Research Article

s_u-Based Shaft Friction Design Method and Evaluation for Pipe Pile

Bin Huang, 1 Yuting Zhang, 2 Xudong Fu, 3 and Ruiyu Ye 1

1 School of Architecture and Civil Engineering, Huizhou University, Huizhou, China
2 Discipline of Civil, Surveying and Environmental Engineering, The University of Newcastle, Newcastle, Australia
3 School of Civil Engineering, Wuhan University, Wuhan, China

Correspondence should be addressed to Yuting Zhang; yuting.zhang11@uon.edu.au

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The shaft friction in clay is essential to bearing capacity of pipe piles. Reasonable selection of design methods and parameters is very important for offshore piles. This paper focuses on the current popular vertical load design methods based on undrained strength for pipe piles, especially for offshore piles. Based on the database, the accuracy and reliability of various pile design methods based on undrained strength s_u in clay were reviewed. The small-scale model tests were conducted to evaluate s_u-based methods. For the shortcomings of small-diameter piles in the database, a field test of full-scale offshore pile was carried out and analysed. By comparing the calculated and the measured capacity for each method, the reliability of various design methods is evaluated for large-diameter piles. For the design methods based on undrained strength, it demonstrates the importance and reliability of the determination of undrained strength parameters. In view of the vertical loading conditions of offshore large-diameter steel pipe piles, reasonable suggestions for design methods and parameters determination are given.

1. Introduction

At present, the popular methods for the vertical bearing capacity of pile foundations in offshore engineering are either based on the empirical formula of CPT tip resistance or directly related to soil parameters, such as undrained strength, yield stress, sensitivity, internal friction angle, etc. Among the API-00 [1], Fugro-05 [2], ICP-05 [3], and UWA-05 [4] methods which are applicable to sand, all but API-00 are based on CPT. Based on the four CPT-based methods, a unified CPT-based method has been developed by the JIP group of UWA, NGI, Imperial College, Fugro, BP, Delft University of Technology, University of Texas, DNV-GL, and Lloyd’s Register EMEA [5]. The methods applicable to clay include API-00, Fugro-96 [6], NGI-05 [7], ICP-05, UWA-13 [8], and Fugro-10 [9] methods, of which API-00, Fugro-96, and NGI-05 are all based on the undrained strength of clay.

The unified CPT-based method of driven pile in sand has replaced the API-00 method as the preferred recommended method in the new version of the API specification. It is noted that the API recommendations for clay, which have not changed since 1987, are still based on undrained strength and have a lot of experience in practice. On average, base resistance amounted to approximately 20% of the total bearing capacity in clay, especially for open-ended piles, which have a very small proportion of base resistance [10]. Therefore, this paper focuses on the analysis and evaluation of shaft friction design method of pile in clay and compares the undrained strength-based API method with its improved method Fugro-96 and NGI-05 method, especially in the determination of undrained strength parameters.

2. Design Methods for Shaft Friction of Piles

2.1. API-00 Method. The unit shaft friction is calculated from

\[ \tau_f = \alpha \cdot s_u, \]

\[ \alpha = 0.5 \cdot \psi^{-0.5} \quad \text{for } \psi \leq 1.0, \]  \hspace{1cm} (1)

\[ \alpha = 0.5 \cdot \psi^{-0.25} \quad \text{for } \psi > 1.0, \]
where $s_u = \text{undrained share strength at depth } z, \psi = s_u / \sigma_{v0}' = \text{strength ratio at depth } z,$ and $\sigma_{v0}' = \text{effective vertical stress at depth } z.$

### 2.2. Fugro-96 Method

The unit shaft friction is calculated from

$$\tau_f = \alpha \cdot s_u$$

$$\alpha = \min \left[ 0.9 \cdot \left( \frac{L - z}{D} \right)^{-0.2} \cdot \left( \frac{s_u}{\sigma_{v0}'} \right)^{-0.3}, 1.0 \right],$$

(2)

where $s_u = \text{undrained share strength at depth } z,$ $L = \text{pile length},$ $z = \text{depth below ground surface},$ $D = \text{pile outer diameter},$ and $\sigma_{v0}' = \text{effective vertical stress at depth } z.$

### 2.3. NGI-05 Method

The unit shaft friction is calculated from

$$\tau_f = \max \left[ \alpha \cdot s_u \cdot F_{tip}, \beta_{min} \cdot \sigma_{v0}' \right],$$

$$\beta_{min} = 0.06 \cdot \left( I_p - 12 \right)^{0.33}; 0.05 < \beta_{min} < 0.20,$$

