SKIN FRICTION OF NON-DISPLACEMENT PILES RELATED TO SIMPLE SHEAR MODE WITH LARGE STRAIN STATE FRICTION ANGLE

Noriyuki Yasufuku\(^a\) and Hidetoshi Ochiai\(^b\)

ABSTRACT

A rational method for evaluating the skin friction of non-displacement piles in sandy soils is presented by reconsidering the model which has already been proposed by the authors. The revision of the model is mainly made by assuming the soil-pile interaction mechanism as a simple direct shear mode in relation to the large strain state of soils. The model is characterized by a coefficient of effective horizontal stress averaged through the whole length of the pile derived as a function of a large strain state friction angle. The characteristics and ability of the model are shown by the parametric studies and the applicability of the model is verified through a database of full scale pile load tests.

Key words: critical state friction angle, direct shear mode, effective stress analysis, non-displacement pile, sandy soil, skin friction (IGC: D6/E4)

INTRODUCTION

The total axial bearing capacity of a pile is derived from the sum of the end bearing resistance and skin friction. However, given the constraint of allowable displacement, in practical designs for a floating pile, the major part of the vertical bearing capacity is usually mobilized from the skin friction, because relatively large displacements are required to mobilize the end bearing capacity. Thus a rational and accurate evaluation of shaft resistance is therefore very important.

There are two main approaches for evaluating pile skin friction: a semi-empirical method using SPT-N values; and a theoretical approach based on geomechanical considerations of the slip failure mode between the pile and the surrounding ground. The former semi-empirical method is widely used in geotechnical codes in Japan for practical evaluation of the pile skin friction. However, since it relies on using previously collected data, it does not directly reflect the individual ground characteristics and produces uncertainty in the resulting analyses. Thus, as highlighted by Randolph et al. (1994), further research work is still needed for the rational evaluation of pile skin friction, including the method of determination of the design parameters. In particular, it is desirable to establish a method for practical applications, taking into account the failure mechanisms and deformation properties of soils.

Non-displacement piles are frequently used in urban areas because of noise and vibration considerations. A method related to the soil characteristics under critical state conditions for the evaluation of the skin friction of a pile with a relatively rough surface has been proposed by Yasufuku et al. (1997, 1999). This was acknowledged by Poulos et al. (2001) as an effective stress approach, such that:

\[
f_t = K \sigma' \tan \phi'_c
\]

\[
K = \left(1 - \frac{z}{L}\right)^\alpha K_0 + \left(\frac{z}{L}\right)^\alpha K_0
\]

where, \(f_t\) is the pile skin friction, \(K\) is the coefficient of horizontal effective stress, \(\sigma'\) is the effective overburden pressure, \(\phi'_c\) is the friction angle at the critical state, \(z\) and \(L\) are the depth and total length of pile respectively, \(K_0\) and \(K_\alpha\) are the coefficients of passive earth pressure and earth pressure at rest and \(\alpha\) is a parameter related to the depositional environment of the ground including stress history and aging effects. This has been empirically determined as 0.2 based on the characteristics of the averaged horizontal effective stresses obtained from pressuremeter tests at pile loading sites (Yasufuku et al., 1997, 2001). The usefulness of this model has been verified through the use of laboratory model pile tests, in-situ friction tests and a database obtained from full scale pile load tests.

Based on the results of simple shear tests by Ochiai (1975) and Ochiai et al. (1983), the model will be extended by considering the empirical parameter \(\alpha\) from a geomechanical point of view, in which the mechanism of soil-pile interaction is one of simple shear. Consequently, the parameter \(\alpha\) will be derived as a function of a large strain state friction angle alone which is roughly regarded as a critical state friction angle in this study. The efficacy of the extended model for practical use are discussed by

\(^a\) Associate Professor, Department of Civil Engineering, Kyushu University, Japan (yasufuku@civil.kyushu-u.ac.jp).
\(^b\) Professor, ditto.

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comparing the predicted results with the data from previous pile load tests. It should be noted that this study is mainly focused on sandy soil and thus the model is effective only for estimating the ultimate shaft friction for fully drained conditions or if the effective stresses are known in the field.

