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Reliability-based code revision for design of pile foundations: Practice in Shanghai, China

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Abstract

This paper describes how the code for the design of pile foundations in Shanghai, China is revised based on the reliability theory. With quality static load test data, both within-site and cross-site variabilities for design methods of piles in Shanghai are characterized. It is found that the amount of uncertainties associated with the design of piles in Shanghai is less than the typical values reported in the literature. With the partial factors specified in the previous design code, the reliability indexes of piles designed with empirical methods are in the range of 3.08–4.64, while those of piles designed with the load test-based method are in the range of 5.67–5.89. The load factors in the revised local design code have been reduced according to the national design code. As a result, the resistance factors have been increased in the revised code based on a combination of a reliability analysis and engineering judgment. In the revised design code, the reliability level of piles designed with the empirical methods is similar to that in the previous design code; the reliability level of piles designed with the load test-based method is lowered to achieve cost-effectiveness. Partial factors have been suggested for side and toe resistances based on the reliability theory considering their relative importance as well as the uncertainties involved.

Keywords: Pile foundations; Design code; Load and resistance factor design; Reliability

1. Introduction

Although the global factor of safety (FOS) method has been successfully used for decades, its disadvantage is obvious, namely, that the true level of safety is uncertain for a given FOS, as the method does not explicitly consider the level of uncertainty involved in a design. As a result, designs with the same FOS may in fact correspond to different levels of safety. To overcome the limitations of the FOS method, probabilistic methods can be used to explicitly model the uncertainties, through which the safety of a design can be assured by limiting the chances of an unsatisfactory performance to an acceptably low level. In past decades, extensive research was conducted to develop partial factors for the design of pile foundations based on the reliability theory (e.g., Honjo et al., 2002; Phoon et al., 2003; AASHTO, 2007; Ching et al., 2008; Yu et al., 2012).

As part of the worldwide efforts to implement a reliability-based design in geotechnical engineering, resistance factors for

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Table 1
Suggested side and toe resistances in different soil layers.
Adapted from SUCCC (2010).

Layer no.	Soil description	Depth (m)	Driven piles		Bored piles	
			f_s (kPa)	q_t (kPa)	f_s (kPa)	q_t (kPa)
②	Brownish or grayish yellow clay	0–4	15		15	
	Gray clayed silt	4–15	20–40	500–1000	15–30	
	Gray sandy silt	4–15	30–50	1000–2000	25–40	600–800
	Gray silty sand	4–15	40–60	2000–3000	30–45	700–900
③	Very soft gray silty clay	4–15	15–30	200–500	15–25	150–300
	Gray sandy silt or silty sand	4–15	35–55	1500–2500	30–45	800–1000
④	Very soft gray clay	4–20	15–40	200–800	15–30	150–250
⑤ or ⑤$_1$	Gray clay	20–35	45–65	800–1200	40–55	350–650
	Gray sandy silt	20–35	50–70	2000–3500	40–60	850–1250
⑤$_2$	Gray silty sand	20–35	70–100	4000–6000	55–75	1250–1700
⑤$_3$	Gray or dark gray clay	25–40	50–70	1200–2000	45–60	450–750
⑥	Dark green or brownish yellow clay	22–26	60–80	1500–2500	50–60	750–1000
		26–40	80–100	2000–3500	60–80	1000–1200
⑦$_1$	Straw yellow sandy silt or silty sand	30–45	70–100	4000–6000	55–75	1250–1700
⑦$_2$	Gray fine sand with silt	35–60	100–120	6000–8000	55–80	1700–2550
⑧$_1$	Gray silty clay with silty sand	40–55	55–70	1800–2500	50–65	850–1250
⑧$_2$	Gray silty clay interlayed with silty sand	50–65	65–80	3000–4000	60–75	850–1700
⑨	Gray fine, medium or coarse sand	60–100	110–120	8000–10,000	70–90	2100–3000

the design of pile foundations were calibrated in Shanghai, China when the local foundation design code (SUCCC, 2000) was revised in 2000. After ten years of accumulating new data, knowledge and experience, the foundation design code was revised again recently (SUCCC, 2010). Previously, SUCCC (2000) was developed based on the national foundation design code MOC (1989), for which the load factor for a dead load (γ_D) was 1.2 and the load factor for a live load (γ_L) was 1.4. In 2002, the national design code (MOC, 2002) changed the load factors to $\gamma_D = 1.0$ and $\gamma_L = 1.0$, respectively. The mismatch between the local design code and the national design code caused inconvenience to those involved with foundation design in Shanghai; it is also one of the important reasons for the code revision.

A team of experts, including experts in the geotechnical reliability theory and experienced practitioners with sound engineering judgment, carried out the code revision work. For ease of communication, the resistance factors for the design of piles were calculated based on the simple, but sound, reliability theory. The calibrated results were then interpreted with engineering judgment, and the code was revised based on the consensus of all participating parties. The new features of the revised design code include:

(1) Both within-site variability and cross-site variability are calibrated and considered in the design of pile foundations.
(2) The reliability level of the static load test-based method has been assessed and lowered as supported by the reliability theory.
(3) Partial factors are developed based on the reliability theory for side and toe resistances considering their relative importance and the associated uncertainties.

The objective of this paper is to introduce how the resistance factors for the design of piles in Shanghai are revised based on the reliability theory supplemented with engineering judgment. It is hoped that the experience in Shanghai may provide a useful reference for developing and revising reliability-based geotechnical design codes in other regions. This paper is organized as follows. First, the engineering background of the subsurface deposits and piling practices in Shanghai is introduced. Then, the design methods and calibration database are described. Thereafter, the reliability level, corresponding to the existing partial factors, is assessed. Finally, the resistance factors for the design of piles are calibrated based on the reliability theory, and the design code is revised based on a combination of reliability-based calibration results and engineering judgment.

2. Engineering background

Shanghai is located at the deltaic deposit of the Yangtze River on the eastern coast of China. The subsoil of Shanghai is composed of sediments containing clay, silt and sand, resulting from the alternating warm and cold climates and the changes in sea level over the past 3 million years. The elevation of the ground surface is generally 3–5 m above sea level. The depth of the bedrock could be up to 300–400 m. The soil stratum in Shanghai is relatively uniform. Most civil engineering constructions are within a depth of 80 m below the ground surface, and the typical soil layers within such a depth are shown in Table 1. The left column in Table 1 shows the layer numbers used in the local profession. Among the eight layers shown in Table 1, layers ③, ④, and ⑧ are soft soils with low permeability, high compressibility, and low strength. One can refer to Dassargues et al. (1991), Shen and Xu (2011), and Ng et al. (2013) for more information on the geological conditions of the Shanghai soil stratum.

