authors:	Tang, Chong; National University of Singapore, Phoon, Kok-Kwang; National University of Singapore, Department of Civil & Environmental Engineering
Is the invited manuscript for consideration in a Special Issue? :	N/A
Keyword:	Reliability-based design, Ultimate limit state, Serviceability limit state, Driven pile, Model factor

SCHOLARONE™
 Manuscripts

Statistics of Model Factors in Reliability-Based Design of Axially Loaded Driven Piles in Sand

Chong Tang¹,

Kok-Kwang Phoon²

¹Research fellow, Department of Civil and Environmental Engineering, National University of Singapore, Block E1A, #07-03, 1Engineering Drive 2, Singapore 117576, E-mail: ceetc@nus.edu.sg

²Professor, Department of Civil and Environmental Engineering, National University of Singapore, Block E1A, #07-03, 1Engineering Drive 2, Singapore 117576, E-mail: kkphoon@nus.edu.sg

Abstract: This paper compiles 162 reliable field load tests for axially loaded driven piles in sand from previous studies. The L_1 - L_2 method is adopted to interpret the measured resistance from the load-settlement data. The accuracy of resistance calculations with the ICP-05 and UWA-05 methods based on cone penetration test profile is evaluated by the ratio (bias or model factor) of the measured resistance to the calculated resistance. A hyperbolic model with two parameters, where the load component is normalized by the measured resistance, is utilized to fit the measured load-settlement curves. The means, coefficients of variation, and probability distributions for the resistance model factor and the hyperbolic parameters are established from the database. Copula theory is employed to characterize the correlation structure within the hyperbolic parameters. The statistical properties of the model factors are applied to calibrate the resistance factors in simplified reliability-based designs of closed-end piles driven into sand at the ultimate and serviceability limit state by Monte Carlo simulations. A simple example is provided to illustrate the application of the proposed resistance factors to estimate the allowable load for an allowable settlement at the desired serviceability limit probability.

Keywords: Reliability-based design, Ultimate limit state, Serviceability limit state, Driven pile, Model factor

Introduction

It has been recognized that most geotechnical designs are implemented with considerable uncertainties from resistances and applied loads. The Working or Allowable Stress Method (WSD or ASD) with a single factor of safety was previously adopted to account for these uncertainties. The limitations of ASD have been extensively discussed by Becker (1996) and Kulhawy and Phoon (2002). Following the lead of structural design practice, geotechnical design codes have been migrating towards reliability-based design (RBD) concepts worldwide. For example, section 6 on “Foundations and geotechnical systems” of the latest edition of Canadian Highway Bridge Design

Code (CHBDC) (Canadian Standards Association 2014) presented reliability calibrated resistance factors for the ultimate limit state (ULS, dealing with resistance) and serviceability limit state (SLS, dealing with settlement) (Fenton et al. 2016). Compared to ASD, RBD concepts can achieve a more consistent level of safety and a compatible reliability between superstructures and substructures. Phoon (2017) further discussed that reliability calculations play a useful complementary role in handling complex real-world information (multivariate correlated data), information imperfections (scarcity of information or incomplete information), and spatial variability that cannot be easily treated using deterministic methods.

The fourth edition of ISO 2394 “General Principles on Reliability for Structures” (International Organization for Standardization 2015) contains an informative Annex D “Reliability of Geotechnical Structures”. The emphasis in Annex D is to identify and characterize critical elements of geotechnical reliability-based design (RBD) process, while respecting the diversity of geotechnical engineering practice (Phoon et al. 2016). In contrast to structural materials (e.g. steel and concrete), naturally occurring geomaterials (e.g. soil and rock) are not manufactured to meet prescribed quality specifications and spatial variability is an inherent feature of a site profile. The most important element is the characterization of geotechnical variability. The key features of this element are: (1) coefficient of variation (COV) of a geotechnical design parameter and (2) multivariate nature of geotechnical data that can be exploited to reduce the COV, and (3) spatial variability affects the limit state beyond reduction in COV because of spatial averaging (Phoon et al. 2016). A detailed overview of the characterization of soil properties was presented in Ching (2017). Due to the simplifications, assumptions and approximations made in the respective design model, the second important element is the characterization of model uncertainty. It is usually carried out by taking the ratio of the measured result to the calculated result (International Organization for Standardization 2015), which is known as model factor in Annex D. A comprehensive summary of the statistics of model factors was given in Lesny (2017), which outlined the importance of model uncertainty in geotechnical RBD process. These elements are applicable to any implementations of RBD in a simplified form such as the Load and Resistance Factor Design (LRFD) or in a full probabilistic form (Phoon et al. 2016).

LRFD is the preferred RBD format in North America (Canadian Standard Association 2014 and AASHTO 2014), where the uncertainties in load and resistance are quantified separately and reasonably incorporated into the design process (Kulhawy and Phoon 2002). A suitable foundation design should satisfy both ULS and SLS. Ideally, the ULS and SLS should be checked using the same RBD principle. Nevertheless, the ULS still received most of the attention and more studies should be performed to develop reliability-based serviceability limit state design, as the SLS is often the governing criterion in foundation design (Becker 1996; Phoon and Kulhawy 2008; Wang and Kulhawy 2008; and Uzielli and Mayne 2011). At the ULS, a consistent load test interpretation criterion is used to produce the measured resistance and then, the resistance factor in LRFD is commonly calibrated from the statistics of the resistance model factor (AbdelSalam et al. 2012; Abu-Farsakh et al. 2009, 2013; Motamed et al. 2016; Ng and Fazia 2012; Ng et al. 2014; Paikowsky et al. 2004; Reddy and Stuedlein 2017a; Stuedlein et al. 2012; and Tang and Phoon 2018a, b). It is natural to follow the same approach for the SLS, where the ultimate resistance is replaced by an allowable resistance that depends on the allowable displacement (Zhang et al. 2008). The distribution of the SLS bias or model factor can be established from a load test database in the same way. The main limitation is that the SLS model factor has to be re-evaluated when a different allowable settlement is prescribed. In addition, the allowable settlement could also be random (Zhang and Ng 2005), which cannot be easily considered in the method of a single SLS model factor. In this regard, Phoon and Kulhawy (2008) presented an empirical and alternative way, which involves the use of a bivariate load-settlement model to fit the load-settlement data. The uncertainty in the entire load-displacement curve is represented by a bivariate random vector containing the bivariate load-settlement model factors as its components. Applications of this approach for RBD at the SLS can be found in Huffman and Stuedlein (2014), Huffman et al. (2015), Phoon and Kulhawy (2008), Reddy and Stuedlein (2017b), Stuedlein and Reddy (2013), Uzielli and Mayne (2011), and Wang and Kulhawy (2008).

The main objective of this paper is to propose simplified reliability-based designs of axially loaded driven piles in predominately granular soils at the ULS and SLS. First, a high-quality database with well-documented soil profiles and load test results is developed. An appropriate failure criterion is adopted to define the measured resistance from the load-settlement data and reliable methods are

chosen to calculate the axial resistance. Second, a bivariate load-settlement model is utilized to fit the measured load-settlement curves. The bivariate load-settlement model factors are determined from the least-squares regression of the load test data. Third, statistical properties (mean, COV, and probability distribution) of the resistance model factor and the bivariate load-settlement model factors are evaluated from the available data. Copula theory is employed to quantify the correlation structure within the bivariate load-settlement model factors. Fourth, the resistance factors in LRFD of driven piles at the ULS and SLS are calibrated by Monte Carlo simulations of the model factors. Finally, an example is presented to show the application of the calibrated resistance factors to estimate the allowable load for an allowable settlement at the prescribed serviceability limit probability.

Model Uncertainty Assessment

Resistance model factor

The model uncertainty at the ULS can be simply characterized as the ratio of the measured resistance to the calculated resistance (Eq. D.1 in Annex D)

$$M_u = R_{um}/R_{uc} \quad (1)$$

where R_{um} =measured resistance interpreted from the load test data using a certain criterion, R_{uc} =calculated resistance using the chosen design model, and M_u =model factor which represents the deviation of the predicted from the measured resistance. This approach is empirical, but it is practical and grounded on a load test database.

The model factor M_u is frequently termed as the resistance bias. The statistics has been recently incorporated into the calibration of the resistance factor in LRFD of foundations. Some examples can be found in Paikowsky et al. (2004), Abu-Farsakh et al. (2013), and Motamed et al. (2016) for drilled shafts; Stuedlein et al. (2012) and Reddy and Stuedlein (2017a) for augered cast-in-place (ACIP) piles; Paikowsky et al. (2004), AbdelSalam et al. (2012), and Tang and Phoon (2018a) for steel H-piles; Tang and Phoon (2018b) for torque-driven helical piles; and Paikowsky et al. (2004) and Abu-Farsakh (2009) for driven concrete or steel pipe piles. At present, model factors for foundation resistance are the most prevalent and the main challenge is to characterize model factors for other geotechnical systems (Phoon et al. 2016). Eq. (1) has been applied to quantify the model uncertainty for evaluating

slope stability (Travis et al. 2011 and Bahsan et al. 2014) and basal heave stability of wide excavations in clay (Wu et al. 2014) based upon limit equilibrium concepts, where the quantity is related to the factor of safety.

Bivariate load-settlement model

The following hyperbolic model with two curve-fitting parameters is adopted, which can provide a good representation of the load-settlement behaviour of pile foundations (Phoon et al. 2006, 2007; Phoon and Kulhawy 2008; Dithinde et al. 2011; Stuedlein and Reddy 2013; Reddy and Stuedlein 2017b; and Tang and Phoon 2018a, b):

$$\frac{Q}{R_{um}} = \frac{s}{a + bs} \quad (2)$$

where Q =applied load (kN); s =settlement (mm); a and b =hyperbolic parameters with reciprocals of “ a ” and “ b ” representing the initial slope and asymptotic value of the normalized hyperbolic curve. It was demonstrated that this approach can be easily used in conjunction with a random allowable settlement (Stuedlein and Reddy 2013, Huffman and Stuedlein 2014, Huffman et al. 2015, and Reddy and Stuedlein 2017b).

Statistics of the load-settlement model factors were reported in Uzielli and Mayne (2011), Huffman and Stuedlein (2014), Huffman et al. (2015), and Tang et al. (2017a) for spread footings; Stuedlein and Reddy (2013), and Reddy and Stuedlein (2017b) for ACIP piles; Dithinde et al. (2011) for drilled shafts; Tang and Phoon (2018a) for steel H-piles; Dithinde et al. (2011) for driven concrete or steel pipe piles; and Tang and Phoon (2018b) for torque-driven helical piles. Applications of the load-settlement model factors for RBD of foundations at the SLS were presented in Wang and Kulhawy (2008), Uzielli and Mayne (2011), Stuedlein and Reddy (2013), Huffman and Stuedlein (2014), Huffman et al. (2015, 2016), and Reddy and Stuedlein (2017b).

Database for Axially Loaded Driven Piles in Sand

ZJU-ICL database (Yang et al. 2015)

Yang et al. (2015) developed an extensive database ZJU-ICL for piles driven in predominately silica sands and described the quality filters to assemble the database with details for each data entry and examine the reliability of four advanced methods calculating the pile resistance. The ZJU-ICL

database comprises of 52 sites with 75 compression and 41 uplift load tests. Among these data, 54 load tests were compiled from the ICP-05 set, 14 load tests in the UWA-05 set, 12 load tests in the Deep Foundation Load Test Database (DFLTD) maintained by the Federal Highway Administration (FHWA), and additional 36 load tests available in literature. The following information is provided in the ZJU-ICL database:

- (1) Site investigation: test site location, a complete cone penetration test (CPT) profile, general soil description, water tables, interface shearing angles, and sand grain size distribution.
- (2) Foundation information: driving records (method and pile age after driving), pile width or diameter, wall thickness, embedment depth, pile tip end conditions (open or closed), pile shape (square, circular or octagonal), and pile material (concrete or steel).
- (3) Load test data: applied load direction (axial compression or uplift) and measured load-settlement curves.

FHWA DFLTD

Since the 1980s, FHWA began the collection of pile load test data with subsurface information and developed DFLTD, which is the most comprehensive database for deep foundations in the United States. It is intended to be used as a centralized data repository of soil and load test information by States, universities, consultants, contractors, and other agencies with the principal goal of optimizing the design, construction, and maintenance of bridge foundations and other high infrastructure as well as other geotechnical design activities. In total, the first version of FHWA DFLTD contains over 2,500 soil tests and over 1,500 load tests on a wide range of pile types such as driven concrete (square, circular, and octagonal) or steel (open- and closed-end), and drilled shafts from nearly 850 sites. In 2014, FHWA initiated a study to evaluate the bearing resistance of Large-Diameter Open-End Piles (LDOEPs) and 155 additional axial load tests on LDOEPs were documented in the second version of FHWA DFLTD. A brief introduction of FHWA DFLTD was given by Abu-Hejleh et al. (2015). As the number of load tests in the ZJU-ICL database is relatively limited, another 116 static load tests (quick procedure) for driven piles in sand are compiled from FHWA DFLTD to give a more precise characterization of the bivariate load-settlement model factors a and b in Eq. (2).