OUTLINE OF THE MODEL PRESENTED

Skin Friction at Any Depth

Skin friction of a pile is generally determined as the sum of pile to soil cohesion and friction components as shown in the following equation:

\[ f_s = c_s + \sigma'_s \tan \phi_s \]  

(2)

where \( f_s \) is the pile skin friction, \( c_s \) and \( \phi_s \) are the adhesion and friction parameters between pile and soil, and \( \sigma'_s \) is the horizontal effective stress acting on the pile.

When using the above equation, it is important to precisely and rationally determine the parameters relating to the soil characteristics and the mobilized effective horizontal stress \( \sigma'_s \). For instance, the maximum skin friction of cast-in-place piles in sandy soil is mobilized after a displacement equal to approximately 2% of the pile diameter as shown by Yasufuku et al. (1997). It was considered from such a result that the mechanism for the mobilization of skin friction was essentially due to shear failure in a thin layer of soil surrounding the pile, which corresponds to a sufficiently large strain level. When assuming such a mechanism, it is reasonable to use the critical state strength parameters such that:

\[ c_s = 0 \]  

(3)

\[ \phi_s = \phi_{sv} \]  

(4)

where, \( \phi_{sv} \) is the critical state friction angle defined as the condition where large shear strains are produced without any change in mean principal effective stress, shear stress or volumetric strain. This friction angle is uniquely determined irrespective of density and overburden pressure, which has advantages for practical use (Yasufuku et al., 1997, 2001).

The horizontal effective stress \( \sigma'_s \) in Eq. (2) is also important in the evaluation of pile skin friction. The mobilization of the skin friction is dependent on the horizontal effective stress \( \sigma'_s \) and this in turn is dependent on the overburden pressure \( \sigma'_o \), as shown in the following equation:

\[ \sigma'_s = K \sigma'_o \]  

(5)

Therefore, the estimation of the horizontal effective stress depends on the evaluation of the coefficient of horizontal effective stress \( K \). For this, it is necessary to consider the characteristics and depositional environment of the soil surrounding the pile, and the interaction between pile and soil. In the case of non-displacement piles, the range of \( K \) values is normally \( K_o \leq K \leq K_t \). In practice, the \( K \) value is generally treated as constant, irrespective of pile length and depth of ground. However, there is evidence that \( K \) values change depending on the effective overburden pressure and pile length (e.g. Vesic, 1970) and a proper estimation of its dependency is important for a better prediction of the pile skin friction. Based on field measurements and experimental data, the characteristics of \( K \) values can be itemised as follows:

1) Near the ground surface the effective overburden stress is small and thus the dilatation which accompanies the shear deformation in a thin shear zone, surrounded by the elastic zone in the ground, is the dominant factor and therefore \( K \) approaches the passive value.

2) The overburden pressure increases with depth from the ground surface. In the end bearing zone, the confinement developed as a result of the dilation due to shear gives at rest earth pressure conditions.

3) The rate of decrease of \( K \) with depth is affected by the depositional environment. In order to reflect these properties in the evaluation of \( K \), the following equation was proposed by Yasufuku et al. (1997, 1999):

\[ K = \left( 1 - \left( \frac{z}{L} \right)^\alpha \right)^{K_o} + \left( \frac{z}{L} \right)^\alpha \]  

(6)

where, \( \alpha \) is a parameter related to the depositional environment, including the stress history and aging effects. The range of \( \alpha \) may be \( 0 \leq \alpha \leq 1 \) although as mentioned before, a value of \( \alpha = 0.2 \) was used based on the results of pressuremeter tests. Based on the assumed failure mode, \( K_f \) has been evaluated as a function of \( \phi_{sv} \) as follows:

\[ K_f = \frac{1 + \sin \phi_{sv}}{1 - \sin \phi_{sv}} \]  

(7)

Assuming \( K_o \) is mobilized under non-plastic deformation conditions (Ochiai, 1976), Wood's (1991) empirical equation:

\[ K_o = (1 - \sin \phi_{sv}) \sqrt{OCR} \]  