In Shanghai, reinforced concrete driven piles and bored piles are most commonly used to carry structural loads. The application of reinforced concrete driven piles is a cost-effective solution for many projects, but can only provide a relatively small bearing capacity. Bored piles are preferred when the required bearing capacity is large, when the driven piles are hard to install, or when the protection of adjacent buildings, tunnels, and facility pipes is needed. The most common end-bearing strata for reinforced concrete driven and bored piles are soil layer ⑤ and soil layer ⑦ (see Table 1), respectively. When layer ⑤ is locally missing, the end-bearing stratum of the driven piles can also be layer ⑦. In addition to the reinforced concrete driven piles and bored piles, steel pipe piles are also used as driven piles in Shanghai. The focus of this study is on the design of reinforced concrete driven piles and bored piles. For convenience, the reinforced concrete driven piles will simply be called driven piles in the following sections unless otherwise stated.

Three typical pile design methods used in Shanghai are the static load test-based method, the design table method, and the Cone Penetration Test (CPT) method. Let R, Q_D, and Q_L denote the total pile capacity, the dead load, and the live load, respectively. The design equation in Shanghai is written as follows:

$$\frac{R_n}{\gamma_R} = \gamma_D Q_{Dn} + \gamma_L Q_{Ln} \tag{1}$$

where γ_R, γ_D, and γ_L are the partial factors for R, Q_D, and Q_L, respectively, and R_n, Q_{Dn}, and Q_{Ln} are the nominal values for R, Q_D, and Q_L, respectively. In SUCCC (2000), $\gamma_R = 1.6$ is recommended for all design methods in accordance with $\gamma_D = 1.2$ and $\gamma_L = 1.4$, as specified in the national foundation design code MOC (1989). As the mean value for a random variable is equal to the product of its nominal value and bias factor (e.g., Ang and Tang, 1984), the nominal FOS associated with the above set of partial factors can be written as follows:

$$F_{sn} = \frac{R_n}{Q_{Dn} + Q_{Ln}} = \frac{(\gamma_D Q_{Dn} + \gamma_L Q_{Ln})\gamma_R}{Q_{Dn} + Q_{Ln}} = \frac{(\gamma_D + \gamma_L \rho)\gamma_R}{1+\rho} \tag{2}$$

where F_{sn} is the nominal FOS and ρ is the ratio of the live load to the dead load, i.e., Q_{Ln}/Q_{Dn}. Substituting a typical value for ρ of 0.2 into the above equation yields $F_{sn} \approx 2.0$. For comparison, the nominal FOS used in both Hong Kong and South Korea is 3.0 (GEO, 2006; Kwak et al., 2010). Based on a comparison of the design FOS, it seems that the design of piles in Shanghai is less conservative. However, as will be seen later in this study, this conclusion may not be true because FOS is not a consistent measure of safety, as uncertainties involved in the design cannot be explicitly considered in the computed FOS.

3. Design methods under investigation

3.1. Load test-based method

The load test-based method is considered to be the most reliable approach for pile design. When the static load test is used to determine the design capacity, at least 3 piles or 1% of the total number of production piles should be tested. However, if the number of production piles is less than 50, then the number of piles to be tested can be decreased to a minimum of 2. The ultimate pile capacity at the site is chosen as the mean of the measured bearing capacities. In the load test-based method, the piles are initially designed based on the design table method or the CPT-based method. If static load tests show that the bearing capacity of the initially designed piles is not adequate and the design needs to be revised, the bearing capacity of the revised piles should also be measured by static load tests. The load test-based method is often used for important projects or when the site conditions are complex, new construction methods are employed, or new types of piles are used.

3.2. Design table method

As the soil stratum in Shanghai is relatively uniform, the intense construction activities in this small region over the last decades have yielded abundant experience on the side resistance and toe resistance of piles. In the design table method, the bearing capacity of a pile is determined empirically based on the soil profile as follows:

$$R_c = R_s + R_t = U_p \sum f_{si} l_i + q_t A_t \tag{3}$$

where R_c=calculated total capacity; R_s=side resistance; R_t=toe resistance; U_p=perimeter of pile, f_{si}=unit side resistance for the ith layer, l_i=thickness of the ith layer, q_t=unit toe resistance; and A_t=cross sectional area of the pile end. The values for f_s and q_t, as recommended in the local design code, are shown in Table 1. As can be seen in this table, the recommended values for f_s and q_t are in a range of values for each soil layer. To minimize the effect of human judgment, the mid-point values of the recommended ranges are adopted for calculating the bearing capacity when calibrating the model bias factor of the design table method.

3.3. CPT-based method

In the CPT-based method, the bearing capacity of a pile is calculated using the following equation:

$$R_c = R_s + R_t = U_p \sum f_{si} l_i + \alpha_b q_{ct} A_t \tag{4}$$

where α_b=correction factor and q_{ct}=cone tip resistance measured at the pile toe. The equations used to estimate f_{si} and α_b in the CPT-based method are summarized in Table 2. Note that the cross sectional area of the CPT cone used in China is typically 15 cm^2 larger than the 10 cm^2 cone that is commonly used outside of China. To minimize the effect of human judgment in calibrating the CPT-based method, the mean values of the measured shaft and cone toe resistances in each soil layer are used to calculate the bearing capacity of the piles.

Table 2
Equations for estimating f_s and α_b in the CPT method.
Adapted from SUCCC (2010).

Parameters to estimate	Equations
f_s	(1) For soils with a depth less than 6 m, $f_s = 15$ kPa (2) For clay with a depth larger than 6 m, f_s can be estimated using the following equations through cone friction resistance p_s: $f_s = \begin{cases} p_s/20 & p_s \le 1000 \text{ kPa} \\ 0.025p_s + 25 & p_s > 1000 \text{ kPa} \end{cases}$ (3) For silt and sand with a depth larger than 6 m, $f_s = p_s/50$ (kPa) (4) The estimated f_s, based on the CPT method, should not exceed 100 kPa.
α_b	(1) α_b can be calculated using the following equation based on pile length L $\alpha_b = \begin{cases} 2/3 & L \le 7 \text{ m} \\ 5/6 & 7 \text{ m} < L \le 30 \text{ m} \\ 1 & L > 7 \text{ m} \end{cases}$ (2) The estimated toe resistance, based on the CPT method, should not exceed 8000 kPa.

4. Uncertainty analysis of design methods

The uncertainties in the characterization of the subsurface, the input parameters, and model errors may all affect pile capacity predictions. Traditionally, a global model bias factor is often applied to the bearing capacity equation in order to consider the lumped effects of the aforementioned uncertainties. The statistics of the model bias factor are determined through a comparison of the calculated and the measured bearing capacities of a large number of piles (e.g., McVay et al., 2000; Paikowsky et al., 2004; Kwak et al., 2010). The advantage of this technique is that it can conveniently utilize the information from a large number of full-scale load tests for a future design. The disadvantage of the global model bias factor approach is that the effect of uncertainties from different sources cannot be separately considered. In addition, when calibrating the model bias factor, the measured capacity could also be uncertain due to the existence of multiple criteria for interpreting the ultimate bearing capacity (Zhang et al., 2005; Haldar and Sivakumar Babu, 2008). The uncertainty in the failure criteria is seldom considered in the global model bias factor approach. Recently, a soil variable-based approach has been suggested for developing resistance factors for pile design (e.g., Basu and Salgado, 2012). The approach stems from a separate consideration and characterization of parameter and model uncertainties in predicting the pile capacity. The soil-variable approach requires very specific data for a detailed uncertainty characterization. Roberts and Misra (2010) suggested that the resistance factor be developed based on the *t–z* model, and that the statistics on the uncertain parameters of the *t–z* model be determined through a back analysis of site-specific load test data.