(3)

where $s_u = \text{undrained share strength at depth } z,$ $\beta_{min} = \text{minimum } \beta - \text{value observed from previous pile tests},$ $\sigma_{v0}' = \text{effective vertical stress at depth } z,$ $I_p = \text{clay plasticity index (in%)},$ and $\psi = s_u / \sigma_{v0}' = \text{strength ratio at depth } z.$

For $\psi < 0.25,$

$$\alpha = 0.32 \cdot \left( I_p - 10 \right)^{0.3}; 0.2 < \alpha < 1.0,$$

$$F_{tip} = 1.0,$$

(4a)

For $\psi > 1.0$

$$\alpha = 0.5 \cdot \psi^{-0.3},$$

$$F_{tip} = 1.0, \text{ for open - ended},$$

$$F_{tip} = \min \left[ 0.8 + 0.2 \cdot \psi^{0.5}, 1.25 \right], \text{ for closed - ended},$$

(4b)

For $0.25 < \psi < 1.0, \alpha$ is

determined by linear interpolation

(in a semi - logarithmic scale)

between $\psi = 0.25$ and $\psi = 1.0.$

(4c)

### 3. Review of Design Methods for Shaft Friction of Piles

Lehane et al. [11] analysed the radial stress of the jacked pile in clay. The radial stress decreases with the increase of the penetration depth during pile installation. After the installation, the radial stress of the pile shows a power function degradation with the distance from pile tip, and the effective radial stress of the pile after the pore pressure dissipates decreases with the increase of the penetration depth. Gavin et al. [12] suggested that the degree of fatigue degradation in clay is smaller than that in sand. The API-00 method does not consider friction fatigue due to installation. The NGI-05 method is still improved in accordance with the API-00 method, while the Fugro-96 method considers the fatigue degradation of the shaft friction.

### 4. Field Tests of Small-Scale Pile

#### 4.1. Soil Conditions

Field tests of small-scale pile and CPTu were carried out on the Bayswater clay test site which is located roughly 10 km from Perth CBD and next to the Swan River in West Australia (Figure 1). The geological conditions and CPT profile of this site are shown in Figure 2. The soil stratigraphy at the site comprises a relatively thin layer of sand, overlying a deep layer of very soft clay. This clay is associated with a paleochannel and has similar characteristics to offshore clay, making it suitable for both onshore and offshore research.

The CPT piezocone is a cylindrical penetrometer with a 60° conical tip which has a base area of 1000 mm² (35.7 mm diameter) and a friction sleeve with area of 15000 mm². The average rate of penetration is 23 mm/s. The CPT closest to the test area are CPT-5, CPT-6, and CPT-7. The corrected cone resistance ($q_c$), sleeve friction ($f_s$), and pore pressure ($u_2$) are plotted in Figure 2(b) and show the uniformity of the soil conditions. The CPT tip resistance of the clay increases linearly with depth, which also shows that the uniformity of the clay in the test site is very good. The pile diameter is 16.5 cm and the embedded length of pile is 5 m.
which ranges from 1.5 m to 6.5 m deep in clay. The plasticity index of clay is 38%.

The average effective vertical stress of the soil layer within the embedded length of the model pile is 48.5 kPa, and the undrained strength is 19 kPa. This paper uses an empirical method based on CPT to determine the undrained strength of clay, as shown in the following equation:

\[ s_u = \frac{(q_t - \sigma_0)}{N_k} \]  

(5)

\( N_k \) is generally determined by in situ vane shear test or laboratory undrained strength test with high-quality samples. Using \( N_k = 10 \), the calculated averaged undrained strength at 1.5~6.5 m depth is 19 kPa, as shown in Figure 3.

4.2. Field Test of Pipe Piles. After removing the surface ~0.5 m depth soil, an excavator was used to push the piles into the clay to the embedded length as shown in Figure 4(a). In order to avoid any influence of the upper 1 m crust on the pile capacities, a 1 m deep bore was augered and supported by a 190 mm inner diameter casing (see Figure 4(b)). After installation, the buried depth of the pile is 1.5~6.5 m in soft clay, and the embedded length of the pile is 5 m. The soil plug length ratios are 74%~95%. The arrangement employed for all tests is shown in Figure 4(c).

The results of model pile pull-out test are shown in Figure 5. A total of 3 sets of tests were carried out, two piles were tested 3 days after pile driving, and the other was tested 28 days after pile driving. The test results were basically the same. The tension bearing capacity of the pile is equal to the sum of the total shaft resistance and the effective weight of the pile and the soil plug in the pile, so the measured total shaft resistance after deducting the weight should be 40 kN.