(8)

was used to estimate the \( K_o \)-value in Eq. (6), where, OCR in (8) is the overconsolidation ratio, which is defined as the ratio of the current overburden pressure to the compression yield stress in the \( e \)-log \( \sigma'_o \) curve. The OCR is an important factor for evaluating the horizontal effective stresses of soils surrounding the pile as shown in Eq. (6). This relationship is applicable to sandy and clayey soils as shown by Yasufuku et al. (1999). Equation (2) then is reduced to:

\[ f_s = K \sigma'_o \tan \phi_{sv} \]  

(9)

Figure 1 shows the variation of \( K \) with \( z/L \) for different values of the parameter \( \alpha \), with OCR = 1 and \( \phi_{sv} = 40^\circ \). It can be seen from Eq. (6) that \( K = K_o \) when \( z = 0 \) and \( K = K_o \) when \( z = L \), and also for the special case of \( \alpha = 0 \), \( K = K_o \) irrespective of \( z/L \). The value of \( \alpha \), which may reflect the depositional environment, has a large effect on the variation of \( K \), although in this case a value of 0.2 was initially adopted from in-situ test data.
Averaged Coefficient of Horizontal Effective Normal Stress

When averaging the values of $K$ from Eq. (6) along the pile depth, an averaged coefficient of horizontal effective stress for estimating the total skin friction can be derived as follows:

$$K = \frac{1}{L} \int_0^L Kdz = \frac{1}{\alpha + 1} \left( \alpha K_p + K_0 \right)$$  \hspace{1cm} (10)$$

where, $K_p$ and $K_0$ are defined as the averaged $K_p$ and $K_0$ values in Eqs. (7) and (8). In addition, when introducing $\tilde{\sigma}' = (1/2)\gamma' L$ as the mean effective overburden stress, the total skin friction $F_t$ in a non-displacement pile is then given by:

$$F_t = \frac{1}{2} \gamma' (\pi DL^2) K \tan \tilde{\phi}' = \frac{\gamma' (\pi DL^2)}{2(\alpha + 1)} (\alpha K_p + K_0) \tan \tilde{\phi}'$$  \hspace{1cm} (11)$$

where, $D$ is diameter of pile, $\gamma'$ and $\tilde{\phi}'$ are mean values of $\gamma'$ and $\phi'$, in each layer of soil surrounding the pile. In the case of ground with $n$ layers, $\gamma'$ and $\tilde{\phi}'$ are given by $\gamma' = \sum \gamma_i l_i / L$ and $\tilde{\phi}' = \sum \phi_{ci} l_i / L$, respectively, in which $l_i$ is the thickness of the $i$-th layer. It can be seen that just four parameters, consisting of averaged values of $\gamma'$, $\tilde{\phi}'$, OCR and $\alpha$ are needed in the analysis. The model with $\alpha = 0.2$ gave a good agreement with the experimental results from both the model and full scale pile load tests. In the following, this parameter will be evaluated from a more geotechnical point of view, related to the simple shear mode in soil-pile interaction.

GEOTECHNICAL CONSIDERATION OF THE PARAMETER $\alpha$

Basic Idea

The principal stress axes in the simple shear test rotate during the progressive increase of shear stress on the horizontal plane. Oda and Konishi (1974) proposed the following relationship between the inclination angle $\psi$, an angle between the major principal stress axis and the vertical direction, and the effective stress ratio $\tau/\sigma'$, acting on the horizontal plane (see Fig. 2(a)):

$$\frac{\tau}{\sigma'} = \kappa \tan \psi$$  \hspace{1cm} (12)$$

in which $\sigma'$ is the normal effective stress on the horizontal plane and $\kappa$ is a material constant. Based on theoretical considerations, Ochiai (1975) showed that the constant $\kappa$ can be expressed in terms of the internal friction angle $\phi_c$ at the critical state such that:

$$\kappa = \sin \phi_c$$  \hspace{1cm} (13)$$

In addition, it has been shown that the principal stresses $\sigma_i$ and $\sigma_j$ acting on the specimen in the simple shear apparatus can be formulated from Eq. (12) and considerations of the geometry of Mohr’s circle of stress. The Mohr’s circle of stress showing the relationship between $\sigma_i$, $\sigma_j$, $\sigma'$, $\tau$ and $\psi$ is shown in Fig. 2(b). Consequently, the principal stresses are formulated as a function of $\sigma'$, $\tau$ and $\kappa$ as follows:

$$\sigma_i = (\kappa \sigma'^2 + \tau^2) / \kappa \sigma'$$  \hspace{1cm} (14a)$$

$$\sigma_j = (1 - \kappa) \sigma'$$  \hspace{1cm} (14b)$$

The applicability of Eqs. (12)–(14) has been experimentally verified by many researchers. This idea will be utilized here.