Flynn (2014) (driven cast-in-situ piles)

Flynn (2014) summarized 90 maintained compression load tests on temporary-cased driven cast-in-place (DCIS) piles in layered or uniform sandy deposits at a number of sites in the United Kingdom, where majority of load tests were collected from Keller Foundations files. Unlike preformed displacement piles, less attention was paid to DCIS piles. This may be due in part to the differences between installation processes for DCIS and traditional displacement piles. The primary construction processes of a DCIS pile is described as follows (Flynn and McCabe 2016): (1) place a hollow steel tube with a sacrificial circular steel plate at the base of the tube preventing soil and/or water from entering it; (2) place the reinforcement and pour concrete into the tube, when the tube reaches the required depth; and (3) extract the tube from the soil with the steel plate remaining at the base when concreting is complete.

Interpretation Load Test Results

Determination of measured resistances

Measured load-settlement curves are presented in Fig. 1 for driven piles with closed-end and in Fig. 2 for driven piles with open-end. It can be seen that most of measured load-settlement curves do not show a clear peak or asymptote, i.e. failure is difficult to be identified and not all failure criterion lead to consistent results (Lesny 2017). The measured resistance needs to be interpreted from the load-settlement data using an appropriate criterion. Different approaches are available for determining the ultimate pile resistance, which are based on one or more of the following techniques: (1) settlement limitation, (2) graphical construction, and (3) mathematical modeling.

Marcos et al. (2013) utilized 152 field compression load tests to evaluate eight methods interpreting the ultimate resistance of driven piles. The results demonstrated that the L_1 - L_2 method proposed by Hirany and Kulhawy (1989a, b) can produce a more reasonable interpretation of axial load tests on driven piles. The L_1 - L_2 method is employed to determine the measured resistance of axially loaded driven piles in sand. This approach was proposed from the observation that the load-settlement curve can generally be simplified into three distinct regions: initial linear, nonlinear curve transition, and final linear. Point L_1 (elastic limit) defines the load and displacement at the end of the initial linear region, while point L_2 corresponds to the load and displacement at the initiation of the

final linear region. Beyond the point L_2 , a small increase in load gives a significant increase in displacement. It is therefore appropriate to interpret the load at the point L_2 as the measured resistance (Hirany and Kulhawy 1989a, b).

Classification of load tests

In terms of applied load direction, the collected data is categorized into two broad types, namely, axial compression and uplift. In each load test type, the tested piles are subdivided into two types, i.e., large- and small-displacement pile. Displacement piles cause the soil to be displaced radially as the pile shaft is driven or jacked into the ground. With non-displacement piles (or replacement piles), soil is removed and the resulting hole filled with concrete or a precast concrete pile is dropped into the hole and grouted in. According to Tomlinson and Woodward (2008), large displacement piles comprise solid-section piles or hollow-section piles with a closed-end. DCIS piles come into this category (Tomlinson and Woodward 2008 and Flynn 2014). Small displacement piles are driven or jacked into the ground with relatively small cross-sectional area, including steel H-piles and driven open-end piles (Tomlinson and Woodward 2008). When these piles plug with soil during driving, they become large-displacement piles. The pile material includes concrete and steel.

Assessment of data quality

Since not each load test can produce dependable results, it is important to assess the data quality, which could significantly affect the evaluation of model factors as well as the calibration of resistance factors in LRFD. The classification system proposed by Roling et al. (2011) to assess the database PILOT maintained by the Iowa Department of Transportation is utilized here to catalog driven pile load tests as “reliable” and “usable”.

Due to the costs and time consumption, pile load tests are usually performed only as proof tests, where the maximum applied load is generally equivalent to 1.5 times the design load. Many examples can be found in the load tests on DCIS piles documented by Flynn (2014). In this case, the load test will not exhibit sufficient settlement which would correspond to the ultimate load criterion. In this case, extrapolation techniques may be considered (Paikowsky and Tolosko 1999). However, NeSmith and Siegel (2009) stated that there still exists a reluctance, and even an opposition to extrapolate the load test results in the absence of established guidelines for extrapolation. Paikowsky and Tolosko

(1999) analyzed 63 pile load tests with three extrapolation methods. They showed that the over-prediction of the ultimate pile resistance based on extrapolation could be as high as 50%. Hence, the extrapolation of pile load test was not recommended for model calibration (Lesny 2017).

The first tier assigns the reliable classification to a pile load test, where the measured resistance can be interpreted directly from the load-settlement curve using the L_1 - L_2 method. Among the collected 322 static load tests, 162 (134 for axial compression and 28 for axial uplift) are identified as reliable, which will be used to determine the measured resistance R_{um} and evaluate the bivariate load-settlement model factors a and b . The test site location, information for tested piles [e.g. type (closed- or open-end), dimension (pile diameter or width B and embedment depth D), material (concrete or steel), and shape (square, circular, or octagonal)], the measured resistance R_{um} and the hyperbolic parameters a and b are given in Appendix A1. The case number in FHWA DFLTD, ZJU-ICL and Flynn (2014) is retained for ease of reference.

The second tier assigns the usable classification, which identifies those pile load tests with sufficient CPT information to calculate the pile resistance, to a load test. Out of 162 reliable load tests, 96 are classified as useable to characterize the model factor M_u . Based on the classification of data, 134 axial compression tests are subdivided as follows: (1) driven piles with open-end (6 load tests for concrete piles and 11 load tests for steel pipe piles) and (2) driven piles with closed-end (78 load tests for concrete piles and 39 load tests for steel pipe piles). 28 axial uplift tests are categorized into: (1) driven piles with open-end (19 load tests for steel pipe piles) and (2) driven piles with closed-end (5 load tests for steel pipe piles and 4 load tests for concrete piles).

One needs to keep in mind that the main problem of using databases to characterize model uncertainties is the limited number of tests coming from very different sources with each covering only a limited range of possible design situations (Lesny 2017). The ranges of pile geometries B and D/B in the database are summarized in Table 2 to show the calibration domain.

Normalized load-settlement curves

The original load-settlement curves illustrated in Fig. 1(a, c) and Fig. 2 (a, c) vary in a wide range, because of different pile geometries (pile diameters and embedment lengths) and surrounding soil properties. However, when the load Q is divided by the measured resistance R_{um} , these normalized

load-settlement curves appear to vary in a narrower range as shown in Fig. 1 (b, d) and Fig. 2 (b, d). This is favorable for the characterization of the bivariate load-settlement model factors. Similar results were also observed by Phoon and Kulhawy (2008), Ching and Chen (2010), Dithinde et al. (2011), Huffman et al. (2016), Tang et al. (2017a), and Tang and Phoon (2018a, b). It could be explained as that the effects of pile geometries and surrounding soil profiles on the load-settlement behaviour are lumped within the measured resistance.

Predicted Resistance of Axially Loaded Driven Piles

CPT-Based Design Methods

Different static analyses methods have been developed to predict the pile resistance. As opposed to dynamic formulae, these methods are based on the classical soil mechanics theory. In this context, the axial resistance of a driven pile is generally calculated as the sum of the shaft resistance (R_s) and base resistance (R_b), which is given below

$$R_{uc} = R_b + R_s = A_b q_b + \pi B \int \tau_f dz \quad (3)$$

where A_b =the area of the pile tip, q_b =unit base resistance, B =shaft diameter, and τ_f =local shear stress at failure. The value of q_b is specified as that mobilized at a pile tip settlement of $10\%B$. For the case of axial uplift loading, the base resistance is usually considered to be negligible.

Eq. (3) assumes that the pile tip and pile shaft have moved sufficiently with respect to the adjacent soil to simultaneously develop the base and shaft resistance. Although such an assumption is not completely consistent with the reality, where the displacement to mobilize the shaft resistance is smaller than that for base resistance, it is widely applied for all piles except LDOEPs with diameters of 914 mm (36 inches) or greater (Hannigan et al. 2016). The NCHRP Synthesis 478 (Brown and Thompson 2015) stated that LDOEPs present a unique challenge for foundation designers owing to the combination of several factors: (1) uncertainty of “plug” behaviour during installation, (2) potential for installation difficulties and pile damage, (3) axial resistance from internal friction, and (4) verification of large nominal axial resistance is more challenging and expensive. Therefore, LDOEPs are beyond the scope of this article.

Because of the complex stress-strain history of the soil in which the pile is founded (Randolph 2003), it is a challenging task for designers to calculate the axial resistance of a driven pile in sand. The conventional methods expressed the shaft resistance as a product of the coefficient of lateral earth pressure (K) and the vertical effective stress (σ_v') (Hannigan et al. 2016). The difficulty in applying these methods is to estimate K , which strongly depends on the level of soil displacement during installation and the in-situ soil state. For simplicity, K was generally assumed to be constant, implying a linear relation between the local shear stress τ_f and the vertical effective stress σ_v' . This was contrast with observations of shaft resistance during loading of a driven pile in the field (Randolph et al. 1994). Because of the over-simplification, previous studies presented the poor reliability of the design methods based on lateral earth pressure theory (Toolan et al. 1990 and Schneider et al. 2008).

During the past 25 years, high-quality test results (Lehane 1992 and Chow 1997) illustrated the mechanical behaviour of piles driven in sand and identified several influential factors on pile behaviour including (1) the extent of soil displacement during installation and loading, (2) the friction fatigue (i.e. the reduction in shaft friction due to increasing load cycles during installation), (3) increasing radial stresses because of dilation at the pile-soil interface, (4) loading type (compression or uplift), and (5) pile ageing (i.e. the increase in shaft resistance with time). Recently, four advanced design methods based on CPT profiles were developed to consider these influential factors. These methods correlate directly q_b and τ_f with CPT profiles, avoiding the intermediate estimation of soil properties.

Yang et al. (2017) employed 117 high-quality load tests to assess the reliability of the four CPT-based design methods. The ICP-05 method (Jardine et al. 2005) and the UWA-05 method (Lehane et al. 2005) were found to have significant advantages in eliminating potential biases in resistance predictions, which are utilized to calculate the axial resistance of driven piles in sand. The details are summarized in Table 1.

Comparison between measured and calculated resistances

Despite the differences in the construction sequence as described earlier, Flynn et al. (2014) and Flynn and McCabe (2016) showed that the shaft and base resistance as well as the ultimate resistances of a DCIS pile are similar to a preformed closed-end displacement pile with equivalent dimensions. Thus,

the ICP-05 and UWA-05 methods in Table 1 are also applicable to estimate the axial resistance of DCIS piles (Flynn 2014).

The calculated resistances (R_{uc}) for 96 usable load tests are presented in Appendix A1. Fig. 3 shows that the ICP-05 and UWA-05 provide similar predictions of the axial resistance. Comparison between the measured resistance and the calculated resistance from the ICP-05 method is given in Fig. 4. A reasonable agreement is observed, where the mean trend line is close to the 45° trend line. The arithmetic means of the model factor M_u are 1.09 and 1.25 for axial compression and uplift, respectively. The results suggest that the ICP-05 method slightly under-estimates the axial resistance (Flynn 2014 and Yang et al. 2017). The standard deviation of M_u is around 0.3, implying a moderate model uncertainty. This is because the ICP-05 and UWA-05 methods are empirical in nature, where most quantities in Table 1 were derived from a set of field load tests. Moreover, Lesny (2017) discussed that the model uncertainty expressed by Eq. (1) cannot be separated from the inherent variability of soil profiles and measurement errors. Although static load tests are considered as the most definite way of assessing pile resistance, they are not free of uncertainties. As the load measurement is done directly, the procedure used for the test (maintained load test, maintained rate of penetration, or creep test), measurement technique and the interpretation introduce some degree of uncertainty.

Statistical Analyses of Model Factors

Note that the sample size for piles with open-end under axial compression ($N=17$) and piles under axial uplift ($N=19$ for open-end and $N=9$ for closed-end) is small, which could be insufficient for model uncertainty assessment, especially for hyperbolic parameters. As a result, only the case of driven piles with closed-end under axial compression (78 for concrete piles and 39 for steel pipe piles) is investigated subsequently. A set of observations on the model factors (M_u , a , and b) are obtained from the database, which take on a range of values. It is natural to consider the model factors as random variables. To evaluate the statistical properties of the model factors, the following procedures are used: (1) detection of data outliers, (2) verification of randomness, (3) calculation of sample statistics (mean and COV), and (4) identification of probability distributions.