In this study, the global model bias factor method is adopted. As noted in Zhang et al. (2004), when an empirical relationship, constructed based on a regional database, is used for a site-specific prediction, the prediction is subjected to two types of uncertainties, i.e., (1) the within-site variability and (2) the cross-site variability. Different from previous studies, the uncertainty in the model bias factor is decomposed into two components in this study, such that the within-site variability and the cross-site variability can be modeled separately. As will be seen later, the separate characterization of within-site and cross-site variabilities is very important for assessing the reliability of the load test-based method for the design of piles.

4.1. Probabilistic model

To model the within-site variability and the cross-site variability explicitly, it is assumed that the actual bearing capacity of a pile R is related to the calculated bearing capacity, R_c, as follows:

$$R = NR_c = N_1 N_2 R_c \tag{5}$$

where N_1 = bias factor accounting for the within-site variability; N_2 = bias factor accounting for the cross-site variability; and N = lumped model bias factor. The within-site variability is mainly caused by a variation in the soil properties within a site and by the construction errors associated with the site-specific workmanship. The cross-site variability is mainly caused by the regional variation in soil properties and by the construction errors associated with the workmanship in a region. Let λ_{R1} and λ_{R2} denote the mean values of N_1 and N_2, respectively. As the uncertainties associated with N_1 and N_2 are from different sources, it might be reasonable to assume that N_1 and N_2 are statistically independent. With such an assumption, it can be shown that

$$\lambda_R = \lambda_{R1}\lambda_{R2} \tag{6}$$

$$\mathrm{COV}_R = \sqrt{\mathrm{COV}_{R1}^2 + \mathrm{COV}_{R2}^2} \tag{7}$$

where λ_R and COV_R are the mean and COV of N, respectively.

Table 3
Summary of driven piles used for code calibration.

Site no.	Site name	Pile dimensions (mm)	End-bearing stratum	No. of piles	Measured ultimate bearing capacity (kN)	COV of measured bearing capacity
1	Shanghai Jiangping High School	250 × 250	⑤$_1$	6	558; 496; 558; 558; 558; 550	0.046
2	Tangzhen Commodity Housing	250 × 250	⑤$_1$	3	765; 720; 810	0.083
3	Shanghai F1 Speedway	250 × 250	⑤$_1$	3	980; 900; 1000	0.055
4	Yueda Residential Building	300 × 300	⑤$_1$	3	462; 594; 600	0.141
5	Shenyuan Construction Site #1	200 × 200	④	3	500; 450; 425	0.083
6	Shenyuan Construction Site #2	250 × 250	④	4	750; 780; 720; 636	0.086
7	Kangtai Residential Building	200 × 200	⑤	4	460; 440; 396; 480	0.081
8	Shanghai Haide Apartment	250 × 250	④	3	240; 270; 210	0.125
9	Donglan Xincun	250 × 250	⑤$_2$	8	700; 770; 840; 770; 700; 770; 840; 630	0.096
10	Xincheng Residential Building	250 × 250	⑤$_1$	5	540; 540; 600; 540; 540	0.049
11	Rongnanyuan Extension	250 × 250	⑤$_1$	11	403; 403; 403; 403; 403; 403; 403; 403; 403; 403; 358	0.034
12	Qingchi Project Phase I	250 × 250	④	3	567; 491; 567	0.081
13	Shanghai Dekui Plant	250 × 250	⑤	8	461; 461; 461; 461; 461; 461; 461; 403	0.045
14	2205 Gonghexin Rd	250 × 250	④	3	736; 736; 644	0.075
15	Shanghai Tennis Club	300 × 300	⑤	3	720; 600; 720	0.102
16	Jinda Residential Building	300 × 300	⑦$_1$	14	786; 672; 784; 896; 672; 896; 672; 784; 784; 784; 1008; 1008; 1008; 1008	0.155
17	Runjiang Project Phase I	300 × 300	⑤	3	448; 538; 538	0.102
18	Wenhuayuan Phase V	350 × 350	⑦$_1$	3	1550; 1639; 1806	0.078
19	Shanghai Dingxin Apartment	350 × 350	⑦$_2$	8	1280; 1440; 1280; 1280; 1280; 1280; 1600; 1600	0.106
20	ECNU Middle School	350 × 350	④	3	630; 720; 717	0.074
21	ECNU Science Park	350 × 350	⑤	6	1079; 960; 960; 960; 969; 969	0.048
22	Feilipu Plant	400 × 400	⑤	3	1425; 1425; 1350	0.031
23	Site #2	–	⑤	3	1250; 1200; 1500	0.122
24	Site #4	–	⑤	3	2700; 2300; 2600	0.082
25	Site #7	–	⑤	4	470; 560; 440; 420	0.131
26	Site #9	–	⑤	3	890; 1050; 1100	0.108
27	Site #14	–	⑤	3	1300; 1520; 1550	0.094
28	Site #19	–	–	4	–	0.060
29	Site #24	–	–	3	–	0.080
30	Site #30	–	–	4	–	0.103
31	Site #31	–	–	4	–	0.122
32	Site #32	–	–	4	–	0.097

4.2. Calibration database

As will be shown later, the within-site variability can be characterized by comparing the bearing capacities of piles within a site, while the cross-variability can be characterized by comparing the measured and the predicted bearing capacities of piles from different sites. To characterize the within-site variability, only sites with at least three static load tests on piles with identical geometry are incorporated in the calibration database. For the driven piles, the load test data of 111 driven piles from 22 sites are collected. All these piles are square with width in the range of 250–400 mm. In developing the previous local design code SUCCC (2000), another database was also compiled to calibrate the pile design methods in Shanghai. This database is reviewed using the current criterion, and data on 35 piles from 10 sites are found from this database. These two sources of data are then combined to yield a new database that consists of 146 piles from 32 sites. Details on these 146 piles are summarized in Table 3.

For the bored piles, a database comprising 37 piles with diameters in the range of 550–900 mm from 10 sites are compiled, as summarized in Table 4. The number of bored piles in the calibration database is smaller than that of driven piles because bored piles usually have much larger bearing capacities, and hence, it is more expensive to conduct static load tests on them.

4.3. Characterization of within-site variability

As within-site variability refers to the variability in the pile capacity values within a site, it is unbiased (Zhang et al.,

Table 4
Summary of bored piles used for code calibration.