Estimation of Parameter $\alpha$

It is assumed that the simple shear mode is the mechanism under which skin friction of a pile is mobilized. Figure 3, in which $\sigma_i$ and $\sigma_j$ are the vertical and horizontal effective stresses respectively, schematically shows the
assumed simple shear mode when the skin friction is fully mobilized in the ground surrounding the pile. According to the geometry of the Mohr’s circle of stress in Fig. 2(b) and Eq. (13), the ratio of \( \sigma_{ht} \) to \( \sigma_{lt} \) can be expressed as:

\[
\frac{\sigma_{ht}}{\sigma_{lt}} = \frac{1}{1 + \kappa (\tan^2 \psi - 1)} = \frac{1}{1 + \sin \phi_c^* (\tan^2 \psi - 1)}
\]

(15)

It should be noted that the ratio of \( \sigma_{ht} \) to \( \sigma_{lt} \) is simply related to only two parameters. Roscoe et al. (1967) showed that at large shear strains the principal axes of stress and strain increment almost coincide for medium to loose sand, though there is some divergence for dense sand. It may therefore be assumed for practical purposes that at the critical state the principal axes of stress and strain increment coincide. At the critical state in the simple shear test, the strain increment in the vertical direction is zero, so that the inclination angle of the maximum principal strain increment axis to the vertical direction is equal to \( \pi/4 \). Hence, \( \psi \) is also equal to \( \pi/4 \) at the critical state with the friction angle \( \phi_c^* \). Rowe (1969) also indicated that when the sample at simple shear failure satisfies the critical state condition, \( \psi \) always approaches to \( \pi/4 \), irrespective of the magnitude of \( \phi_c^* \). Thus, by substituting \( \psi = \pi/4 \) at the critical state into Eq. (15) \( \tan \psi \) reduces to 1.0, and thus Eq. (15) becomes:

\[
\sigma_{ht} = \sigma_{lt}
\]

(16)

This relationship gives an averaged value over the pile length and its effectiveness can be indirectly verified by comparing the predicted \( \alpha \) values with the empirical ones as discussed below. If Eq. (16) represents the average stress condition of the ground surrounding the pile, then based on the definition of \( K_{dl} = (\sigma_{ht}/\sigma_{lt}) \) and the assumption that \( \sigma_{ht} = \sigma_{lt} \):

\[
K_{dl} = 1.0
\]

(17)

This means that when \( \phi_c^* \) is fully mobilized between soil and pile and the averaged horizontal and vertical normal effective stresses are equal. Further, if we equate Eqs. (17) and (10), parameter \( \alpha \) can be expressed as a function of \( \phi_c^* \) as follows:

\[
\alpha = \frac{1 - \sin \phi_c^*}{2}
\]

(18)

It is important to emphasize that as a logical consequence, parameter \( \alpha \), which was previously empirically determined, is given by a function of \( \phi_c^* \) alone. Figure 4 shows the characteristics of \( \alpha \) against \( \phi_c^* \) from 30 to 42 degrees, in which the experimental values of \( \alpha = 0.2 \) obtained by fitting the results of the full scale pile load tests are also plotted for comparison (Yasufuku et al., 1999). The predicted values of \( \alpha \) roughly change from 0.25 to 0.17 as \( \phi_c^* \) varies from 30 to 42 degrees, and Eq. (18) gives \( \alpha = 0.2 \) when \( \phi_c^* \) is 36.9 degrees. It might be mentioned that although the curve for \( \alpha \) from Eq. (18) tends to decrease with increasing \( \phi_c^* \), the curve roughly fits with the empirical values in the expected range of \( \phi_c^* \). Introducing Eq. (18) into Eq. (11), the corresponding total skin friction of pile \( F_s \) can be formulated as:

\[
F_s = K\sigma' \tan \phi_c^*
\]