Detection of data outliers

Data outliers are extreme values that deviate significantly from the main trend of a data set. The presence of outliers could lead to a biased evaluation of model factors. As recommended by Dithinde et al. (2011), the detection of data outliers can be performed with the aid of (i) scatter plots of measured resistance versus calculated resistance (see Fig. 3) and (2) normalized load-settlement curves [see Fig. 1 (b, d) and Fig. 2 (b, d)]. Visual inspection from these plots indicates there are no potential outliers.

Verification of randomness

In practice, the ratio of the measured over predicted result could be systematically affected by input parameters. It is incorrect to treat the model factor as a random variable directly in this situation (Phoon et al. 2016). Examples are given in Reddy and Stuedlein (2017b), Stuedlein and Reddy (2013), Tang and Phoon (2017), Tang et al. (2017a, b), and Zhang et al. (2015). In these studies, a function of the influential parameters was introduced to represent the statistical dependency of model factor. The COV of the transformed model factor can be decreased considerably. This is similar to employ the correlation structure within multivariate geotechnical data to reduce the COV of a design parameter (Ching 2017).

Figs. 5-7 present scatter diagrams of the resistance bias M_u and the bivariate load-settlement model factors b and a against pile slenderness ratio D/B , pile diameter B , and relative density D_r . These model factors appear to be randomly distributed over the ranges of the parameters. Moreover, the dependency of model factors on the parameters (D/B , B , and D_r) can be partially checked using Spearman rank correlation analyses with the r - and p -values. If p is smaller than 0.05, the correlation r is significantly different from zero, implying statistical dependency of model factors on the respective parameter. The results are summarized in Appendix A2. For the model factor M_u , all p -values are greater than 0.05, implying the correlations are statistically insignificant. Because of this point, M_u can be viewed as a random variable. For the hyperbolic parameters a and b , most p values are larger than 0.05 except for the p -values for pile slenderness ratio D/B . Nevertheless, the r -values ($r=-0.23$ for b and $r=0.2$ for a) suggest a low degree of correlation (Dithinde et al. 2016). It is reasonable to ignore it and treat the hyperbolic parameters a and b as random variables directly.

The resistance model statistics are summarized in Table 3. The mean and COV values of M_u are 1.1 and 0.31 for the ICP-05 method and 1 and 0.39 for the UWA-05 method. The results are different from those obtained by Yang et al. (2017), where (1) the measured resistance was interpreted as the load at a settlement of $10\%B$ and (2) extrapolation was used for load tests with settlement smaller than $10\%B$. The ICP-05 and UWA-05 methods generally produce more accurate estimation of pile resistance on average with smaller COV values than the conventional design methods with lateral earth pressure theory (e.g. Nordlund method or β -method) as calibrated by Paikowsky et al. (2004) (see Table 3). This can be explained as that the ICP-05 and UWA-05 methods were built on a good understanding of the mechanical behaviour of driven piles in sand and calibrated against high-quality field load tests. The factors that have an important influence on the pile behaviour are taken into account appropriately. The statistics of the resistance model factor or bias for driven piles in sand are comparable with the results (mean=1.11 and COV=0.33) of Dithinde et al. (2011) in which the coefficient K of the lateral earth pressure in the static design formula was re-evaluated to fit the load test data well.

The statistics for the bivariate load-settlement model factors a and b are given in Table 4. The mean and COV values of the parameter a are 6.26 and 0.75 (high variation) which is larger than the result (COV=0.54) of Dithinde et al. (2011). The difference could be due to that load tests used in the current work are collected from a wide range of site conditions. The mean and COV values of the parameter b are 0.8 and 0.15 (low variation), which are very similar to the results (mean=0.71 and COV=14) of Dithinde et al. (2011). The parameter a exhibits a significant higher variation than the parameter b . This can be understood from the physical meanings of a and b that uncertainty in soil stiffness parameter is higher than the uncertainty in strength parameter (Phoon and Kulhawy 1999).

Identification of probability distributions

The probability distribution of the observed model factors can be identified according to the goodness-of-fit (GOF) tests. These tests measure the compatibility of a sample with a theoretical probability distribution function. EasyfitTM supports three types of GOF tests, namely, Kolmogorov-Smirnov, Anderson-Darling, and Chi-square. The Kolmogorov-Smirnov (KS) test is employed and implemented by a statistical software EasyfitTM. The KS test results indicate that the observed model

factors M_u , b , and a can be described as Lognormal, Generalized extreme value, and Lognormal distribution, respectively. The cumulative distribution functions of the model factors (M_u , a , and b) are presented in Figs. 8-10. The plots suggest that the selected theoretical distributions provide reasonable representation of the distributions of these model factors.

Hyperbolic parameters simulation

Scatter plots of the hyperbolic parameters a and b are plotted in Fig. 11. It shows the hyperbolic parameters a and b are inversely correlated. The negative correlation between a and b is characterized using Kendall's tau coefficients of $\rho_\tau = -0.56$. Similar results were also observed by Phoon and Kulhawy (2008), Dithinde et al. (2011), Huffman and Stuedlein (2014), Reddy and Stuedlein (2017b), and Tang and Phoon (2018a, b), etc. It can be explained as follows: a small initial slope of the load-settlement curve (i.e. a large a value) implies a slowly decaying curve, and is generally associated with a less well-defined and larger asymptote (i.e. a small b value). This has been discussed in Stuedlein and Reddy (2013) for ACIP piles in granular soils.

To avoid potential bias in the reliability calculations, the correlation within the hyperbolic parameters a and b should be considered reasonably. In general, there are two ways to simulate the correlated hyperbolic parameters: (1) the translation-based probability model with less robust Pearson product-moment correlation used by Phoon and Kulhawy (2008), Dithinde et al. (2011), and Stuedlein and Reddy (2013), and (2) copula theory adopted by Huffman and Stuedlein (2014), Huffman et al. (2015), Reddy and Stuedlein (2017b), and Tang and Phoon (2018a, b). Ching et al. (2016) suggested that the Pearson correlation is the least robust, suffering the most significant uncertainty. In addition, the translation model is unsuitable for non-linear correlations as observed in soil cohesion and friction angle (Li et al. 2013) or the hyperbolic parameters (Huffman and Stuedlein 2014). Copula theory is thus utilized to simulate the negatively correlated hyperbolic parameters.

The lowest values for Akaike information criterion (AIC) (Akaike 1974) or Bayesian information criterion (BIC) (Schwarz 1978) in Appendix A3 suggest that the best-fit copula to model the correlation structure within a and b is the Gaussian-type copula. The 1000 simulated hyperbolic parameters a and b are displayed in Fig. 11. It shows the selected copula qualitatively capture the scatter associated with the observed values. The simulated hyperbolic parameters are applied to Eq. (2)

producing the simulated normalized curves, which are presented by black lines in in Fig. 12. It can be seen that the simulated curves resemble the measured data, red lines in Fig. 12. These results indicate that the established probability models for a and b satisfactorily represent the observed behaviors of driven piles with closed-end in sand.

LRFD Calibration

In this section, simplified RBD procedures for the ULS and SLS are presented and the statistical properties of the model factors (M_u , a , and b) are incorporated into the procedures to calibrate the resistance factors using Monte Carlo-based reliability simulations.

RBD design models

The limit state in a foundation design problem can be simply defined as that in which the resistance is equal to the applied load. The foundation will fail if the resistance is less than this applied load. Otherwise, the foundation performs satisfactorily. These three situations can be described concisely by a single performance function as follows (Phoon and Kulhawy 2008)

$$p_f = \Pr(R - Q \leq 0) \leq p_{ft} \quad (4)$$

where p_f =probability of failure, R =ultimate or allowable resistance, Q =applied load, and p_{ft} =target probability of failure.

The ULS is defined when the applied load is greater than or equal to the ultimate resistance. Considering the combination of dead load and live load for AASHTO Strength Limit I, the performance equation is given below (AASHTO 2014)

$$\psi_R R_n = \gamma_{DL} Q_{DL} + \gamma_{LL} Q_{LL} \quad (5)$$

where ψ_R =resistance factor, R_n =calculated nominal resistance, Q_{DL} =dead load, γ_{DL} =dead load factor, Q_{LL} =live load, and γ_{LL} =live load factor. According to Abu-Farsakh et al. (2009, 2013), Eq. (4) is rewritten as follows

$$p_f = \Pr \left[\left(\frac{\gamma_{DL} + \gamma_{LL}/\eta}{\psi_R} \right) M_u - (\lambda_{DL} + \lambda_{LL}/\eta) \leq 0 \right] \leq p_{ft} \quad (6)$$

where η =ratio of the dead to live load= Q_{DL}/Q_{LL} , λ_{DL} =bias of the dead load, and λ_{LL} =bias of the live load.

The SLS is reached when foundation displacement is equal to or greater than a prescribed allowable value. In terms of resistance, the SLS can be defined as the case when the applied load Q is greater than or equal to the allowable value Q_a . Eq. Based on Eq. (2), the allowable load (Q_a) is approximated by

$$Q_a = M_s R_{um} \quad (7)$$

where s_a =allowable settlement and M_s =SLS model factor defined by

$$M_s = \frac{s_a}{a + bs_a} \quad (8)$$

The probability of failure (p_f) exceeding the SLS is expressed as (Phoon and Kulhawy 2008, Uzielli and Mayne 2011, Stuedlein and Reddy 2013, and Reddy and Stuedlein 2017b)

$$p_f = \Pr(Q_a - Q \leq 0) \leq p_{fr} \quad (9)$$

Substituting Eq. (8) into Eq. (9) results in the following estimation of probability of failure at the SLS (Uzielli and Mayne 2011, Stuedlein and Uzielli 2014, Huffman et al. 2015, and Reddy and Stuedlein 2017b)

$$p_f = \Pr\left(\frac{s_a}{a + bs_a} \leq \frac{1}{\psi_q} \frac{Q'}{M_u}\right) \leq p_{fr} \quad (10)$$

where $Q = Q'Q_n$ with Q_n =nominal applied load, and Q' =normalized random variable; ψ_q =a lumped load-resistance factor= R_{uc}/Q_n , R_{uc} =calculated resistance; M_u and M_s =ULS and SLS model factors which have been defined earlier.

ULS resistance factor

As recommended by AASHTO (2014), $\gamma_{DL}=1.25$ and $\gamma_{LL}=1.75$, the bias λ_{DL} for the dead load is assumed to be a lognormal random variable with mean of 1.05 and COV of 0.1, and the bias λ_{LL} for the live load is a lognormal random variable with mean of 1.15 and COV of 0.2. The steps of Monte Carlo simulations to calibrate the resistance factor (ψ_R) are summarized as follows (Abu-Farsakh et al. 2009 and 2013):

- (1) Select a trial resistance factor and generate random numbers for the resistance model factor

M_u , the bias factors λ_{DL} and λ_{LL} in the ULS performance function $g(R, Q)$ defined by Eq. (6).

- (2) Find the number N_f of cases where $g(R, Q)$ is smaller than or equal to zero. The probability of failure is given by $p_f = N_f/N_s$ (N_s =total number of Monte Carlo simulations=50, 000 here) and the reliability index is estimated as $\beta = -\Phi^{-1}(p_f)$, where Φ^{-1} =inverse standard normal cumulative function.
- (3) Repeat steps (1)-(2) until $|\beta - \beta_T| < \text{tolerance}$ (0.01 in this study), where β_T =the target reliability index. As recommended by Paikowsky et al. (2004), $\beta_T=2.33$ for redundant piles defined as 5 or more piles per pile cap, and $\beta_T=3$ for non-redundant piles defined as 4 or fewer piles per pile cap.

The calibrated resistance factors ψ_R for the ICP-05 and UWA-05 methods with $\eta = Q_{DL}/Q_{LL} = 1 \sim 10$ for $\beta_T=2.33$ (i.e. $p_f=1\%$) and $\beta_T=3$ (i.e. $p_f=0.1\%$) are given in Table 5. Fig. 13 shows the target reliability index (β_T) has more significant effect on the resistance factor ψ_R than η . For instance, ψ_R for $\beta_T=2.33$ decreases by 8% ($\psi_R=0.61$ for $\eta=1$ and $\psi_R=0.56$ for $\eta=5$), while ψ_R for $\eta=3$ reduces by 21% ($\psi_R=0.57$ for $\beta_T=2.33$ and $\psi_R=0.45$ for $\beta_T=3$). The resistance factor ψ_R almost becomes constant as $\eta \geq 5$. Similar results for the resistance factor in LRFD of pile foundations have been reported in literature (Paikowsky et al. 2004, Abu-Farsakh et al. 2009, and AbdelSalam et al. 2012). Since the COV of M_u for the UWA-05 method is higher, namely, COV=0.39 compared to 0.31 of the ICP-05 method, smaller resistance factors ψ_R are obtained, as shown in Table 5.