Site no.	Site name	Diameter (mm)	End-bear- ing stratum	No. of piles	Measured bearing capacity (kN)	Statistics of measured bearing capacity		Predicted bearing capacity (kN)
						Mean (kN)	COV	
1	Zhongjian Mansion	850	⑦$_2$	3	8192; 8192; 7168	7850	0.075	9717
2	Metro Line 10 Parking Lot 1	600	⑦$_1$	3	4008; 5025; 3685	4239	0.131	3900
3	Metro Line 10 Parking Lot 2	600	⑦$_1$	4	3900; 3600; 3600; 3300	3600	0.068	2965
4	Yuyuan Community in Jingan District	550	⑦$_2$	6	2420; 2420; 2240; 1960; 2240; 2800	2347	0.119	2140
5	Gaofufang	700	⑧$_2$	3	8840; 8840; 7480	8387	0.094	7572
6	Shanghai Quality Inspection	650	⑥	3	1860; 1780; 1670	1770	0.054	2218
7	Taihongxinyuan	600	⑦$_1$	5	2700; 2700; 3000; 2700; 2700	2760	0.049	2830
8	Yueyang Plaza	850	⑨	3	10,000; 8000; 11,500	9833	0.179	11,529
9	Zhongsheng Commercial Center	900	⑦$_2$	3	5460; 5460; 4550	5157	0.102	4078
10	Shanghai Chunguang Garment Factory	600	⑦$_2$	4	2400; 2400; 2400; 2700	2475	0.061	3273

2004), i.e., $\lambda_{R1}=1$. The COV of N_1, i.e., COV_{R1}, can be determined by calculating the COV of the bearing capacity of the piles within a site. Take Site 1 in Table 3 as an example. There are six measurements for the bearing capacity of piles of the same geometry at this site. Based on the method of moments (Ang and Tang, 2007), the COV of the measured bearing capacities is 0.046, which is regarded as the COV of the within-site variability at this site (COV_{R1}). Applying this procedure to the measured bearing capacity data at each site in Table 3, the values for COV_{R1} at each site are determined, as summarized in the last column of Table 3. The values for the COV_{R1} of driven piles vary from site to site between 0.031 and 0.155 with a mean of 0.087 and a standard deviation of 0.031. Based on the data in Table 4, the values for the COV_{R1} of bored piles are in the range of 0.049–0.179 with a mean of 0.093 and a standard deviation of 0.046. The bearing capacity of bored piles has slightly larger within-site variability as it is more affected by the construction details. In this study, $\text{COV}_{R1}=0.087$ and $\text{COV}_{R1}=0.093$ are adopted for the analysis of driven and bored piles, respectively.

4.4. Characterization of cross-site variability

4.4.1. Load test-based method

As the load test-based method is based on the measured data within a site only, it is free from the cross-site variability, i.e., $\lambda_{R2}=1$, $\text{COV}_{R2}=0$.

4.4.2. Design table method

The cross-site variability is evaluated by comparing the measured bearing capacity with the predicted bearing capacity across different sites. To minimize the effect of within-site variability, a bearing capacity ratio is first obtained for each site by dividing the mean of the measured bearing capacity at this site by the calculated bearing capacity. The mean and the COV of the bearing capacity ratios of different sites determined in such a way are regarded as estimates of λ_{R2} and COV_{R2}, respectively. As an example, consider Site 1 in Table 4. At this site, the mean of the measured bearing capacity is 7850 kN. Based on the design table method, the computed bearing capacity is 9717 kN. Comparing the mean of the measured bearing capacity with the computed bearing capacity yields a bearing capacity ratio of 0.808. With such a procedure, we can calculate a bearing capacity ratio for each site, yielding 10 values for the bearing capacity ratio. Using the method of moments (Ang and Tang, 2007), the mean and the COV of the 10 values for the bearing capacity ratio are 0.996 and 0.184, respectively, i.e., $\lambda_{R2}=0.996$ and $\text{COV}_{R2}=0.184$.

During the development of the previous code for the design of driven piles in Shanghai, the design table method was found to be associated with a mean bias of 1.025 and a COV of 0.144 (SUCCC, 2000). These results have been reviewed and judged as being representative of the current study. Thus, $\lambda_{R2}=1.025$ and $\text{COV}_{R2}=0.144$ are adopted for the design table method for driven piles. The piles in Table 3 are not used for characterizing the cross-site variability; and hence, the bearing capacity of these piles are not calculated during the code revision process. As expected, the design of bored piles is associated with larger cross-site variability.

4.4.3. CPT-based method

During the development of the previous design code for driven piles in Shanghai, the CPT method was found to have a bias factor with a mean of 1.006 and a COV of 0.093 (SUCCC, 2000). These results have been reviewed and judged as being representative of the current study. Thus, $\lambda_{R2}=1.006$ and $\text{COV}_{R2}=0.093$. For the bored piles, as no consensus can be achieved regarding whether or not the CPT-based method should be used, no attempt has been made to calibrate the CPT-based method for the design of bored piles.

4.5. Lumped uncertainty

Based on the characterized within-site variability and the cross-site variability, the statistics of the lumped variability, i.e., λ_R and COV_R, are calculated based on Eqs. (6) and (7) for each design method, as summarized in Table 5. For the empirical methods, the mean bias factor is in the range of

0.996–1.025 and the COV of the bias factor is in the range of 0.127–0.206. For the load test-based method, the COV of the bias factor for the driven and bored piles are 0.087 and 0.093, respectively. Among the methods considered, the load test-based method has the smallest COV, as it is free from cross-site variability. For comparison, the design table method has the largest uncertainty. For the driven piles, the within-site variability is comparable to the cross-site variability for both the design table method and the CPT-based method. In such a case, it is unconservative to ignore the within-site variability. For the bored piles, the cross-site variability dominates the total uncertainty when the design table method is used.

As a comparison, Table 6 summarizes the uncertainties associated with different methods for the design of driven piles and bored piles reported in the literature. Comparing Tables 5 and 6, we can see that the amount of uncertainties associated with the design methods in Shanghai is less than that reported in the literature, such as the methods for the design of bored piles in Hong Kong (Zhang and Chu, 2009). In Hong Kong, the stratum is often made up of residual materials, such as completely decomposed granite (CDG) or completely decomposed volcanic (CDV) soil with a large amount of variability. The relatively uniform stratum in Shanghai may be responsible for the smaller amount of uncertainties associated with the design methods in Shanghai.

Table 5
Calibrated uncertainties associated with different design methods.