4.3. Comparison between Design Methods. The load transfer behavior and the distribution of shaft resistance with depth calculated by different design methods are shown in Figure 6. There is a casing in the depth range of 0~1.5 m to isolate the pile from the sand on the side of the pile, so there is no shaft resistance in this range, and the load transfer does not change. It can be seen that the friction gradually increases with the depth, which obtained by each design method is almost the same.

The comparison between the calculated results of different design methods and the measured results is shown in Table 2. It can be seen that the total shaft resistance

<table>
<thead>
<tr>
<th>Methods</th>
<th>No. of samples</th>
<th>( \mu_w )</th>
<th>( \sigma_w )</th>
<th>( \text{CoV}_w )</th>
<th>( \mu_{gw} )</th>
<th>( \sigma_{gw} )</th>
<th>( \text{CoV}_{gw} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>API-00</td>
<td>23</td>
<td>0.73</td>
<td>0.29</td>
<td>0.40</td>
<td>0.68</td>
<td>1.47</td>
<td>0.47</td>
</tr>
<tr>
<td>Fugro-96</td>
<td>23</td>
<td>0.86</td>
<td>0.24</td>
<td>0.28</td>
<td>0.83</td>
<td>1.32</td>
<td>0.32</td>
</tr>
<tr>
<td>NGI-05</td>
<td>47</td>
<td>0.97</td>
<td>0.36</td>
<td>0.37</td>
<td>0.91</td>
<td>1.43</td>
<td>0.43</td>
</tr>
</tbody>
</table>

Table 1: Method uncertainties for piles in clay [13].

Figure 1: Pile test site location.
calculated by each design method is very close to the measured result.

5. Case Analysis of Full-Scale Offshore Pile

5.1. Soil Conditions. A field test of large-diameter steel pipe piles was carried out in an offshore wind farm project in Pinghu waters of Hangzhou Bay (Figure 7). The site is about 20 km offshore, the seabed topography of the total site area changes little, and the water depth is 8~12 m. As for regional geological information, the local quaternary sediments within the exploration depth, the upper part is the silty clay and clay deposited in the Holocene littoral facies, and the lower part is the silty clay and silty sand deposited in the upper Pleistocene estuary-littoral facies. The pile end is buried 69 m deep and is located in the supporting layer of silty sand.
The stratum distribution and profile of cone tip resistance, sleeve friction, and pore pressure are seen in Figure 8. The base area of CPT piezocone is 1000 mm² (35.7 mm diameter), and the area of friction sleeve is 15000 mm². The rate of penetration is 20 ± 5 mm/s. Two CPT results at test site are almost the same, which shows uniformity of soil conditions. The clays at the depth of 0–45 m are nearly uniform with the tip resistance gradually increasing with depth. The plasticity indices of clays are shown in Figure 8.

5.2. Undrained Strength of Offshore Clay. There is a big difference between marine soil and land soil in sampling, sample transportation, and sample storage conditions. On-
site test conditions for marine soil are limited and are affected by waves, tides, ocean currents, test platforms, instruments, and equipment. Cohesionless soils are generally difficult to obtain core samples, while cohesive soils are easily affected by disturbances such as sampling and transportation. Ladd and Lambe [14] concluded that the soil sampling disturbance makes the undrained strength of the soil sample not equal to the undrained strength under in situ stress. The former is only
**Figure 7:** Pile test site location.

**Figure 8:** Cone penetration test results.
40%~79% of the latter. If the undrained shear test is directly carried out on the marine clay samples transported to the laboratory, the undrained strength would be very low, which leads to excessive conservative design of the offshore engineering and serious waste of cost.

For the test of marine soil samples, the conditions and influence factors of field test and lab test should be fully considered. The test method is more complicated and requires more scientific and rigorous research. Recompression of the sample can reduce the disturbance of the sample caused by sampling and sample processing [15]. The selected isotropic consolidation effective stress for recompression type tests should not equal the in situ vertical effective stress, especially for soft clays which have $K_0$ values well below 1. This would cause too much volumetric change during consolidation and, hence, can result in a measured undrained shear strength that is too high [16]. According to BS EN ISO 19901–8:2015 [17], if the recompression method is used, the specimen should be preloaded to approximately 75%~80% of the estimated preconsolidation stress of the sample, and then unloaded back to the estimated in situ vertical effective stress.

In order to eliminate the influence of sample disturbance, besides the recompression method, there is also a SHANSEP (stress history and normalized soil engineering parameters) method [18]. The SHANSEP method requires preparation of multiple specimens for normal consolidation and overconsolidation triaxial compression test, which is suitable for low sensitivity, unnatural cementation, and cohesive soils with less structured properties.