Paikowsky et al. (2004) pointed out that only the resistance factor does not provide an evaluation regarding the effectiveness of the pile resistance prediction methods. Such efficiency can be evaluated through the ratio of the resistance factor ψ_R to the model (or bias) factor M_u , i.e., ψ_R/M_u . A higher ψ_R/M_u value for a design method which can estimate the pile resistance more accurately regardless of the bias corresponds to a more economical design (Paikowsky et al. 2004 and AbdelSalam et al. 2012). The ψ_R/M_u values are also given in Table 5.

Lumped load-resistance factor for SLS

As adopted by Reddy and Stuedlein (2017b), the mean of the allowable settlement (μ_{sa}) varies from 2.5 mm to 50 mm, while the COV value has not been well characterized yet. According to the observations of Zhang and Ng (2005), Phoon and Kulhawy (2008) and Uzielli and Mayne (2011) considered a COV value up to 60%. In the present work, the allowable settlement (s_a) is treated as a

lognormal variable with $\mu_{sa}=2.5\sim 50$ mm and $COV=0, 0.2, 0.4$, and 0.6 . To be consistent with the national LRFD specifications (AASHTO 2014), the normalized random variable (Q') for the applied load (Q) is assumed to follow the lognormal distribution with mean of 1 and COV of 0.1 and 0.2 for dead and live loads, respectively.

For each lumped load-resistance factor ψ_q , 5×10^6 simulations are implemented to compute the failure of probability p_f and reliability index β in which each random variable including M_u for the ICP-05 method, a , b , s_a , and Q' is randomly sampled from their source distributions. The results of the reliability simulations are presented in Fig. 14 for $COV_Q=0.1$. The reliability index β increases as the mean value of s_a increases and the COV value of s_a decreases. For a given allowable settlement s_a , a linear relation exists between β and $\ln\psi_q$. These results are similar to those reported in Huffman and Stuedlein (2014). Hence, β can be expressed as the following logarithmic function of ψ_q as suggested by Uzielli and Mayne (2011)

$$\beta = p_1 \ln\psi_q + p_2 \quad (11)$$

where p_1 and p_2 =best-fit coefficients. Eq. (11) has been applied to reliability-based design of spread footings on aggregate pier reinforced clay at the SLS by Huffman and Stuedlein (2014).

Fig. 15 shows that the best-fit coefficients p_1 and p_2 can be simply approximated by a logarithmic function of the mean value of s_a . Similar results were given in Uzielli and Mayne (2011) and Huffman and Stuedlein (2014). Introducing the logarithmic models of p_1 and p_2 in Eq. (11) leads to the following expression of the reliability index β

$$\beta = (a_1 \ln s_a + a_2) \ln\psi_q + a_3 \ln s_a + a_4 \quad (12)$$

where a_1 and a_2 =best-fit coefficients for p_1 and a_3 and a_4 =best-fit coefficients for p_2 . Table 6 summarizes the best-fit coefficients a_1 , a_2 , a_3 , and a_4 to estimate β for given values of s_a and ψ_q , which are applicable for $\beta > 0$, $s_a = 5 \sim 50$ mm and $\psi_q = 1 \sim 10$. With Eq. (12), the lumped load-resistance factor ψ_q for given β and s_a values could be obtained as follows

$$\psi_q = \exp\left(\frac{\beta - a_3 \ln s_a - a_4}{a_1 \ln s_a + a_2}\right) \quad (13)$$

Application of SLS Resistance Factors

To show the use of the proposed RBD at the SLS, a design scenario for a driven steel pile P1 in ZJU-ICL database with $B=0.61$ m and $D=45$ m is presented. The nominal allowable settlement s_a is assumed to be 25 mm with $COV_{sa}=20\%$. The COV of the applied load Q' is 10%. The procedure for estimating the allowable load with $p_{fr}=1\%$ ($\beta=2.33$) exceeding the SLS is summarized below

- (1) Calculate the nominal pile resistance R_{uc} using the ICP-05 method given in Table 1. For this example, $R_{uc}=3885$ kN.
- (2) Determine the load-resistance factor ψ_q using Eq. (13) with $\beta=2.33$ and coefficients a_1 , a_2 , a_3 , and a_4 in Table 6. The resulting ψ_q is equal to 2.41.
- (3) The nominal allowable load Q_n that limits settlement to 25 mm or less with $p_{fr}=1\%$ exceeding the SLS is obtained as $R_{uc}/\psi_q=1612$ kN.

Conclusions

This paper utilized 162 reliable field load tests to interpret the measured resistances via the L_1 - L_2 method and determine the bivariate load-settlement model factors of driven piles in sand. Among these data, 92 usable load tests were applied to evaluate the accuracy of the ICP-05 and UWA-05 methods with CPT profile. It was observed that the ICP-05 and UWA-05 methods can give a more accurate prediction of resistance than the conventional design methods based on the lateral earth pressure theory.

For 111 reliable compression tests on driven piles with closed end, statistical analyses were implemented to characterize the probability models (mean, COV and distribution functions) of the resistance bias M_u and the bivariate load-settlement model factors a and b . Copula theory was applied to simulate the correlation structure within the model factors a and b . The statistics of the model factors M_u , a , and b were incorporated into simplified RBD procedures to calibrate the resistance factor ψ_R at the ULS and the lumped load-resistance factor ψ_q at the SLS using Monte-Carlo simulations. An example was given to show the application of ψ_q to estimate the allowable load for an allowable settlement at the described serviceability limit probability.

It should be pointed out that the number of load tests on driven piles with open-end is limited. More reliable field load tests should be conducted to evaluate the model factors M_u , a , and b . Due to

different behaviors, the applicability of the ICP-05 and UWA-05 methods for LDOEPs and reliability-based calibration of resistance factors ψ_R at the ULS and ψ_q at the SLS need to be further investigated in future.

References

- AASHTO. 2014. AASHTO LRFD Bridge Design Specifications, 7th ed. American Association of State Highway and Transportation Officials, Washington, D.C.
- AbdelSalam, S. S., Ng, K. W., Sritharan, S., Suleiman, M. T., and Roling, M. 2012. Development of LRFD procedure for bridge pile foundations in Iowa-volume III: recommended resistance factors with consideration of construction control in setup. *Report No. IHRB Projects TR-584*, Iowa Department of Transportation, February, 2012.
- Abu-Farsakh, M. Y., Yoon, S. M., and Tsai, C. 2009. Calibration of resistance factors needed in the LRFD design of driven piles. *Report No. FHWA/LA.09.449*, Louisiana Transportation Research Center, May 2009.
- Abu-Farsakh, M. Y., Chen, Q. M., and Haque, M. N. 2013. Calibration of resistance factors for drilled shafts for the new FHWA design method. *Report No. FHWA/LA.12/495*, Louisiana Transportation Research Center, January 2013.
- Abu-Hejleh, N. M., Abu-Farsakh, M., Suleiman, M. T., and Tsai, C. 2015. Development and use of high-quality databases of deep foundation load tests. *Transportation Research Record: Journal of the Transportation Research Board*, No. 2511, 27-36.
- Akaike, H. 1974. A new look at the statistical model identification. *IEEE Trans. Autom. Control*, 19(6), 716-723.
- Becker, D. E. 1996. Eighteenth Canadian Geotechnical Colloquium: Limit States Design for Foundations. Part 1. An overview of the foundation design process. *Can. Geotech. J.*, 33 (6): 956-983.
- Bahsan, E., Liao, H. J., and Ching, J. Y. 2014. Statistics for the calculated safety factors of undrained failure slopes. *Engineering Geology*, 172, 85-94.

- Brown, D. A., and Thompson III, W. R. 2015. Design and load testing of large diameter open-ended driven piles. *Report NCHRP Synthesis 478*, Transportation Research Board, Washington, D.C.
- Ching, J. Y., and Chen, J. R. 2010. Predicting displacement of augered cast-in-place piles based on load test database. *Struct. Safe.*, 32, 372-383.
- Ching, J. Y., Phoon, K. K., and Li, D.-Q. 2016. Robust estimation of correlation coefficients among soil parameters under the multivariate normal framework. *Struct. Safe.*, 63, 21-32.
- Ching, J. Y. 2017. Transformation models and multivariate soil databases. *Chapter 1 in Final report for Joint ISSMGE TC 205/TC 304 Working Group on "Discussion of statistical/reliability methods for Eurocodes"*, September 2017.
- Canadian Standards Association. 2014. *Canadian Highway Bridge Design Code*. CAN/CSA-S6-14, Mississauga, Ontario.
- Chow, F. C. 1997. Investigations into the behaviour of displacement piles for offshore foundations. *Ph.D. thesis*. Department of Civil & Environmental Engineering, Imperial College, London.
- Davisson, M. T. 1972. High capacity piles. *Proc., Lecture Series on Innovations in Foundation Construction*, ASCE, Illinois Section, Chicago.
- Dithinde, M., Phoon, K. K., De Wet, M., and Retief, J. V. 2011. Characterization of Model Uncertainty in the Static Pile Design Formula. *J. Geotech. Geoenviron. Eng.*, 137 (1), 70-85.
- Dithinde, M., Phoon, K. K., Ching, J. Y., Zhang, L. M., and Retief, J. V. 2016. Statistical characterization of model uncertainty. *Chapter 5 in Reliability of Geotechnical Structures in ISO2394*, Eds. K. K. Phoon & J. V. Retief, CRC Press/Balkema, 127-158.
- Fenton, G. A., Naghibi, F., Dundas, D., Bathurst, R. J., and Griffiths, D. V. 2016. Reliability-based geotechnical design in 2014 Canadian Highway Bridge Design Code. *Can. Geotech. J.*, 53 (2), 236-251.
- Flynn, K. N., McCabe, B. A., and Egan, D. 2014. Driven cast-in-situ piles in granular soil: application of CPT methods to pile capacity estimation. Proceedings of the 3rd International Symposium on Penetration Testing, Las Vegas, USA, paper #3-03.
- Flynn, K. N. 2014. Experimental investigations of driven cast-in-situ piles. *Ph.D. thesis*. College of Engineering and Informatics, National University of Ireland, Galway, Ireland.

- Flynn, K. N., and McCabe, B. A. 2016. Shaft resistance of driven cast-in-situ piles in sand. *Can. Geotech. J.*, 53 (1), 45-59.
- Hannigan, P. J., Rausche, F., Likins, G. E., Robinson, B. R., and Becker, M. L. 2016. Design and Construction of Driven Pile Foundations-Volume I. *Publication No. FHWA-NHI-16-009*, U. S. Department of Transportation, Federal Highway Administration, September 2016.
- Hirany, A., and Kulhawy, F. H. 1989a. Interpretation of load test on drilled shafts 1: Axial compression. *Foundation engineering: Current principles & practices* (GSP 22), F. H. Kulhawy, ed., ASCE, New York, 1132-1149.
- Hirany, A., and Kulhawy, F. H. 1989b. Interpretation of load tests on drilled shafts. 2: Axial uplift. *Foundation engineering: Current principles and practices* (GSP 22), F. H. Kulhawy, ed., ASCE, New York, 1150-1159.
- Huffman, J. C., and Stuedlein, A. W. 2014. Reliability-Based Serviceability Limit State Design of Spread Footings on Aggregate Pier Reinforced Clay. *J. Geotech. Geoenviron. Eng.*, 10.1061/(ASCE)GT.1943-5606.0001156, 04014055.
- Huffman, J. C., Strahler, A. W. and Stuedlein, A. W. 2015. Reliability-based serviceability limit state design for immediate settlement of spread footings on clay. *Soils Found.*, 55 (4), 798-812.
- Huffman, J. C., Martin, J. P., and Stuedlein, A. W. 2016. Calibration and assessment of reliability-based serviceability limit state procedures for foundation engineering. *Georisk: Assessment and Management of Risk for Engineered Systems and Geohazards*, 10 (4), 280-293.
- International Organization for Standardization. 2015. *General principles on reliability of structures. ISO2394:2015*, Geneva, Switzerland.
- Jardine, R. J., Chow, F. C., Overy, R., and Standing, J. R. 2005. *ICP design methods for driven piles in sands and clays*. Thomas Telford, London.
- Kulhawy, F. H., and Phoon, K. K. 2002. Observations on geotechnical reliability-based design development in North America. *Proceedings of International Workshop on Foundation Design Codes & Soil Investigation in View of International Harmonization & Performance Based Design*, edited by Y. Honjo, O. Kusakabe, K. Matsui, and G. Pokharel, Tokyo, Japan, 31-48.