		Driven piles			Bored piles	
		Load test	Design table	CPT	Load test	Design table
Within-site variability	λ_{R1}	1	1	1	1	1
	COV_{R1}	0.087	0.087	0.087	0.093	0.093
Cross-site variability	λ_{R2}	1	1.025	1.006	1	0.996
	COV_{R2}	0	0.144	0.093	0	0.184
Lumped uncertainty	λ_R	1	1.025	1.006	1	0.996
	COV_R	0.087	0.168	0.127	0.093	0.206

5. Reliability of piles designed with existing partial factors

5.1. Relationship between partial factors and reliability index

In the design of pile foundations, the performance function is expressed as

$$Z = R - Q_D - Q_L \tag{8}$$

Based on a first order second moment (FOSM) approximation and assuming FOS follows the lognormal distribution, reliability index β can be calculated as (e.g., Withiam et al., 2001)

$$\beta \approx \frac{\ln\left(\mu_{F_S}\sqrt{\frac{1+COV_Q^2}{1+COV_R^2}}\right)}{\sqrt{\ln\left[\left(1+COV_R^2\right)\left(1+COV_Q^2\right)\right]}} \tag{9}$$

where μ_{Fs}=mean FOS, i.e., μ_R/μ_Q; Q=the total load, i.e., Q_D+Q_L; COV_R=COV of R; and COV_Q=COV of Q. In this study, COV_Q is calculated as follows:

$$COV_Q = \frac{1}{1+\rho}\sqrt{COV_D^2+\rho^2 COV_L^2} \tag{10}$$

It should be noted that the reliability of a pile may be evaluated more accurately using methods like a Monte Carlo simulation or the first order reliability method (FORM) (e.g., Ang and Tang, 1984). As mentioned previously, the code revision committee was comprised of both experts in geotechnical reliability and professionals without much reliability background. For ease of communication among different parties, the simple FOSM approximation was used in the code revision process.

Based on Eq. (1), resistance factor γ_R can be related to μ_{Fs} using the following equation:

$$\gamma_R = \frac{R_n}{\gamma_D Q_{Dn}+\gamma_L Q_{Ln}} = \frac{\frac{\mu_R}{\lambda_R}}{\frac{\gamma_D Q_{Dn}+\gamma_L Q_{Ln}}{\lambda_D Q_{Dn}+\lambda_L Q_{Ln}}\mu_Q}$$
$$= \frac{\mu_{Fs}(\lambda_D Q_{Dn}+\lambda_L Q_{Ln})}{\lambda_R(\gamma_D Q_{Dn}+\gamma_L Q_{Ln})} = \frac{\mu_{Fs}(\lambda_D+\lambda_L\rho)}{\lambda_R(\gamma_D+\gamma_L\rho)} \tag{11}$$

where λ_D and λ_L=bias factors of the dead load and live load, respectively.

Combining Eqs. (9) and (11), the relationship between the reliability index and resistance factor γ_R is as follows:

$$\beta \approx \frac{\ln\left[\frac{\lambda_R\gamma_R(\gamma_D+\gamma_L\rho)}{\lambda_D+\lambda_L\rho}\sqrt{\frac{1+COV_Q^2}{1+COV_R^2}}\right]}{\sqrt{\ln\left[\left(1+COV_R^2\right)\left(1+COV_Q^2\right)\right]}} \tag{12}$$

According to the national code for the reliability based design in China (MOC, 2001), COV_D=0.07, COV_L=0.29, and λ_D=1.0. The value for λ_L, however, depends on several parameters, such as the type of building and the height of the building. For simplicity, it is judged that λ_L=1.0 is representative. Table 7 compares the load statistics used in the present study and typical load statistics used in the literature. The values adopted for λ_D, λ_L, and COV_D in this study are lower than those used in the literature. The value adopted for COV_L is greater than those used in other studies. Overall, the load statistics used here are closest to those suggested in Ellingwood and Tekie (1999).

For the case in which the values for COV_R and COV_Q are both less than 0.3 and are of similar magnitudes, the following two approximations are valid:

$$\sqrt{\frac{1+COV_Q^2}{1+COV_R^2}} \approx 1.0 \tag{13}$$

$$\ln\left[\left(1+COV_R^2\right)\left(1+COV_Q^2\right)\right] \approx COV_R^2+COV_Q^2 \tag{14}$$

Substituting Eqs. (13) and (14) into Eq. (12), the following simplified relationship between γ_R and β can be obtained:

$$\beta \approx \frac{\ln\left[\frac{\lambda_R\gamma_R(\gamma_D+\gamma_L\rho)}{\lambda_D+\lambda_L\rho}\right]}{\sqrt{COV_R^2+COV_Q^2}} \tag{15}$$

Table 6
Summary of model bias factors of different methods reported in the literature.

	Design method	No. of cases	λ_R	COV_R	References
Driven piles	Alpha method, clay type I[a]	–	1.104	0.208	Sidi (1985), Barker et al. (1991), Zhang et al. (2001)
	Alpha method, clay type II[a]	–	2.340	0.568	Sidi (1985), Barker et al. (1991), Zhang et al. (2001)
	Beta method	–	1.032	0.213	Sidi (1985), Barker et al. (1991), Zhang et al. (2001)
	Cone penetration test method	–	1.030	0.360	Orchant et al. (1988), Barker et al. (1991), Zhang et al. (2001)
	Lambda method, clay type I	–	1.020	0.414	Sidi (1985), Barker et al. (1991), Zhang et al. (2001)
	Meyerhof's standard penetration test method	–	1.300	0.500	Orchant et al. (1988), Barker et al. (1991), Zhang et al. (2001)
Bored piles	O'Neill and Reese (1999)	11	0.59	0.65	Zhang and Chu (2009)
	Hong Kong Beta method	17	1.03	0.31	Zhang and Chu (2009)
	Method based on correlation with SPT-N	11	0.97	0.45	Zhang and Chu (2009)
	Method based on correlation with vertical effective stress	17	0.96	0.56	Zhang and Chu (2009)

[a]*Note*: Type I refers to soils with undrained shear strength $S_u < 50$ kPa; Type II refers to soils with $S_u > 50$ kPa.

Table 7
Typical load statistics used in different studies.

λ_D	λ_L	COV_D	COV_L	References
1.00	1.05	0.10	0.18	Ellingwood et al. (1980)
1.05	1.0	0.1	0.25	Ellingwood and Tekie (1999),
1.08	1.13	1.15	0.18	Nowak (1999) and AASHTO (2007)
1.03–1.05	0.08–0.10	1.1–1.2	0.18	Nowak (1994) and FHWA (2001)
1.00	1.00	0.07	0.29	Present study based on MOC (2002)

Table 8
Reliability index of piles designed with partial factors recommended in SUCCC (2000).

Load ratio	Driven piles			Bored piles	
	Load test	Design table	CPT	Load test	Design table
$\rho=0.2$	5.89	3.82	4.64	5.67	3.08
$\rho=0.4$	5.37	3.73	4.41	5.21	3.05

Due to easy access to all participating parties, Eq. (15) is used in the code revision process to interpret the reliability level of piles designed with the partial factors specified in SUCCC (2000), in which $\gamma_R=1.6$, $\gamma_D=1.2$, and $\gamma_L=1.4$ are adopted.