First of all, a sample disturbance simulation study is carried out for a certain clay sample. The plasticity index of the clay is 21.1%, the dry density of the sample is 1.68 g/cm³, and the water content is 21%. Regarding the prepared remoulded sample as an intact sample, the sample was shaken with a vibrating machine to simulate the influence of disturbance, and the number of shakes was 500 times. Comparing the mechanical properties of the intact sample and the disturbed sample, the shaking disturbance simulated caused more than 8.6% strength loss of the sample.

Take the shallow marine silty clay samples for the study of disturbance treatment. Cut a Φ101 mm × H200 mm sample from a core sample at a depth of 10 m. The effective weight $y'$ of the soil sample is 8 kN/m³, and the internal friction angle $\varphi$ is 12.4° as shown in Figure 9. The static lateral stress coefficient is $K_0 = 1 - \sin \varphi = 1 - \sin 12.4° = 0.785$, and then the average consolidation stress of in situ sample is

$$
\sigma_c = \sigma_{1c} + 2K_0\sigma_{1c}/3
$$

$$
= \frac{y' h}{3} (1 + 2K_0)
$$

$$
= \frac{8 \times 10}{3} (1 + 2 	imes 0.785)
$$

$$
= 68.5 \text{ (kPa)}.
$$

According to the SHANSEP method proposed by Ladd and Foott [18], it is believed that the consolidated undrained shear strength of the soil sample under 1.5~2.5 times the preconsolidation stress can more truly reflect the intact strength characteristics of the site soil. Therefore, a confining pressure of 100 kPa is applied isotropically which is about 1.5 times the in situ consolidation stress of 68.5 kPa. After consolidation, $\Phi39.1 \text{ mm} \times H80 \text{ mm}$ samples are cut from the $\Phi101 \text{ mm} \times H200 \text{ mm}$ sample. Regarding the cut sample as the intact sample, the corresponding preconsolidation stress is 100 kPa.

A vibrating machine is used to shake the $\Phi39.1 \text{ mm} \times H80 \text{ mm}$ samples to simulate the influence of disturbance, and the number of shakes is 500 times. The CU triaxial tests were performed on the disturbance samples, and the confining pressure was 600, 900, and 1200 kPa. Figure 9 shows the total stress strength results of CU triaxial test under high stress relative to the consolidation pressure of 100 kPa.

The UU triaxial test was performed on the intact sample. The results of intact UU and disturbed CU triaxial test are summarized, as shown in Figure 10. The undrained strength in the intact UU triaxial test is 40.9 kPa. According to the Mohr–Coulomb theory in Figure 9, the strength of 40.9 kPa corresponds to the consolidation stress of 62.0 kPa.

In this test, the vibration of the shaking machine is used to simulate the influence of the disturbance on the sample. Regarding the cut sample without vibration as the intact sample, the preconsolidation stress of the intact sample is 100 kPa, and the recompression stress of the disturbance sample is 62.0 kPa, which means that after the disturbed sample is recompressed at

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**Figure 9:** The total stress strength of the disturbed sample in high-stress triaxial test.

**Figure 10:** UU triaxial test with intact sample and CU triaxial test with disturbed samples.
62% of the preconsolidation stress, the same undrained strength can be obtained. This is different from 75% to 80% of the preconsolidation stress of the sample recommended by BS EN ISO 19901–8:2015. It can be seen that the selection of recompression stress is related to the disturbance pattern and degree of the soil disturbance. However, there are certain differences in marine soil sedimentation environments and investigation techniques in different countries, and it is unscientific to blindly apply experiences or standards. Due to the harsh marine environment, borehole sampling is inevitably affected by disturbances, and there are also disturbances in marine transportation and road transportation. In view of the existing geotechnical investigation technology, how to reasonably select the recompression stress to determine the undrained strength of soil requires continuous accumulation of engineering experience, which can be further promoted and applied after engineering verification.

Therefore, the SHANSEP method is more recommended to determine the undrained strength of marine clay. The SHANSEP method requires preparation of multiple specimens for normal consolidation and overconsolidation triaxial compression test, which is suitable for low sensitivity, unnatural cementation, and cohesive soils with less structured properties. The undrained shear strength of normally consolidated or slightly overconsolidated clay is proportional to the consolidation stress, and the undrained shear strength corresponding to the preconsolidation pressure should be adopted. The undrained strength of the intact sample with a depth of 10 m obtained by the SHANSEP method is 40.9 \times 68.5/100 = 28.0 kPa.