- Lehane, B. M. 1992. Experimental investigations of pile behaviour using instrumented field piles. *Ph.D. thesis*. Department of Civil & Environmental Engineering, Imperial College, London.
- Lehane, B. M., Schneider, J. A., and Xu, X. 2005. The UWA-05 method for prediction of axial capacity of driven piles in sand. In *Proceedings of the International Symposium on Frontiers in Offshore Geomechanics ISFOG*. Taylor & Francis, London. Pp. 683-689.
- Lesny, K. 2017. Evaluation and consideration of model uncertainties in reliability based design. *Chapter 2 in Final report for Joint ISSMGE TC 205/TC 304 Working Group on "Discussion of statistical/reliability methods for Eurocodes"*, September 2017.
- Li, D. Q., Tang, X. S., Phoon, K. K., Chen, Y. F., and Zhou, C. B. 2013. Bivariate simulation using copula and its application to probabilistic pile settlement analysis. *Int. J. Numer. Anal. Meth. Geomech.*, 37 (6), 597-617.
- Marcos, M. C., Chen, Y. J., and Kulhawy, F. H. 2013. Evaluation of compression load test interpretation criteria for driven precast concrete pile capacity. *KSCE Journal of Civil Engineering-Geotechnical Engineering*, 17 (5), 1008-1022.
- Motamed, R., Elfass, S., and Stanton, K. 2016. LRFD resistance factor calibration for axially loaded drilled shafts in the Las Vegas valley. *Report No. 515-13-803*, Nevada Department of Transportation, July 19, 2016.
- NeSmith, V. M., and Siegel, T. C. 2009. Shortcomings of the Davisson offset limit applied to axial compressive load tests on cast-in-place piles. *Proceedings of 2009 International Foundation Congress and Equipment Expo*, Orlando, Florida, GSP No. 185, 568-574.
- Ng, K. W., Sritharan, S., and Ashlock, J. C. 2014. Development of preliminary load and resistance factor design of drilled shafts in Iowa. *Report No. InTrans Project 11-410*, Iowa Department of Transportation.
- Ng, T. T., and Fazia, S. 2012. Development and Validation of a Unified Equation for Drilled Shaft Foundation Design in New Mexico. *Report No. NM10MSC-01*, New Mexico Department of Transportation, Albuquerque, December 3, 2012.

- Paikowsky, S. G., and Tolosko, T. A. 1999. Extrapolation of pile capacity from non-failed load tests. *Report No. FHWA-RD-99-170*, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C.
- Paikowsky, S. G., Birgisson, B., McVay, M., Nguyen, T., Kuo, C., Baecher, G. B., Ayyub, B., Stenersen, K., O'Malley, K., Chernauskas, L., and O'Neill, M. 2004. Load and Resistance Factors Design for Deep Foundations. *NCHRP Report 507*, Transportation Research Board of the National Academies, Washington, D.C.
- Phoon, K. K., and Kulhawy, F. H. 1999. Characterization of geotechnical variability. *Can. Geotech. J.*, 36 (4), 612-624.
- Phoon, K. K., Chen, J. R., and Kulhawy, F. H. 2006. Characterization of model uncertainties for augered cast-in-place (ACIP) piles under axial compression. In: *Foundation Analysis & Design: Innovative Methods (GSP 153)*, Reston, 82-89.
- Phoon, K. K., Chen, J. R., and Kulhawy, F. H. 2007. Probabilistic hyperbolic models for foundation uplift movements. In: *Probabilistic Applications in Geotechnical Engineering (GSP 170)*. Reston, ASCE. CD-ROM.
- Phoon, K. K., and Kulhawy, F. H. 2008. Serviceability-limit state reliability-based design. In: Phoon, K. K. (ed.) *Reliability-Based Design in Geotechnical Engineering: Computations and Applications*. London, Taylor & Francis. Pp. 344-383.
- Phoon, K. K., Retief, J. V., Ching, J. Y., Dithinde, M., Schweckendiek, T., Wang, Y., and Zhang, L. M. 2016. Some observations on ISO2394:2015 Annex D (Reliability of Geotechnical Structures). *Structural Safety*, 62, 24-33.
- Phoon, K. K. 2017. Role of reliability calculations in geotechnical design. *Georisk: Assessment and Management of Risk for Engineered Systems and Geohazards*, 11 (1), 4-21.
- Randolph, M. F., Dolwin, R., and Beck, R. 1994. Design of driven piles in sand. *Géotechnique*, 44 (3): 427-448.
- Randolph, M. F. 2003. Science and empiricism in pile foundation design. *Géotechnique*, 53 (10), 847-875.

- Reddy, S. C., and Stuedlein, A. W. 2017a. Ultimate limit state reliability-based design of augered cast-in-place piles considering lower-bound capacities. *Can. Geotech. J.*, 54 (12), 1693-1703.
- Reddy, S. C., and Stuedlein, A. W. 2017b. Serviceability limit state reliability-based design of augered cast-in-place piles in granular soils. *Can. Geotech. J.*, 54 (12), 1704-1715.
- Roling, M. J., Sritharan, S., and Suleiman, M. T. 2011. Development of LRFD procedures for bridge pile foundations in Iowa—Volume I: An electronic database for pile load tests in Iowa (PILOT). *Report No. IHRB Project TR-573*, Iowa Department of Transportation, June 2010 (updated in January 2011).
- Schneider, J. A., Xu, X., and Lehane, B. M. 2008. Database assessment of CPT-based design methods for axial capacity of driven piles in siliceous sands. *J. Geotech. Geoenviron. Eng.*, 134 (9): 1227-1244.
- Schwarz, G. 1978. Estimating the dimension of a model. *Ann. Stat.*, 6 (2), 461-464.
- Stuedlein, A. W., Neely, W. J., and Gurtowski, T. M. 2012. Reliability-based design of augered cast-in-place piles in granular soils. *J. Geotech. Geoenviron. Eng.*, 138 (6), 709-717.
- Stuedlein, A. W., and Reddy, S. C. 2013. Factors affecting the reliability of augered cast-in-place piles in granular soils at the serviceability limit state. *DFI Journal-The Journal of the Deep Foundations Institute*, 7 (2), 46-57.
- Tang, C., and Phoon, K. K. 2017. Model uncertainty of Eurocode 7 approach for bearing capacity of circular footings on dense sand. *Int. J. Geomech.*, 17 (3), 04016069.
- Tang, C., Phoon, K. K., and Akbas, S. O. 2017a. Model uncertainties for the static design of square foundations on sand under axial compression. *Geo-Risk 2017: Reliability-based design and code developments (Geotechnical Special Publication 283)*, edited by Huang, J. S., Fenton, G. A., Zhang, L. M., and Griffiths, D. V., pp. 141-150.
- Tang, C., Phoon, K. K., Zhang, L., and Li, D. Q. 2017b. Model uncertainty for predicting the bearing capacity of sand overlying clay. *Int. J. Geomech.*, 17 (7), 04017015.
- Tang, C., and Phoon, K. K. 2018a. Evaluation of model uncertainties in reliability-based design of steel H-piles in axial compression. *Can. Geotech. J.*, accepted.

- Tang, C., and Phoon, K. K. 2018b. Statistics of model factors and consideration in reliability-based design of axially loaded helical piles. *J. Geotech. Geoenviron. Eng.*, accepted.
- Terzaghi, K., and Peck, R. B. 1967. *Soil mechanics in engineering practice*. 2nd Ed., Wiley, New York.
- Tomlinson, M., and Woodward, J. 2008. *Pile design and construction practice*. Fifth edition, Taylor & Francis.
- Toolan, F. E., Lings, M. L., and Mirza, U. A. 1990. An appraisal of API RP2A recommendations for determining skin friction of piles in sand. *Proceedings of the 22nd Offshore Technology Conference (OTC 6422)*, Houston, Tex., 7-10 May 1990, pp. 33-42.
- Travis, Q. B., Schmeeckle, M. W., and Sebert, D. M. 2011. Meta-analysis of 301 slope failure calculations. II: database analysis. *J. Geotech. Geoenviron. Eng.*, 137 (5), 471-482.
- Uzielli, M. and Mayne, P. W. 2011. Serviceability limit state CPT-based design for vertically loaded shallow footings on sand. *Geomechanics and Geoengineering*, 6 (2), 91-107.
- Wang, Y., and Kulhawy, F. H. 2008. Reliability index for serviceability limit state of building foundations. *J. Geotech. Geoenviron. Eng.*, 134 (11), 1587-1594.
- Wu, S. H., Ou, C. Y., and Ching, J. Y. 2014. Calibration of model uncertainties for basal heave stability of wide excavations in clay. *Soils Found.*, 54 (6), 1159-1174.
- Yang, Z. X., Jardine, R. J., Guo, W. B., and Chow, F. 2015. *A comprehensive database of tests on axially loaded piles driven in sands*. Zhejiang University Press & Elsevier.
- Yang, Z. X., Guo, W. B., Jardine, R. J., and Chow, F. 2017. Design method reliability assessment from an extended database of axial load tests on piles driven in sand. *Can. Geotech. J.*, 54 (1), 59-74.
- Zhang, D. M., Phoon, K. K., Huang, H. W., and Hu, Q. F. 2015. Characterization of model uncertainty for cantilever deflections in undrained clay. *J. Geotech. Geoenviron. Eng.*, 141 (1), 04014088.
- Zhang, L. M., and Ng, A. M. Y. 2005 Probabilistic limiting tolerable displacements for serviceability limit state design of foundations. *Géotechnique*, 55 (2), 151-161.

- Zhang, L. M., Xu, Y., and Tang, W. H. 2008. Calibration of models for pile settlement analysis using 64 field load tests. *Can. Geotech. J.*, 45 (1), 59-73.
- Zhang, L. M., and Chu, L. F. 2009a. Calibration of methods for designing large-diameter bored piles: ultimate limit state. *Soils Found.*, 49(6), 883-896.
- Zhang, L. M., and Chu, L. F. 2009b. Calibration of methods for designing large-diameter bored piles: serviceability limit state. *Soils Found.*, 49(6), 897-908.

Draft

List of Figure Caption

- Fig. 1. Measured load-settlement curves for driven piles with closed-end
- Fig. 2. Measured load-settlement curves for driven piles with open-end
- Fig. 3. Predicted resistances from the ICP-05 and UWA-05 methods
- Fig. 4. Comparison between the measured resistance and the calculated resistance from the ICP-05 method
- Fig. 5. Scatter diagrams of M_u versus (a) B , (b) D/B , and (c) D_r
- Fig. 6. Scatter plots of b versus (a) B , (b) D/B , and (c) D_r
- Fig. 7. Scatter diagrams of a versus (a) B , (b) D/B , and (c) D_r
- Fig. 8. Cumulative distribution function for M_u
- Fig. 9. Cumulative distribution function for b
- Fig. 10. Cumulative distribution function for a
- Fig. 11. Observed and simulated of correlation within the hyperbolic parameters a and b
- Fig. 12. Simulated and observed load-settlement curves for driven piles with closed-end in sand
- Fig. 13. Effect of the ratio η of dead to live load on the resistance factor ψ_R
- Fig. 14. Variation of reliability index β with allowable settlement s_a and load-resistance factor ψ_q for $COV_Q=0.1$
- Fig. 15. Variation of best-fit coefficients p_1 and p_2 with allowable settlement s_a for $COV_Q=0.1$

Fig. 1. Measured load-settlement curves for driven piles with closed-end

Fig. 2. Measured load-settlement curves for driven piles with open-end

Fig. 3. Predicted resistances from the ICP-05 and UWA-05 methods

Fig. 4. Comparison between the measured resistance and the calculated resistance from the ICP-05 method

Fig. 5. Scatter diagrams of M_u versus (a) B , (b) D/B , and (c) D_r

Fig. 6. Scatter plots of b versus (a) B , (b) D/B , and (c) D_r

Fig. 7. Scatter diagrams of a versus (a) B , (b) D/B , and (c) D_r

Fig. 8. Cumulative distribution function for M_u

Draft

Fig. 9. Cumulative distribution function for b

Fig. 10. Cumulative distribution function for a

Draft

Fig. 11. Observed and simulated of correlation within the hyperbolic parameters a and b

Fig. 12. Simulated and observed load-settlement curves for driven piles with closed-end in sand