5.2. Reliability level corresponding to existing partial factors

As mentioned previously, the values for λ_L and λ_D are both 1.0. The values for λ_R and COV_R for different methods are summarized in Table 5. To evaluate the reliability index corresponding to the partial factors adopted in SUCCC (2000), COV_Q needs to be assessed. It is related to the ratio of live load to dead load [see Eq. (10)]. A typical value of $\rho=0.2$ is first used. Based on Eq. (10), when $\rho=0.2$, $COV_Q=0.076$. Substituting the value for COV_R of each design method into Eq. (15), the corresponding reliability index can be obtained, as summarized in Table 8. Also shown in this table are the reliability indexes corresponding to design methods calculated based on $\rho=0.4$. The results obtained based on $\rho=0.2$ and $\rho=0.4$ are similar, indicating that the reliability index is not very sensitive to the value of ρ adopted. This phenomenon was also noticed in previous studies, such as McVay et al. (2000). Thus, the following discussion will be based on the results obtained using $\rho=0.2$.

Among the methods considered, piles designed with the load test-based method have the largest reliability index, i.e., 5.89 for driven piles and 5.67 for bored piles. This is consistent with the view in the profession that the load test-based method is the most reliable. Piles designed with the design table method have the smallest reliability indexes, i.e., 3.82 for driven piles and 3.08 for bored piles. The reliability index of a bored pile designed with the CPT-based method is 4.64. As a comparison, Phoon et al. (2003) reported that the reliability index of existing drilled shafts under compression, to support transmission line structures in North America, is in the range of 2.4–3.6. Zhang and Chu (2009) noticed that the reliability indexes of large-diameter bored piles in Hong Kong are in the range of 1.61–2.90 for piles in soils and in the range of 2.29–3.11 for piles in rocks if a FOS=3.0 is adopted. Kwak et al. (2010) noted that reliability indexes of driven steel piles in South Korea are in the range of 1.5–2.9 when the design FOS is in the range of 3.0–5.0. Note that the implied design FOS in SUCCC (2000) is 2.0. Although a smaller FOS is adopted in Shanghai, the reliability level is indeed higher as the amounts of uncertainties associated with the design methods in Shanghai are fewer. This further illustrates the limitation of the FOS design approach, i.e., the same FOS does not imply the same level of safety.

5.3. Determination of target reliability index

The national unified reliability design code in China (MOC, 2001) specifies the target reliability index of civil engineering structures in China. According to MOC (2001), the target reliability index against brittle failure mode and ductile failure mode for conventional structures should be 3.7 and 3.2, respectively. In Shanghai, the bearing capacity of a pile is measured according to the national code for testing building foundation piles (MOC, 2003). If there is an abrupt change in the slope of the load-settlement curve, the ultimate bearing capacity is chosen at the point where the abrupt change in slope occurs. In such a case, the failure mode is brittle. If an abrupt change in the slope of the load settlement curve cannot be observed, the ultimate bearing capacity is determined based on a settlement threshold, i.e., 40 mm for piles with diameter D of less than 800 mm and 5% D for piles with diameter D of larger than 800 mm. As excessive settlement (i.e., exceeding the settlement threshold) often indicates failure, it may also be interpreted as a brittle failure mode. Hence, a target reliability index of 3.7 is adopted for the design of driven piles and bored piles in Shanghai. Choosing a reliability index of 3.7 also seems to be more consistent with the reliability level associated with the piles designed using the partial factors, as specified in SUCCC (2000) and summarized in Table 7. For comparison, Meyerhof (1970) suggested that the target probability of failure for foundations should be within 10^{-3} to 10^{-4}, which corresponds to a target reliability index between 3.1 and 3.7. Considering the beneficial group effect and system effect in a pile group (Zhang et al., 2001), Paikowsky et al. (2004) suggested that the target reliability index for a single pile be 3.00 if four or fewer piles are used as a group, and it can be reduced to 2.33 if five or more piles are used as a group.

6. Calibration of partial factors for total capacity

Based on Eq. (12), the resistance factor γ_R required to achieve a target reliability index β_T can be written as follows (e.g., Withiam et al., 2001):

$$\gamma_R = \frac{\lambda_D + \lambda_L \rho}{\lambda_R(\gamma_D + \gamma_L \rho)} \sqrt{\frac{1+\mathrm{COV}_R^2}{1+\mathrm{COV}_Q^2}} \exp\left(\beta_T \sqrt{\ln\left[\left(1+\mathrm{COV}_R^2\right)\left(1+\mathrm{COV}_Q^2\right)\right]}\right) \tag{16}$$

Considering the approximations expressed in Eqs. (13) and (14), Eq. (16) can be further written as follows:

$$\gamma_R = \frac{\lambda_D + \lambda_L \rho}{\lambda_R(\gamma_D + \gamma_L \rho)} \exp\left(\beta_T \sqrt{\mathrm{COV}_R^2 + \mathrm{COV}_Q^2}\right) \tag{17}$$

Due to its simplicity, Eq. (17) is used in the code calibration process to determine the required resistance factor.

In the present study, $\lambda_D = \lambda_L = 1.0$, and $\gamma_D = \gamma_L = 1.0$. The values for λ_R and COV_R of the different methods are summarized in Table 5. As shown previously, the relationship between resistance factor γ_R and reliability index β is not very sensitive to ρ. Thus, a typical value of $\rho = 0.2$ is used. With Eq. (17), the required resistance factor for each design method to achieve $\beta_T = 3.7$ is calculated, as summarized in Table 9. The values for γ_R for the design table method for driven piles, the CPT-based method for driven piles, and the design table method for bored piles are 1.93, 1.72, and 2.26, respectively. To achieve the same target reliability index of 3.7, the design table method for bored piles requires a larger resistance factor as it is associated with more uncertainties. For the load test-based method, the calibrated partial factors based on the reliability theory for driven piles and bored piles are 1.53 and 1.56, respectively. The required resistance factors for the load test-based methods are smaller due to the fewer uncertainties involved.

Table 9
Calibrated and recommended partial factors for the total capacity.

	Load factors	Driven piles			Bored piles	
		Load test	Design table	CPT	Load test	Design table
Calibrated	$\gamma_L = 1.0$ $\gamma_D = 1.0$	1.53	1.93	1.72	1.56	2.26
	$\gamma_D = 1.2$ $\gamma_L = 1.4$	1.24	1.56	1.39	1.26	1.83
Suggested	$\gamma_L = 1.0$ $\gamma_D = 1.0$	1.80	2.00	2.00	1.90	2.00
β Corresponding to the suggested partial factors		5.10	3.90	4.73	5.35	3.14

In the above calculation, $\gamma_D = 1.0$ and $\gamma_L = 1.0$ are adopted according to the national code MOC (2002); they are smaller than the load factors in the previous national code MOC (1989). To study how the load factors affect the resistance factor for design, the resistance factors for different design methods are also calculated assuming $\gamma_D = 1.2$ and $\gamma_L = 1.4$ to achieve the same target reliability index, and the results are also summarized in Table 9. When $\gamma_D = 1.2$ and $\gamma_L = 1.4$, the obtained resistance factors are only about 80% of those calibrated based on $\gamma_D = 1.0$ and $\gamma_L = 1.0$, indicating that a larger resistance factor should be used as the load factors decrease. In the load resistance factor design method studied here, the safety margin is stored in both the load and the resistance factors. As the load factors decrease, a lower safety margin is stored in the load factors; and hence, a larger resistance factor should be adopted.