Jamiolkowski et al. [19] suggested that for clays of low to moderate plasticity index the undrained strength ratio can be estimated by the following equation:

\[
\frac{s_{u}}{\sigma'_{v0}} = (0.23 \pm 0.04)\text{OCR}^{0.8}. \tag{7}
\]

The OCR at 10 m depth is 1.8; according to equation (7), the undrained strength of the intact sample at 10 m depth is 22.8–32.4 kPa. It can be seen that the undrained strength obtained by the SHANSEP method is reliable.

This paper uses an empirical method based on CPT to determine the undrained strength of clay, as shown in equation (5). Using \( N_k = 20 \), the calculated undrained strength at 9.5–10.5 m depth is 28.0 kPa, as shown in Figure 11. It can be seen that the cone tip resistance of the clay with a depth of 4–50 m increases linearly with depth, which is a relatively uniform soil layer.

5.3. Field Test of Offshore Piles. The pile outer diameter is 1.8 m, the total pile length is 93 m, and the wall thickness is 20–40 mm. The IHC S800 hydraulic hammer is used for pile
Figure 12: Axial tension load-displacement curves.

Table 3: Comparison of measured and calculated results of tension capacity with different design methods.

<table>
<thead>
<tr>
<th>Methods</th>
<th>Shaft resistance ( Q_s ) (MN)</th>
<th>Base resistance ( Q_b ) (MN)</th>
<th>Total resistance ( Q_t ) (MN)</th>
<th>Ratio of calculated and measured capacity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured</td>
<td>14.150</td>
<td>0</td>
<td>14.150</td>
<td>NA</td>
</tr>
<tr>
<td>API-00</td>
<td>18.075</td>
<td>0</td>
<td>18.075</td>
<td>127.7</td>
</tr>
<tr>
<td>Fugro-96</td>
<td>15.672</td>
<td>0</td>
<td>15.672</td>
<td>110.8</td>
</tr>
<tr>
<td>NGI-05</td>
<td>13.410</td>
<td>0</td>
<td>13.410</td>
<td>94.8</td>
</tr>
</tbody>
</table>

Note. The tension capacity measured deducts the weight of pile and soil plug (4.550 MN); the shaft resistance in sand all uses the Unified CPT-based method [5]; the base resistance in tension is not considered.

Figure 13: Load transfer behavior and shaft resistance distribution with depth during ultimate tension bearing calculated by different design methods.
driving. There is no slipping phenomenon during the pile driving process, and the pile embedded depth is 69 m. After the pile driving, the soil plug/core is 1.45 m slightly above the mudline, and the soil plug ratio of the steel pipe pile with an inner diameter of about 1.7 m is 1.0, which is a soil coring mode.

The vertical static tension test was carried out 44 days after the pile was driven using the slow maintained loading method. Figure 12 shows the axial tension test results of the pile. The axial tension test of the single pile achieves failure, and the ultimate axial tension capacity of the single pile is 18.7 MN.

5.4. Comparison between Design Methods. Comparisons of measured and calculated results of capacity with different design methods for tension are shown in Table 3. It can be seen that in tension, the methods based on undrained strength, except for the NGI-05 method, are 11%~28% higher than the measured value.

Mostly with the methods based on the undrained strength, the tension capacity is higher than the actual measured value. This is because the undrained strength in this project is estimated through CPT data, and the parameter $N_k$ is only determined by the SHANSEP method of the disturbed sample at a depth of 10 m. In fact, $N_k$ is related to soil properties and depth. The NGI-05 method based on undrained strength is 94.8% of the measured capacity, which is safer than the API-00 method and Fugro-96 method.

Figure 13 is the friction distribution with depth during ultimate bearing calculated by different methods. Obviously, the shaft friction of clay calculated by the NGI-05 method is the smallest, then Fugro-96 and API-00. But the trends of shaft friction changing with depth are almost the same.

6. Conclusions

This paper introduces and evaluates the $s_u$-based design methods for friction of pipe pile under vertical load which are currently popular in the industry. The main conclusions are as follows:

(a) According to the statistical analysis of the database, the Fugro-96 and NGI-05 method based on the undrained strength are improved compared with API-00 method. NGI-05 method is safer for full scale piles in offshore engineering.

(b) It is difficult for offshore engineering to determine the undrained strength of the intact clay sample. The strength parameters in different soil layers and different depths can be determined by the relatively mature SHANSEP method, but this method is only suitable for low sensitivity, unnaturally cemented, and low structured cohesive soil.

Data Availability

The data used to support the findings of this study are included within the article.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

Acknowledgments

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