Fig. 13. Effect of the ratio η of dead to live load on the resistance factor ψ_R

Fig. 14. Variation of reliability index β with allowable settlement s_a and load-resistance factor ψ_q for $COV_{Q'}=0.1$

Fig. 15. Variation of best-fit coefficients p_1 and p_2 with allowable settlement s_a for $COV_Q = 0.1$

List of Tables

Table 1. Summary of ICP-05 and UWA-05 methods to calculate q_b and τ_f in sand, taken from Yang et al. (2015)		
Method	Base resistance q_b	Shaft friction τ_f
ICP-05 (Jardine et al. 2015)	Closed-end: $q_b = \max[1 - 0.5 \log(B/D_{CPT}), 0.3] q_{c,avg}$ Open-end, unplugged $q_b = A_r q_{c,avg}$; Plugged: $q_b = \max[0.5 - 0.25 \log(B/D_{CPT}), 0.15, A_r] q_{c,avg}$, where $A_r = 1 - (B_i/B)^2$ If $(B_i \geq 2(D_r - 0.3))$ or $B_i \geq 0.083(q_{c,avg}/p_a) D_{CPT}$, pile is unplugged, otherwise plugged (B_i in meters); p_a =reference stress (=100 kPa); $D_r = 0.4 \ln(q_{c1N}/22)$ as decimal For non-circular piles, use equivalent base area Square/rectangular: $q_b = 0.7 q_{c,avg}$ H section: $q_b = q_{c,avg}$	$\tau_f = a(\sigma'_{rc} + \Delta\sigma'_{rd}) \tan \delta_f$ $\sigma'_{rc} = 0.029 b q_c (\sigma'_{v0}/p_a)^{0.13} [\max(h/R^*, 8)]^{-0.38}$ $\Delta\sigma'_{rd} = 2G\Delta y/R^*$, $\Delta y \approx 0.02$ mm $a = 0.9$ for open-end piles in uplift and 1 for all other cases $b = 0.8$ for piles in uplift and 1 for piles in compression δ_f measured or estimated as a function of d_{50} $G \approx 185 q_c (q_{c1N})^{-0.7}$ or $G \approx q_c [0.0203 + 0.00125 q_{c1N} - 1.216 e^{-6} (q_{c1N})^2]^{-1}$ $q_{c1N} = (q_c/p_a)/(\sigma'_{v0}/p_a)^{0.5}$
	UWA-05 (Lehane et al. 2015) $q_b = 0.6 q_{c,avg}$ for closed-end $q_b = (0.15 + 0.45 A_{rb,eff}) q_{c,avg}$ for open-end $A_{rb,eff} = 1 - FFR(B_i/B)^2$; $FFR \approx \min[1, (B_i/1.5)^{0.2}]$	$\tau_f = (f_i/f_c) \{0.03 q_c (A_{rs,eff})^{0.3} [\max(h/B, 2) - 0.5] + \Delta\sigma'_{rd}\} \tan \delta_f$ $A_{rs,eff} = 1 - IFR(B_i/B)^2$; $IFR \approx \min[1, (B_i/1.5)^{0.2}]$ $f_i/f_c = 1$ for compression and 0.75 for uplift

Notation:

(1) ICP-05 method: h =height above pile tip; B =external diameter of pile shaft; B_i =internal pile diameter; D_{CPT} =cone diameter=0.036 m; $q_{c,avg}=q_c$ averaged $\pm 1.5B$ over pile tip level; A_r =area ratio; R^* =modified radius for open-end piles= $(R^2 - R_i^2)^{0.5}$, R =external pile radius= $B/2$, R_i =internal pile radius= $B_i/2$; for square, rectangular and H-piles, $R^*=(A_b/\pi)^{0.5}$, A_b =base area= $w_a \times w_b$ (square or rectangular section), w_a =width, w_b =breadth; $A_b=A_h+2x_p(d_h-2t_h)$ (H section), A_h =area of H section, $x_p=b_h/8$, if $b_h/2 < (d_h-2t_h) < b_h$, $x_p=b_h^2/[16(d_h-2t_h)]$ if $(d_h-2t_h) \geq b_h$, b_h =flange width, d_h =total depth, t_h =flange thickness; σ'_{v0} =effective vertical stress; σ'_{rc} =radial effective stress following pore pressure equalization; σ'_{rd} =change in radial stress during pile loading; δ_f =interface friction angle at failure; d_{50} =mean particle size; G =operational shear modulus; Δy =radial displacement due to dilation during loading; and D_r =nominal relative density.

(2) UWA-05 method: $A_{rb,eff}$ =effective area ratio for pile base; FFR =final filling ratio; $A_{rs,eff}$ =effective area ratio for shaft; and IFR =incremental filling ratio.

Table 2. Description and ranges of the reliable load tests used for analyses

Load type	Pile information			Reliable	Usable	Ranges	
	Type	Material	Shape			B (m)	D/B
Compression	Closed-end	Concrete	Square	56	22	0.14~0.76	13~251
			Circular	17	13		
			Octagonal	5	1		
	Open-end	Steel	Circular	39	16	0.32~0.61	14~133
			Circular	5	5		
			Square	1	1		
Uplift	Closed-end	Steel	Circular	9	9	0.25~0.61	19~84
	Open-end	Steel	Circular	19	19	0.32~0.76	19~62

Draft

Table 3. Summary of the ULS model statistics for driven piles

Soil type	Pile type	Load type	N	Design method	Mean	COV	Data sources
Clay	Steel H-pile	Compression	4	β -Method	0.61	0.61	Paikowsky et al. (2004)
			16	λ -Method	0.74	0.39	
			17	α -Tomlinson	0.82	0.4	
			16	α -API	0.9	0.41	
			8	SPT-97 mob	1.04	0.41	
	Concrete pile	Compression	18	λ -Method	0.76	0.29	
			17	α -API	0.81	0.26	
			8	β -Method	0.81	0.51	
			18	α -Tomlinson	0.87	0.48	
	Pipe pile	Compression	18	α -Tomlinson	0.64	0.5	
			19	α -API	0.79	0.54	
			12	β -Method	0.45	0.6	
			19	λ -Method	0.67	0.55	
			12	SPT-97 mob	0.39	0.62	
Sand	Steel H-pile	Compression	19	Nordlund	0.94	0.4	
			18	Meyerhof	0.81	0.38	
			19	β -Method	0.78	0.51	
			18	SPT-97 mob	1.35	0.43	
	Concrete pile	Compression	36	Nordlund	1.02	0.48	
			35	β -Method	1.1	0.44	
			36	Meyerhof	0.61	0.61	
			36	SPT-97 mob	1.21	0.47	
	Pipe pile	Compression	19	Nordlund	1.48	0.52	
			20	β -Method	1.18	0.62	
			20	Meyerhof	0.94	0.59	
			19	SPT-97 mob	1.58	0.52	
Layered	Steel H-pile	Compression	20	α -Tomlinson/Nordlund/Thurman	0.59	0.39	
			34	α -API/Nordlund/Thurman	0.79	0.44	
			32	β -Method/Thurman	0.48	0.48	
			40	SPT-97	1.23	0.45	

Table 3 (*continued.*)

Soil type	Pile type	Load type	N	Design method	Mean	COV	Data sources
Layered	Concrete pile	Compression	33	α -Tomlinson/Nordlund/Thurman	0.96	0.49	Paikowsky et al. (2004)
			80	α -API/Nordlund/Thurman	0.87	0.48	
			80	β -Method/Thurman	0.81	0.38	
			71	SPT-97 mob	1.81	0.5	
			30	FHWA CPT	0.84	0.31	
	Pipe pile	Compression	13	α -Tomlinson/Nordlund/Thurman	0.74	0.59	
			32	α -API/Nordlund/Thurman	0.8	0.45	
			29	β -Method/Thurman	0.54	0.48	
			33	SPT-97 mob	0.76	0.38	
Sand	Concrete pile	Compression	24	Meyerhof	1.22	0.54	FHWA-HI-98-032
Sand	Driven pile	Compression	28	Static formula	1.11	0.33	Dithinde et al. (2011)
Clay	Driven pile	Compression	59		1.17	0.26	
Sand	Drilled shaft	Compression	30		0.98	0.24	
Clay	Drilled shaft	Compression	53		1.15	0.25	
Clay	Square shaft helical pile (single-helix)	Compression	16	Torque-correlation method	0.88	0.15	Tang & Phoon (2018b)
		Uplift	14		0.74	0.27	
	Square shaft helical pile (multi-helix)	Compression	14		1.04	0.19	
		Uplift	10		0.93	0.26	
	Helical pipe pile (single-helix)	Compression	49	Individual plate bearing	1.25	0.41	
		Uplift	75	Torque-correlation method	1.09	0.26	
		Compression	54		0.92	0.23	
		Uplift	71		1.16	0.18	
	Helical pipe pile (multi-helix)	Compression	69		1.02	0.27	
		Uplift					
Sand	Square shaft helical pile (single-helix)	Compression	6	Torque-correlation method	1.51	0.39	
		Uplift	7		1.2	0.56	
	Square shaft helical pile (multi-helix)	Compression	10		1.54	0.39	
		Uplift	10		1.06	0.22	
	Helical pipe pile (single-helix)	Compression	55	Individual plate bearing	1.46	0.42	
		Uplift	50	Torque-correlation method	1.23	0.37	
		Compression	47		0.98	0.3	
		Uplift					

Table 3 (continued.)

Soil type	Pile type	Load type	N	Design method	Mean	COV	Data sources
Sand	Helical pipe pile (multi-helix)	Compression	49	Torque-correlation method	1.51	0.26	Tang & Phoon (2018b)
		Uplift	51		1.2	0.24	
Clay	Steel H-pile	Compression	20	API α -method	1.15	0.52	AbdelSalam et al. (2012)
Sand			34	Nordlund method	0.92	0.53	
Layered			26	API α /Nordlund method	1.04	0.4	
Clay	Steel H-pile	Compression	26	API α -method	1.1	0.4	Tang & Phoon (2018a)
Sand			46	Nordlund method	0.82	0.47	
Layered			32	API α /Nordlund method	0.92	0.4	
Sand	Open-end (Concrete/Steel)	Compression	16	ICP-05 method	1.07	0.24	This work
				UWA-05 method	1.07	0.21	
	Closed-end (Concrete/Steel)	Compression	52	ICP-05 method	1.1	0.31	
				UWA-05 method	1	0.39	
	Open-end (Steel)	Uplift	19	ICP-05 method	1.36	0.38	
				UWA-05 method	1.3	0.37	
	Closed-end (Steel)	Uplift	9	ICP-05 method	1.02	0.35	
				UWA-05 method	1.02	0.37	

Table 4. Summary of the statistics for the bivariate load-settlement model factors of foundations

Foundation type	Load type	Soil type	<i>N</i>	<i>a</i> (mm)		<i>b</i>		ρ	Data sources
				mean	COV	mean	COV		
ACIP pile ($D/B > 20$)	Compression	Sand	40	5.15	0.6	0.62	0.26	-0.67	Phoon et al.(2006)
Spread footing	Uplift	Clay+sand	85	7.13	0.65	0.75	0.18	-0.24	Phoon e al. (2007)
Drilled shaft	Uplift	Clay+sand	48	1.34	0.54	0.89	0.07	-0.59	
Pressure-injected footing	Uplift	Sand	25	1.38	0.68	0.77	0.27	-0.73	
Driven pile	Compression	Sand	28	5.55	0.54	0.71	0.14	-0.78	Dithinde et al. (2011)
Bored pile	Compression	Sand	30	4.1	0.78	0.77	0.21	-0.88	
Driven pile	Compression	Clay	59	3.58	0.57	0.78	0.11	-0.89	
Bored pile	Compression	Clay	53	2.79	0.57	0.82	0.11	-0.8	
ACIP piles	Compression	Sand	95	0.16	0.47	3.38	0.23	-0.67	Reddy and Stuedlein (2017b)
Spread footing	Compression	Reinforced clay	30	2	0.79	1.15	0.25	—	Huffman and Stuedlein (2014)
Spread footing	Compression	Clay	30	1.3	0.53	0.7	0.16	-0.95	Huffman et al. (2015)
Square foundation	Compression	Sand	64	1.47	0.4	0.72	0.17	-0.76	Tang et al. (2017a)
Steel H-piles	Compression	Clay	47	1.07	0.37	1.01	0.09	-0.51	Tang & Phoon (2018a)
	Compression	Sand	52	1.17	0.6	0.69	0.18	-0.6	
	Compression	Layered	50	1.17	0.59	0.75	0.13	-0.53	
Square shaft helical pile	Compression	Clay	53	5.54	0.36	0.78	0.14	-0.46	Tang & Phoon (2018b)
(multi-helix)	Compression	Sand	49	5.84	0.27	0.76	0.14	-0.36	
Driven piles with closed-end (concrete/steel)	Compression	Sand	111	6.26	0.75	0.8	0.15	-0.56	This work

Note:

- (1) The bivariate load-settlement model for shallow foundations used by Huffman and Stuedlein (2014), Huffman et al. (2015), and Tang et al. (2017a) is $Q/R_{um} = (s/B)/[a + b(s/B)]$.
- (2) The statistics in Reddy and Stuedlein (2017b), Tang & Phoon (2018a) are presented for the transformed model factors with removing the dependency on D/B .
- (3) ρ =correlation within the bivariate load-settlement model factors.