(20a)

\[
K = \left\{ 1 - \frac{z}{L} \right\} \frac{(1 - \sin \phi_c^*)^2}{2} K_p + \frac{z}{L} \frac{(1 - \sin \phi_c^*)^2}{2} K_0
\]

(20b)

where \( K_p \) and \( K_0 \) are given by Eqs. (7) and (8) respectively. It can be seen that \( K \) is approximated by a function of the normalized depth \( z/L \) and \( \phi_c^* \). As a practical approach, \( f_c \) is sometimes represented by the following equation:

\[
f_c = \beta \sigma_c^*
\]

(21)

Comparing Eq. (21) with Eq. (20), parameter \( \beta \) can be defined as a function of \( z/L \) and \( \phi_c^* \) such that:

\[
\beta = K \tan \phi_c^*
\]

(22)

Figure 5 shows \( \beta \)-values derived for various normalized depths \( z/L \) over the working range of \( \phi_c^* \), which can be used to graphically find \( \beta \)-values from \( \phi_c^* \) a fundamental geotechnical parameter.

**Comparison with a Full Scale Pile Load Test**

A series of in situ full-scale cast-in-place pile load tests and laboratory soil tests were performed in volcanic Shirasu soil by the Amori river in Kagoshima Prefecture in the southern part of Kyushu island, Japan (Yasufuku et al., 1997). Figure 6 shows the soil profiles and the various soil parameters with depth for the Shirasu sediments. Figures 6(a) and (b) show a summary of the
geological conditions in relation to the test pile. Figures 6(c) and (d) show the variation with depth of the standard penetration test (SPT) N value, the effective overburden pressure $\sigma'_e$ and horizontal stress $\sigma'_h$ determined as the yield stress from pressureremter tests. Finally Figs. 6(e) and 6(f) show the variation of initial void ratio $e_0$ and the proportion of fines $F$ less than 75 $\mu$m. From this data it can be seen that: 1) the Shirasu consists of six layers, which includes silty, sandy and sandy gravel layers 2) the N values lie between 4 and 22, and there is no clear bearing stratum at a depth of 50 m; 3) the measured effective overburden pressure increases almost linearly with depth, however there is no clear relationship between effective horizontal stress $\sigma'_h$ and depth, which may be influenced by the stress history; 4) the initial void ratio $e_0$ lies between 1 and 1.5 and the proportion of fines $F$ lies between 5% and 80%. These values vary considerably for different depths. The fundamental properties related to depth are summarized in Table 1. In order to determine the critical state strength parameters, drained triaxial compression tests were carried out on undisturbed samples taken from locations T1 to T7 (as shown in Fig. 6(a)) using a triple tube sampler. The tests were carried out over a range of confining pressures of 50 kPa to 400 kPa and a strain rate of 0.05% per min. Figure 7 shows the relationship between $\phi'_c$ and the fines ratio $F$, for the samples taken from T1 to T7, in which $\phi'_c$ was determined from the stress-dilatancy relationships for each sample (Yasufuku et al., 1998; Yasufuku et al., 1999). From this data it can be seen that except for T7, which is below the base of the pile, irrespective of fines content $F$, the mean values of $\phi'_c$ lie in the range of 39° to 42° with a variation of only 3°.

The cast-in-place pile with a diameter of 1.2 m and length of 41 m was constructed using the overall casing method and subjected to several cycles of axial load testing. Four strain gauges were located at each of the cross sections shown by the dots in Fig. 6(b). The skin friction was found from the axial stress difference. Figure 8 shows the relationship at different depths between the skin friction and the pile settlement normalized with respect to the diameter D. For each depth considered, the maximum skin friction occurred at a settlement of approximately $S/D = 2\%$. The relationship between this maximum skin friction $f_{s(max)}$ and the depth is shown in Fig. 9, together with the results predicted by Eq. (20). It was not possible to establish a unique relationship between $f_{s(max)}$ and depth although the results from the pile indicated a peak value, with a similar trend also predicted by Eq. (20) using the proposed method. Figure 10 shows typical predicted and measured $\beta$-values against normalized depth, obtained from the results of full scale pile load tests in typically layered Shirasu, (Fig. 6(a)). The predicted $\beta$-values have a similar trend to