In principle, the results from a reliability analysis should be adopted in the design code to achieve uniform reliability. In practice, however, the determination of the statistics for resistances and loads both need considerable judgment, which are hard to quantify statistically, particularly when the data are limited in quality or quantity (Allen, 2005). As a result, the calculated reliability index in geotechnical engineering is often not the actual quantity, but an expected reliability index, at best (Gilbert and Tang 1995). On the other hand, there have been certain inherent reliability indexes in the past which have been proven successful. Hence, past experience and engineering judgment should also play a useful role in supplementing the reliability theory for determining the resistance factor. For the design table method and the CPT-based method, the results from a reliability analysis are viewed as the evidence confirming the reliability of the past practice, which has largely helped resolve the doubt in the local profession about whether or not the design of piles in Shanghai is unconservative. The average of the calibrated resistance factors for the design table method for driven piles, CPT-based method for driven piles, and the design table method for bored piles is (1.93+1.72+2.26)/3=1.97. Based on an extensive group discussion, it is recommended that $\gamma_R = 2.0$ be adopted for the three design methods. Using Eq. (15), the reliability level corresponding to the recommended partial factors is also calculated, as summarized in Table 9. Compared with Table 8, the reliability level associated with the recommended resistance factor of $\gamma_R = 2.0$ in SUCCC (2010) is similar to that associated with the resistance factor recommended in SUCCC (2000). This implies that for the empirical methods, the code revision committee intends to maintain a level of reliability similar to that of past practice.

As noticed previously, the reliability indexes of piles designed with the load test-based method are larger than those of piles designed with the design table method or the CPT-based method when the resistance factors recommended in SUCCC (2000) are used. If the reliability associated with the design table method and the CPT-based method is acceptable, the reliability associated with the load test-based method can be lowered. Moreover, it might be prudent not to change the reliability level of the load test-based method too abruptly. Based on an extensive group discussion, it is agreed that $\gamma_R = 1.8$ and $\gamma_R = 1.9$ be adopted, respectively, for the design of driven piles and bored piles based on the load test-based method. A larger resistance factor is adopted for the bored piles because more uncertainties are involved in the design of bored piles. As is also shown in Table 9, for the load test-based methods, the reliability level in SUCCC (2010) is lower than that in SUCCC (2000).

Based on the above discussion, we can see that due to the simplifications made in the reliability analysis and the influence of past experience, the results from the reliability analysis are not directly employed in the revised design code. Such a phenomenon was observed when developing the AASHTO Load and Resistance Factor Design Specifications, as documented in Allen (2005). Nevertheless, the reliability analysis has served as an important decision-making tool for confirming the reliability level of past practice and for supporting the adoption of less conservative resistance factors for load test-based design methods.

7. Calibration of partial factors for resistance components

7.1. Method of calibration

When the side resistance and toe resistance are separately considered, the performance function can be written as follows:

$$Z = R_s + R_t - Q_D - Q_L \tag{18}$$

The corresponding design equation is

$$\frac{R_{sn}}{\gamma_s} + \frac{R_{tn}}{\gamma_t} = \gamma_{QD} Q_{Dn} + \gamma_{QL} Q_{Ln} \tag{19}$$

where γ_s and γ_t are the partial factors for R_s and R_t, respectively; and R_{sn} and R_{tn} are the nominal values for R_s and R_t, respectively.

Let μ_s, μ_t, and μ_R denote the mean of R_s, R_t, and R, respectively. If the random variables in Eq. (18) are statistically independent and normally distributed, it can be shown that the following relationship is valid (see Appendix A):

$$\gamma_s = \frac{1}{1 - \frac{\xi_s \mathrm{COV}_s^2}{\xi_t \mathrm{COV}_t^2}\left(1 - \frac{1}{\gamma_t}\right)} \tag{20}$$

when $\xi_s = \mu_s/\mu_R$, $\xi_t = \mu_t/\mu_R$, COV_s = COV of the side resistance, and COV_t = COV of the toe resistance. In the literature, Q_D is often modeled as a normal variable (e.g., Ellingwood and Tekie, 1999), and Q_L is often assumed to follow the lognormal or Type 1 extreme value distributions (e.g., Basu and Salgado, 2012). Also, it is hard to justify that R_s and R_t are statistically independent and normally distributed. As such, the relationship between γ_s and γ_t given by Eq. (20) is approximate.

Comparing Eqs. (1) and (19), the following equation is valid:

$$\frac{R_{sn}}{\gamma_s} + \frac{R_{tn}}{\gamma_t} = \frac{R_n}{\gamma_R} \tag{21}$$

Let the nominal values of the side and toe resistances be equal to their mean values, i.e., $R_{sn} = \mu_s$ and $R_{tn} = \mu_t$. Since the value for λ_R is very close to 1, $R_n \approx \mu_R$. Based on the above relationships, Eq. (21) can be further written as follows:

$$\frac{\xi_s}{\gamma_s} + \frac{\xi_t}{\gamma_t} = \frac{1}{\gamma_R} \tag{22}$$

There are two unknowns in Eqs. (20) and (22), i.e., γ_s, and γ_t. Based on these two equations, γ_s and γ_t can be solved as follows:

$$\gamma_s = \frac{\gamma_R\left[\xi_t^{\,2}(1+\eta) - 2\xi_t + 1\right]}{\gamma_R \xi_t^{\,2}(1+\eta) - \xi_t(1+\gamma_R) + 1} \tag{23}$$

$$\gamma_{\mathrm{t}} = \frac{\gamma_R \gamma_s \xi_t}{\gamma_s - \gamma_R(1-\xi_t)} \tag{24}$$

where η is a parameter measuring the relative magnitude of uncertainties in the side and toe resistances as defined below:

$$\eta = \frac{\mathrm{COV}_s^2}{\mathrm{COV}_t^2}. \tag{25}$$

7.2. Calibration and recommended partial factors

As in most static load tests, the side and toe resistances are not measured separately and no attempt is made to develop separate resistance factors for the load test-based method.

Eqs. (23) and (24) indicate that the partial factors for the side and toe resistances depend on ξ_t, η, and γ_R. The recommended values for γ_R for each design method have been introduced in the previous section and are summarized in Table 9. The values for ξ_t and η, however, may differ from one pile to another. Note that η measures the relative magnitude of uncertainties in the side and toe resistances. As the design code intends to hide the uncertainty analysis involved in it, it is decided that a representative value for η be adopted in the code calibration process. To obtain the representative value for η, 57 sets of CPT data from 26 sites in Shanghai are collected. In China, cone friction resistance p_s and cone tip resistance q_{ct} are typically measured at an interval of 10 cm. The statistics for p_s and q_{ct} of a soil layer are calculated based on the values for p_s and q_{ct} measured within this soil layer using the method of moments and without considering the spatial correlation among the measurements. For example, for a soil layer with a thickness of 3 m, there will be 30 measurements for

Table 10
Recommended partial factors for design table method and CPT method.