Table 5. Resistance factors for the ICP-05 and UWA-05 methods of driven piles with closed-end in sand under axial compression

$\eta=Q_{DL}/Q_{LL}$	Methods	ψ_R		ψ_R/M_u	
		$\beta_T=2.33$	$\beta_T=3$	$\beta_T=2.33$	$\beta_T=3$
1	ICP-05	0.61	0.48	0.55	0.44
	UWA-05	0.47	0.35	0.47	0.35
2	ICP-05	0.59	0.46	0.54	0.42
	UWA-05	0.45	0.34	0.45	0.34
3	ICP-05	0.57	0.45	0.52	0.41
	UWA-05	0.44	0.33	0.44	0.33
4	ICP-05	0.57	0.44	0.52	0.4
	UWA-05	0.44	0.33	0.44	0.33
5	ICP-05	0.56	0.44	0.51	0.4
	UWA-05	0.43	0.32	0.43	0.32

Draft

Table 6. Summary of best-fit coefficients a_1 , a_2 , a_3 , and a_4 to estimate β or ψ_q for the ICP-05 method

$COV_{\phi'}$	COV_{sB}	a_1	a_2	a_3	a_4
0.1	0	0.45	1.3	0.79	-2.63
	0.2	0.47	1.22	0.78	-2.59
	0.4	0.51	1.02	0.76	-2.52
	0.6	0.56	0.76	0.73	-2.42
0.2	0	0.35	1.36	0.74	-2.43
	0.2	0.37	1.29	0.73	-2.41
	0.4	0.41	1.12	0.72	-2.35
	0.6	0.45	0.89	0.69	-2.28

Draft

Appendix A1. Collected Field Load Tests on Axially Loaded Driven Piles in Sand

Table A1-1. Selected axial compression load tests on driven piles in sand

Database	Test site location	Pile No.	Pile information			<i>D</i> (m)	<i>B</i> (m)	<i>R</i> _{um} (kN)	<i>R</i> _{uc} (kN)		<i>b</i>	<i>a</i> (mm)
			Type	Material	Shape				ICP-05	UWA-05		
ZJU-ICL	Wuhu, K24, China	K24-3	Open-end	Concrete	Square	39.8	0.5	4221	4539	3502	0.8	6.95
ZJU-ICL		K24-1	Open-end	Concrete	Circular	33	0.6	1798	2647	3025	0.76	3.35
ZJU-ICL		K24-2	Open-end	Concrete	Circular	39.8	0.6	4204	5234	3859	0.83	5.91
ZJU-ICL	Wuhu K34, China	K34-1	Open-end	Concrete	Circular	29.3	0.6	4813	5500	4634	0.87	5
ZJU-ICL	Wuhu K27, China	K27-1	Open-end	Concrete	Circular	29.2	0.8	4830	5053	4536	0.83	8.11
ZJU-ICL	ABEF Foundation, Brazil	7	Open-end	Concrete	Circular	8.7	0.5	3140	2543	2331	0.87	3.92
ZJU-ICL	Mobile Bay, AL, USA	AL 1	Open-end	Steel	Circular	15.2	0.32	1200	893	892	0.86	1.79
ZJU-ICL		AL 2	Open-end	Steel	Circular	42.7	0.32	3542	3093	2636	0.8	10.4
ZJU-ICL	Hoogzand, The Netherlands	1-C	Open-end	Steel	Circular	7	0.36	2270	1512	1850	0.88	3.24
ZJU-ICL		3-C	Open-end	Steel	Circular	5.3	0.36	1815	1339	1538	0.88	3.37
ZJU-ICL	Euripides, The Netherlands	Ia	Open-end	Steel	Circular	30.5	0.76	7400	8745	10149	0.69	12.4
ZJU-ICL		Ib	Open-end	Steel	Circular	38.7	0.76	13000	14214	15838	0.84	7.55
ZJU-ICL		Ic	Open-end	Steel	Circular	47	0.76	18800	19156	19311	0.82	9.14
ZJU-ICL		II	Open-end	Steel	Circular	46.7	0.76	18400	19708	20357	0.87	6.83
ZJU-ICL	Drammen, Norway	16-P1-11	Open-end	Steel	Circular	11	0.81	1210	791	1007	0.77	9.62
ZJU-ICL	Dunkirk GOPAL, France	C1-C	Open-end	Steel	Circular	10	0.46	2732	2614	2657	0.82	4.32
DFLT	Cleveland, OH, USA	SPO-2	Open-end	Steel	Circular	36.6	0.46	3407			0.63	16.1
ZJU-ICL	Rio de Janeiro, Brazil	PI-1	Closed-end	Concrete	Square	37.2	0.5	2800	3878	4336	0.52	11.8
ZJU-ICL		PI-2	Closed-end	Concrete	Square	21.4	0.5	1869	1748	1700	0.67	6.06
ZJU-ICL		PI-4	Closed-end	Concrete	Square	26.5	0.5	4223	4118	4429	0.89	3.29
ZJU-ICL	Rotterdam Harbor, The Netherlands	P6	Closed-end	Concrete	Square	30.6	0.38	4082	4418	5560	0.38	20.6
ZJU-ICL		P8	Closed-end	Concrete	Square	30.3	0.38	4447	4340	5491	0.69	13.8
ZJU-ICL		P10	Closed-end	Concrete	Square	30.7	0.38	4135	4437	5579	0.39	21
ZJU-ICL	Waddinxveen site, The Netherlands	P2	Closed-end	Concrete	Square	10	0.35	1078	713	968	0.89	3.21
ZJU-ICL	Apalachicola River, USA	BR 1	Closed-end	Concrete	Square	29.9	0.61	4297	6364	10159	0.87	4.03
ZJU-ICL	Los Angeles, CA site, USA	CA	Closed-end	Concrete	Square	29	0.61	5585	6134	9108	0.83	9.75
ZJU-ICL	MS Desota, USA	2108	Closed-end	Concrete	Square	7.6	0.46	1600	1434	2214	0.95	3.98
ZJU-ICL	MS Harrison, USA	3028	Closed-end	Concrete	Square	16.2	0.46	1420	1612	2177	0.93	1.33
ZJU-ICL	Washington MS, USA	3118A	Closed-end	Concrete	Square	7.6	0.41	700	571	937	0.89	1.9
ZJU-ICL		3123B	Closed-end	Concrete	Square	16.6	0.36	990	1122	1371	0.88	1.83
ZJU-ICL		3142A	Closed-end	Concrete	Square	6.2	0.36	850	1203	2122	0.89	3.09
ZJU-ICL	Fittja Straits, Sweden	D-1	Closed-end	Concrete	Square	13	0.24	293	319	351	0.83	2.72
ZJU-ICL		D-5	Closed-end	Concrete	Square	12.8	0.24	308	320	352	0.79	2.61

ZJU-ICL	Ogeechee River, USA	H-2	Closed-end	Concrete	Square	15.2	0.41	2851	2229	2836	0.82	9.04
ZJU-ICL	Baghdad University, Iraq	P1-C	Closed-end	Concrete	Square	11	0.25	951	519	596	0.92	2.27
ZJU-ICL		P2-C	Closed-end	Concrete	Square	15	0.25	1522	754	756	0.79	4.7
ZJU-ICL		P23	Closed-end	Concrete	Square	16.8	0.24	1630	914	1073	0.81	4.63
ZJU-ICL	Jonkoping, Sweden	P25	Closed-end	Concrete	Square	17.8	0.24	1400	1111	1319	0.68	4.44
ZJU-ICL		P26	Closed-end	Concrete	Square	16.2	0.28	1362	953	1160	0.87	3.66
DFLTD	Omaha, NE, USA	SC-2	Closed-end	Concrete	Square	19.8	0.31	1601			0.87	3.54
DFLTD	Alsea Bay, OR, USA	SC-1	Closed-end	Concrete	Square	41.1	0.51	7340			0.56	26
DFLTD	Orlando, FL, USA	SC-1	Closed-end	Concrete	Square	35.1	0.14	3594			0.54	11
DFLTD	Cimarron, OK, USA	SC-3	Closed-end	Concrete	Square	19.3	0.61	7784			0.86	5.73
DFLTD	Harris County, TX, USA	SC-2	Closed-end	Concrete	Square	30.9	0.2	4420			0.85	3.45
DFLTD		SC-1	Closed-end	Concrete	Square	27.4	0.61	4768			0.7	15
DFLTD	Dawhoo, SC, USA	SC-2	Closed-end	Concrete	Square	24.4	0.41	3063			0.96	8.69
DFLTD		SC-1	Closed-end	Concrete	Square	38.1	—	4003			0.82	6.21
DFLTD	Choctawhatchee, FL, USA	SC-1	Closed-end	Concrete	Square	23.1	0.24	2104			0.94	1.28
DFLTD		SC-2	Closed-end	Concrete	Square	29.6	0.76	6606			0.83	6.3
DFLTD		SC-3	Closed-end	Concrete	Square	26.9	0.76	7205			0.78	8.71
DFLTD		SC-4	Closed-end	Concrete	Square	27.9	0.76	3601			0.91	4.69
DFLTD		SC-5	Closed-end	Concrete	Square	28.7	0.76	4402			0.89	4.46
DFLTD		SC-6	Closed-end	Concrete	Square	31.1	0.76	6397			0.7	7.57
DFLTD		SC-7	Closed-end	Concrete	Square	32.4	0.76	6402			0.67	9.46
DFLTD		SC-8	Closed-end	Concrete	Square	25.6	0.61	4452			0.84	3.86
DFLTD		SC-2	Closed-end	Concrete	Square	13.3	0.61	2443			0.7	5
DFLTD		SC-1	Closed-end	Concrete	Square	27.7	0.31	1578			0.49	9.68
DFLTD	Charleston, SC, USA	SC-1	Closed-end	Concrete	Square	26.1	0.61	4329			0.66	7.13
DFLTD		SC-2	Closed-end	Concrete	Square	31	0.76	6757			0.82	1.29
DFLTD	Tampa Bay, FL, USA	SC-2	Closed-end	Concrete	Square	31	0.76	6757			0.82	1.29
DFLTD	Sarasota, FL, USA	SC-1	Closed-end	Concrete	Square	8.84	0.31	890			0.79	3.49
DFLTD	Fort Belvoir, VA, USA	SC-3	Closed-end	Concrete	Square	20.6	0.46	1779			0.83	2.77
DFLTD		SC-1	Closed-end	Concrete	Square	33.5	0.61	2251			0.71	3.87
DFLTD	West Bay, FL, USA	SC-1	Closed-end	Concrete	Square	39.6	0.76	4226			0.9	3.16
DFLTD		SC-2	Closed-end	Concrete	Square	33.2	0.76	3781			0.94	1.71
DFLTD	St. Mary Parish, LA, USA	SC-1	Closed-end	Concrete	Square	19.4	0.36	1468			0.69	3.62
DFLTD		SC-1	Closed-end	Concrete	Square	31.7	0.76	4226			0.91	3
DFLTD		SC-1	Closed-end	Concrete	Square	34.4	0.76	4893			0.92	2.17
DFLTD		SC-1	Closed-end	Concrete	Square	24.5	0.41	979			0.91	1.45
DFLTD		1	Closed-end	Concrete	Square	20.4	—	1779			0.84	2.59
DFLTD	Mobile, AL, USA	2	Closed-end	Concrete	Square	23.5	—	2535			0.83	3.22
DFLTD		3	Closed-end	Concrete	Square	20.4	—	2669			0.81	1.96
DFLTD		4	Closed-end	Concrete	Square	23.5	—	3781			0.72	5.92