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**Fig. 5.** Predicted variation of $\beta$ with $\phi'_c$

**Fig. 6.** Soil profiles and soil characteristics with depth for Shirasu sediments
Table 1. Basic properties of sandy ground with depth

<table>
<thead>
<tr>
<th>Site</th>
<th>Depth (m)</th>
<th>( \rho_s ) (g/cm³)</th>
<th>( \gamma' ) (g/cm³)</th>
<th>( D_{50} ) (mm)</th>
<th>( U_r ) (%)</th>
<th>( F_r ) (%)</th>
<th>( \varepsilon_0 ) (kPa)</th>
<th>( \sigma'_v ) (kPa)</th>
<th>OCR</th>
<th>N-value</th>
<th>( \varphi' ) (°)</th>
<th>( f_{(max)} ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>4</td>
<td>2.484</td>
<td>0.51</td>
<td>0.067</td>
<td>—</td>
<td>52.5</td>
<td>1.892</td>
<td>20.5</td>
<td>36.0</td>
<td>3.4</td>
<td>40.7</td>
<td>37.5</td>
</tr>
<tr>
<td>T2</td>
<td>9</td>
<td>2.436</td>
<td>0.61</td>
<td>0.170</td>
<td>55.6</td>
<td>16.2</td>
<td>1.345</td>
<td>48.6</td>
<td>112.0</td>
<td>5.1</td>
<td>43.0</td>
<td>128.2</td>
</tr>
<tr>
<td>T3</td>
<td>18</td>
<td>2.450</td>
<td>0.61</td>
<td>0.290</td>
<td>3.6</td>
<td>7.5</td>
<td>1.373</td>
<td>103.7</td>
<td>100.0</td>
<td>2.0</td>
<td>41.1</td>
<td>61.1</td>
</tr>
<tr>
<td>T4</td>
<td>26</td>
<td>2.484</td>
<td>0.58</td>
<td>0.290</td>
<td>24.4</td>
<td>10.7</td>
<td>1.582</td>
<td>151.1</td>
<td>130.0</td>
<td>1.2</td>
<td>41.0</td>
<td>98.5</td>
</tr>
<tr>
<td>T5</td>
<td>33</td>
<td>2.418</td>
<td>0.56</td>
<td>0.023</td>
<td>—</td>
<td>76.4</td>
<td>1.520</td>
<td>191.0</td>
<td>71.0</td>
<td>1.0</td>
<td>40.1</td>
<td>72.9</td>
</tr>
<tr>
<td>T6</td>
<td>37</td>
<td>2.436</td>
<td>0.60</td>
<td>0.019</td>
<td>—</td>
<td>81.4</td>
<td>1.393</td>
<td>214.2</td>
<td>81.0</td>
<td>1.2</td>
<td>39.2</td>
<td>91.1</td>
</tr>
<tr>
<td>T7</td>
<td>47</td>
<td>2.660</td>
<td>0.70</td>
<td>0.320</td>
<td>30.8</td>
<td>9.6</td>
<td>1.359</td>
<td>279.4</td>
<td>—</td>
<td>—</td>
<td>38.0</td>
<td>—</td>
</tr>
</tbody>
</table>

Fig. 7. \( \varphi' \) for undisturbed ‘Shirasu’ related to \( F_r \)-values

Fig. 8. Relationship between skin friction and pile normalized settlement obtained from a full scale pile test

the measured values decreasing with the increase in normalized depth.