$\xi_t=\mu_t/\mu_R$	0.05	0.10	0.15	0.20	0.25	0.30	0.35
γ_s	2.09	2.16	2.18	2.13	2.03	1.88	1.73
γ_t	1.08	1.20	1.37	1.61	1.93	2.34	2.83

p_s and q_{ct}. The mean and the COV of p_s and q_{ct} for this soil layer are calculated based on the 30 measurements for p_s and q_{ct}, respectively. Through the relationship between the cone shaft resistance and the side resistance of a pile, as shown in Eq. (4), the COV of the shaft resistance of a pile, COV_s, can be calculated using the first order second moment method based on the statistics for p_s. Similarly, the COV of the toe resistance of the pile, COV_t, can also be calculated based on the statistics for q_{ct}. Applying the above procedure to the 57 sets of CPT data, we can obtain the value for η, which is defined as $\text{COV}_s^2/\text{COV}_t^2$, for each case. It is found that the value for η is in the range of 0.19 to 15.43 with a most-likely value of 2.85. Based on $\eta=2.85$, the values for γ_s and γ_t are calculated with $\gamma_R=2$, as ξ_t varies, as summarized in Table 10. We can see that as ξ_t increases, the value for γ_t also increases. Since an increase in ξ_t indicates that the toe resistance contributes more to the total capacity, it is reasonable to apply a larger resistance factor to the toe resistance in order to maintain the same level of safety. Previously, there was a debate in Shanghai on how the partial factors should be applied to the side and toe resistances separately. The reliability analysis seems to provide a reasonable solution to this problem. The calibrated resistance factors are judged as being reasonable and are incorporated in SUCCC (2010).

8. Summary and conclusions

The research reported in this paper and the findings from it are summarized as follows:

(1) The within-site variability and the cross-site variability associated with three methods for the design of pile foundations in Shanghai are calibrated. For driven piles, the within-site variability is characterized with a COV of 0.087. For bored piles, the within-site variability is characterized by a COV of 0.093. Knowledge about the within-site variability is essential for assessing the reliability of the load test-based method. For the design methods studied in this paper, the mean of the bias factor is in the range of 0.996–1.025, and the COV of the bias factor is in the range of 0.087–0.206. The amounts of uncertainty (in terms of COV) involved in the design of piles in Shanghai are less than the typical values reported in the literature. This is probably because the soil stratum in Shanghai is uniform and subjected to less variation.

(2) Using the partial factors recommended in the previous design code (SUCCC, 2000), the reliability indexes of the piles designed using the design table method and the CPT method are in the range of 3.08–4.64, which are larger than the typical values reported in the literature, indicating that the pile design in Shanghai is more conservative. The reliability indexes of driven and bored piles designed with the load test-based method are 5.89 and 5.67, respectively, which are significantly larger than those designed using empirical methods, confirming that the load test-based method is the least uncertain and the most reliable.

(3) When the load factors are decreased, the resistance factors should be increased to maintain the same level of reliability. In SUCCC (2010), $\gamma_R=2.0$ is recommended for the design table method and the CPT-based method. The reliability level associated with partial factors in SUCCC (2010) for the empirical methods is similar to that in the previous design code. In SUCCC (2010), $\gamma_R=1.8$ and $\gamma_R=1.9$ are recommended for the design of driven and bored piles, respectively, using the load test-based method. The reliability level associated with partial factors in SUCCC (2010) for the load test-based method is lower than that in SUCCC (2000). The reliability analysis has been an important decision-making tool for determining the resistance factors in the revised design code.

(4) Partial factors have been suggested for the design of pile foundations considering the relative importance of the side and toe resistances as well as the relative magnitudes of uncertainties in the side and toe resistances. As the ratio of toe resistance to total capacity increases, the toe resistance contributes more to the total uncertainty, and a larger resistance factor should be applied to the toe resistance to maintain the same level of reliability. The reliability theory has been used to determine partial factors for the side and toe resistances in SUCCC (2010).

It should be noted that the design methods, the characterized uncertainties, and the recommended resistance factors, as discussed in this paper, may be particular to the design of pile foundations in Shanghai only, which has quite a uniform soil stratum with the design and construction of piles under the tight control of local design codes. Due care should be exerted when extrapolating these results to other regions. Nevertheless, the procedure used for code revision and the experience involved could be of interest to the profession for the practical implementation of the reliability-based design of pile foundations, and may also be a useful reference for code revision in other regions.

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Appendix A

Assume the random variables are statistically independent and normally distributed. For a linear performance function, as shown in Eq. (18), the value for R_s at the design point, which is denoted as R_s^* here, can be written as follows (e.g., Ang and

Tang, 1984):

$$R_s^* = \mu_s - \phi_s \beta \sigma_s \tag{A1}$$

$$\phi_s = \frac{\sigma_s}{\sigma_Z} = \frac{\sigma_s}{\sigma_Z} \cdot \frac{\sigma_R}{\sigma_R} = \frac{\sigma_R}{\sigma_Z} \cdot \frac{\sigma_s}{\sigma_R} = \frac{\sigma_R}{\sigma_Z} \cdot \frac{\mu_s \mathrm{COV}_s}{\mu_R \mathrm{COV}_R} \tag{A2}$$

where σ_s = standard deviation of R_s and σ_Z = standard deviation of Z [see Eq. (18)]. Let the nominal value of R_s be equal to its mean value μ_s, i.e., $R_{sn} = \mu_s$. Resistance factor γ_s can then be calculated based on the design point, as follows (e.g., Ang and Tang, 1984):

$$\gamma_s = \frac{R_{sn}}{R_s^*} = \frac{\mu_s}{\mu_s - \phi_s \beta \sigma_s} = \frac{1}{1 - \Delta_s} \tag{A3}$$

where

$$\Delta_s = \beta \mathrm{COV}_s^2 \frac{\sigma_R}{\sigma_Z \mathrm{COV}_R} \xi_s \tag{A4}$$

Similarly, it can be shown that

$$\gamma_t = \frac{1}{1 - \Delta_t} \tag{A5}$$

$$\Delta_t = \beta \mathrm{COV}_t^2 \frac{\sigma_R}{\sigma_Z \mathrm{COV}_R} \xi_t \tag{A6}$$

Comparing Eqs. (A4) and (A6), the following relationship is valid:

$$\Delta_s = \frac{\mathrm{COV}_s^2 \xi_s}{\mathrm{COV}_t^2 \xi_t} \Delta_t \tag{A7}$$

Substituting Eq. (A3) into Eq. (A7) yields Eq. (20).

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