DFLTD		5	Closed-end	Concrete	Square	22.6	—	5115			0.89	2.34
ZJU-ICL		A	Closed-end	Concrete	Circular	8	0.28	280	237	255	0.97	1.12
ZJU-ICL		D/A	Closed-end	Concrete	Circular	16	0.28	495	490	494	0.95	1.41
ZJU-ICL		E-7.5	Closed-end	Concrete	Circular	7.5	0.28	210	221	237	0.95	1.73
ZJU-ICL	Drammen, Norway	E-11.5	Closed-end	Concrete	Circular	11.5	0.28	330	344	350	0.95	1.89
ZJU-ICL		E-15.5	Closed-end	Concrete	Circular	15.5	0.28	470	472	467	0.93	1.98
ZJU-ICL		E-19.5	Closed-end	Concrete	Circular	19.5	0.28	640	762	751	0.89	3.2
ZJU-ICL		E-23.5	Closed-end	Concrete	Circular	23.5	0.28	850	893	893	0.86	4.03
DFLTD	Morgan city, LA, USA	RC-1	Closed-end	Concrete	Circular	30.3	0.76	4893			0.82	5.78
Flynn (2014)	Avonmouth	TP1	Closed-end	Concrete	Circular	8.8	0.38	1134			0.78	7.34
Flynn (2014)		P3792	Closed-end	Concrete	Circular	11	0.42	1055			0.84	4.16
Flynn (2014)	Dagenham	D1	Closed-end	Concrete	Circular	7.7	0.38	2421			0.81	7.16
Flynn (2014)	Erith	E3	Closed-end	Concrete	Circular	11	0.38	1754	2029	1822	0.76	5.53
Flynn (2014)	Pontarddulais	P1	Closed-end	Concrete	Circular	8.5	0.38	940	517	527	0.9	5.46
Flynn (2014)		R1	Closed-end	Concrete	Circular	6	0.38	1592	1605	1654	0.84	5.15
Flynn (2014)	Ryton	R2	Closed-end	Concrete	Circular	7	0.38	1609	1863	1768	0.88	4.4
Flynn (2014)		R3	Closed-end	Concrete	Circular	5.5	0.38	1761	1450	1609	0.66	6.6
Flynn (2014)	Shotton	S1	Closed-end	Concrete	Circular	5.8	0.38	2200	1693	1559	0.75	8.85
ZJU-ICL	Baghdad University, Iraq	P2	Closed-end	Concrete	Octagonal	19.5	0.61	3292	3479	4537	0.86	4.51
DFLTD	Cimarron, OK, USA	OC-2	Closed-end	Concrete	Octagonal	19.6	0.61	3336			0.87	4.33
DFLTD	Annacis, NA, CAN	OC-1	Closed-end	Concrete	Octagonal	29.5	0.61	1933			0.79	4.34
DFLTD	Battery creek, SC, USA	OC-1	Closed-end	Concrete	Octagonal	24.1	0.61	2444			0.65	7.49
DFLTD		OC-2	Closed-end	Concrete	Octagonal	19.5	0.61	4648			0.37	11.8
ZJU-ICL	British Columbia, Canada	P1	Closed-end	Steel	Circular	45	0.61	3750	3885	4010	0.81	14
ZJU-ICL		H-12	Closed-end	Steel	Circular	6.1	0.46	2082	1317	881	0.83	9.75
ZJU-ICL	Ogeechee River, USA	H-13	Closed-end	Steel	Circular	8.9	0.46	2640	1873	2026	0.68	11.1
ZJU-ICL		H-14	Closed-end	Steel	Circular	12	0.46	3264	2181	2248	0.86	7.45
ZJU-ICL		H-15	Closed-end	Steel	Circular	15	0.46	4010	2729	2848	0.64	16.1
ZJU-ICL	Hoogzand, The Netherlands	2-C	Closed-end	Steel	Circular	6.8	0.36	2805	2379	2826	0.89	2.94
ZJU-ICL	Hunter's point, USA	S	Closed-end	Steel	Circular	9.2	0.27	445	520	493	0.94	1.82
ZJU-ICL		3/1	Closed-end	Steel	Circular	14.2	0.31	1125	2000	2049	0.84	2.96
ZJU-ICL	Locks and Dam, USA	3/4	Closed-end	Steel	Circular	14.4	0.36	880	2430	2615	0.73	2.73
ZJU-ICL		3/7	Closed-end	Steel	Circular	14.6	0.41	1573	2827	3188	0.84	3.91
ZJU-ICL	Hsin-Ta, Taiwan, China	TP4	Closed-end	Steel	Circular	34.3	0.61	4244	3175	3454	0.85	7.98
ZJU-ICL	Cimarron River, USA	P1	Closed-end	Steel	Circular	19	0.66	3557	3711	4622	0.95	4.39
ZJU-ICL	Sermide, Italy	S	Closed-end	Steel	Circular	35.9	0.51	5285	3606	4050	0.73	14.3
ZJU-ICL		1	Closed-end	Steel	Circular	6.9	0.36	1500	1773	2064	0.84	4.86
ZJU-ICL	Pigeon River, USA	2	Closed-end	Steel	Circular	7	0.36	1040	1129	1284	0.79	5.18
ZJU-ICL	Lafayette Bridge, USA	MAT2	Closed-end	Steel	Circular	20.3	0.36	2180	1639	1687	0.81	3.16

DFLTD	Omaha, NE, USA	SPC-4	Closed-end	Steel	Circular	20.1	0.32	1246	0.9	1.58
DFLTD		SPC-2	Closed-end	Steel	Circular	36.6	0.46	2313	0.86	4.09
DFLTD	Portland, ME, USA	SPC-3	Closed-end	Steel	Circular	24.5	0.46	2580	0.85	5.51
DFLTD		SPC-4	Closed-end	Steel	Circular	18.2	0.46	1690	0.89	1.94
DFLTD	Orlando, FL, USA	SPC-2	Closed-end	Steel	Circular	27.4	0.32	2224	0.66	7.01
DFLTD		SPC-3	Closed-end	Steel	Circular	53.3	0.32	3336	0.63	12.3
DFLTD	Dubuque, IA, USA	SPC-2	Closed-end	Steel	Circular	30.5	0.36	2847	0.79	4.3
DFLTD		SPC-4	Closed-end	Steel	Circular	24.4	0.36	1779	0.68	8.19
DFLTD	St. Louis, MO, USA	SPC-1	Closed-end	Steel	Circular	26.4	0.36	1423	0.89	3.14
DFLTD		SPC-2	Closed-end	Steel	Circular	18.9	0.36	1068	0.84	2.16
DFLTD	Phoenix, AZ, USA	SPC-3	Closed-end	Steel	Circular	6.85	0.36	3567	0.88	3.6
DFLTD		SPC-4	Closed-end	Steel	Circular	7.56	0.36	4388	0.78	7.35
DFLTD		SPC-1	Closed-end	Steel	Circular	42.8	0.32	3075	0.82	9.13
DFLTD		SPC-2	Closed-end	Steel	Circular	49.1	0.32	2936	0.75	12.1
DFLTD		SPC-3	Closed-end	Steel	Circular	49.1	0.32	2313	0.71	10.3
DFLTD	Milwaukee, WI, USA	SPC-4	Closed-end	Steel	Circular	50.6	0.24	2491	0.82	7.62
DFLTD		SPC-5	Closed-end	Steel	Circular	47.1	0.24	1690	0.96	5.13
DFLTD		SPC-6	Closed-end	Steel	Circular	47.4	0.24	2660	0.62	16.6
DFLTD		SPC-7	Closed-end	Steel	Circular	44.2	0.24	2842	0.56	16.8
DFLTD		SPC-8	Closed-end	Steel	Circular	50.6	0.24	1775	0.87	7.11
DFLTD	San Francisco, CA, USA	SPC-2	Closed-end	Steel	Circular	9.1	0.27	468	0.96	0.62
DFLTD	Harris County, TX, USA	SPC-1	Closed-end	Steel	Circular	42.7	0.61	6719	0.59	13.8
DFLTD	Fort Belvoir, VA, USA	SPC-2	Closed-end	Steel	Circular	20.4	0.51	1779	0.77	5.73

Table A1-2. Selected axial uplift load tests on displacement piles in sand

Database	Test site location	Pile No.	Pile information			D (m)	B (m)	R _{um} (kN)	R _{uc} (kN)		b	a (mm)
			Type	Material	Shape				ICP-05	UWA-05		
ZJU-ICL	Blessington Dublin, Ireland	S5	Open-end	Steel	Circular	7	0.34	967	586	690	0.67	3.41
ZJU-ICL	Horstwalde, Germany	P4D	Open-end	Steel	Circular	17.7	0.71	1280	1071	1213	0.91	3.32
ZJU-ICL	Larvik, Norway	L1	Open-end	Steel	Circular	21.5	0.51	897	489	511	0.68	4.66
ZJU-ICL		L2	Open-end	Steel	Circular	21.5	0.51	940	489	511	0.79	5.51
ZJU-ICL		L3	Open-end	Steel	Circular	21.5	0.51	1085	489	511	0.79	5.81
ZJU-ICL		L4	Open-end	Steel	Circular	21.5	0.51	1067	489	511	0.87	5.47
ZJU-ICL		L5	Open-end	Steel	Circular	21.5	0.51	1080	489	511	0.92	3.09
ZJU-ICL		L6	Open-end	Steel	Circular	21.5	0.51	863	489	511	0.69	5.17
ZJU-ICL		L7	Open-end	Steel	Circular	21.5	0.51	561	489	511	0.72	4.52
ZJU-ICL	Hoogzand, The Netherlands	1-T	Open-end	Steel	Circular	7	0.36	765	562	683	0.73	5.07
ZJU-ICL	Leman BD, North Sea	BD	Open-end	Steel	Circular	38.1	0.66	4800	5382	5350	0.53	16.2
ZJU-ICL	Dunkirk CLAROM, France	CL-T	Open-end	Steel	Circular	11.3	0.32	474	481	527	0.91	2.24
ZJU-ICL		CS-T	Open-end	Steel	Circular	11.3	0.32	395	481	527	0.97	0.6
ZJU-ICL	Dunkirk GOPAL, France	R1-T	Open-end	Steel	Circular	19.3	0.46	1460	1310	1304	0.87	2.51
ZJU-ICL	Dunkirk GOPAL, France	C1-T	Open-end	Steel	Circular	10	0.46	841	945	1047	0.91	3.29
ZJU-ICL	Euripides, The Netherlands	Ia-T	Open-end	Steel	Circular	30.5	0.76	1645	2804	2751	0.8	8.02
ZJU-ICL		Ib-T	Open-end	Steel	Circular	38.7	0.76	7943	7317	6932	0.57	7.91
ZJU-ICL		Ic-T	Open-end	Steel	Circular	47	0.76	11293	10107	9046	0.4	18.4
ZJU-ICL		II-T	Open-end	Steel	Circular	46.7	0.76	9100	10377	9262	0.71	12.1
ZJU-ICL	Hoogzand, The Netherlands	2-T	Closed-end	Steel	Circular	6.8	0.36	915	754	897	0.74	3.13
ZJU-ICL	Locks and Dam, USA	3/2	Closed-end	Steel	Circular	11	0.31	522	554	599	0.91	1.31
ZJU-ICL		3/5	Closed-end	Steel	Circular	11.1	0.36	540	668	740	0.85	1.32
ZJU-ICL		3/8	Closed-end	Steel	Circular	11.1	0.41	800	776	876	0.85	1.38
ZJU-ICL	Hsin-Ta, Taiwan, China	TP5	Closed-end	Steel	Circular	34.3	0.61	2094	2200	1969	0.51	6.96
ZJU-ICL	Baghdad University, Iraq	P1-T	Closed-end	Steel	Circular	11	0.25	572	315	294	0.93	2.66
ZJU-ICL		A(T)	Closed-end	Steel	Circular	8	0.28	87.5	102	99	0.92	1.02
ZJU-ICL		D/A(T)	Closed-end	Steel	Circular	16	0.28	230	230	202	0.84	2.92
ZJU-ICL		E-(T)	Closed-end	Steel	Circular	23.5	0.28	240	452	377	0.76	3.5

Appendix A2. Spearman Rank-Correlation Analyses

Table A2-1. Results of Spearman rank-correlation analyses (driven piles with closed-end under axial compression)

Model factors	N	D/B		B		D_r	
		r -value	p -value	r -value	p -value	r -value	p -value
M_u	52	-0.21	0.08	-0.17	0.16	-0.08	0.54
b	111	-0.23	0.01	-0.08	0.42	-0.11	0.37
a	111	0.2	0.03	0.2	0.04	0.11	0.37

Appendix A3. Copula Analyses for Hyperbolic Parameters

Table A3-1. Results of copula analyses for hyperbolic parameters (driven piles with closed-end under axial compression)

Copula	θ values	AIC values	BIC values
Gaussian	-0.77	-86.3	-83.6
Plackett	0.06	-85.9	-83.3
Frank	-6.85	-80.5	-77.8
No. 16	0.01	-62	-59.3

Draft