**SOME APPLICATIONS**

Applicability of the model expressed by Eq. (19) was verified by comparison with the previous load test data from full scale cast-in-place concrete piles. The basic field data are summarized in Table 2, which includes references, the soil description, pile diameter and length, penetration ratio, the measured total skin frictions, the averaged, \( \gamma' \), \( \varphi' \), and \( \sigma'_v \) and the predicted total skin frictions. In addition, averaged \( \beta \) values defined as \( \beta = K \tan \varphi' \) are also given in this table, where suffix ‘(cal)’ and ‘(m)’ mean the calculated and measured values, respectively. The comparison of the predicted and observed total pile skin frictions is shown in Fig. 11. As shown in Table 2, in many cases, when there was insufficient field data, the parameter \( \gamma' \) was empirically determined based on soil classification, and also parameter \( \varphi' \) needed for the predictions was estimated from BS 8002 (1994) for sandy soils and gravel. \( \varphi' \) in degrees is given by:
Table 2. Summary of field and calculated data for cast-in-place concrete piles

<table>
<thead>
<tr>
<th>Pile test</th>
<th>Diameter</th>
<th>Length</th>
<th>L/d</th>
<th>$F_{cm}$ (kN)</th>
<th>$\gamma_{cm}$ (kN/m^3)</th>
<th>$\sigma_0^t$ (kPa)</th>
<th>$\phi_0^t$ (deg.)</th>
<th>$\alpha_c$</th>
<th>$F_{sm}$ (kN)</th>
<th>$\beta_{hub}^s$</th>
<th>$\beta_{hub}^n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mansur and Kaufman (1970)</td>
<td>0.41</td>
<td>5.3</td>
<td>13</td>
<td>290.8</td>
<td>56.8</td>
<td>35</td>
<td>0.21</td>
<td>271.5</td>
<td>0.70</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>Furlow (1968)</td>
<td>0.46</td>
<td>5.1</td>
<td>11</td>
<td>402.4</td>
<td>71.4</td>
<td>38</td>
<td>0.19</td>
<td>411.1</td>
<td>0.78</td>
<td>0.76</td>
<td></td>
</tr>
<tr>
<td>Shii, Someya and Takeuchi (1977)</td>
<td>1.2</td>
<td>11.0</td>
<td>9.2</td>
<td>5910</td>
<td>---</td>
<td>152.0</td>
<td>38</td>
<td>0.19</td>
<td>4928</td>
<td>0.78</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td>1.2</td>
<td>13.0</td>
<td>11</td>
<td>4361</td>
<td>---</td>
<td>99.7</td>
<td>38</td>
<td>0.19</td>
<td>3820</td>
<td>0.78</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td>1.2</td>
<td>13.0</td>
<td>11</td>
<td>6328</td>
<td>---</td>
<td>137.0</td>
<td>38</td>
<td>0.19</td>
<td>5276</td>
<td>0.78</td>
<td>0.94</td>
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*1: data from Coyle et al. (1981), *2: $\phi_0^t$ was determined from BS8002, *3: $\gamma_{cm}$ and $\phi_0^t$ were determined from BS8002, *5: $\beta_{hub}^s$ and $\beta_{hub}^n$ are defined as $K\tan \phi_0^t$.

Fig. 11. Comparison of predicted and measured total skin friction for cast-in-place concrete piles

\[ \phi_0^t = 30 + \Delta \phi_1 + \Delta \phi_2 \]  
where $\Delta \phi_1$ and $\Delta \phi_2$ are defined in the range of 0 to 4 degrees, related to the angularity and the grading of soils, respectively. The applicability of the relationship has been discussed by Yasufuku et al. (2001).

Although only 18 field data are shown here, it can be seen that the predicted and measured results are in good agreement. Further, the measured $\alpha$-values are shown in Fig. 4, together with the predicted $\alpha$-curve. Although the data is scattered it can be seen that the model roughly replicates the trend of the measured $\alpha$-values and also shows good agreement with an empirical value of the parameter $\alpha$ of 0.2.

CONCLUSIONS

A method for predicting the skin friction of non-displacement piles in sandy soils is presented by reconsidering a model, previously proposed by authors. The key idea was to link the model with a simple direct shear soil-pile interaction mechanism related to the critical state friction angle which may be mobilized at large shear strain. The revised model was characterized as a function of the critical state friction angle, soil unit weight and overconsolidation ratio alone. The method of determining skin friction has been shown to be valid from a practical point of view by comparing the calculated results with those from a database of various full scale pile load tests.

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