Geotechnical analysis of skin friction of cast-in-place piles

Frottement superficiel d'un pieu fabriqué sur place

N. Yasufuku & H. Ochiai – *Kyushu University, Hakozaki, Higashi-ku, Fukuoka, Japan*
Y. Maeda – *Kyushu-Kyoritu University, Jiyugaoka, Yahatanishi-ku, Kitakyushu, Japan*

ABSTRACT: A rational method to evaluate the shaft resistance of cast-in-place piles in sandy soils is presented on the basis of geomechanical considerations. The critical state friction angle is shown to be effective as the design parameter for evaluating skin friction. An empirical equation for the coefficient of horizontal stress has also been proposed considering the soil-pile interaction mechanism. The applicability of the model is verified through the use of a newly developed in-situ friction test and full scale pile load tests in volcanic ash sandy 'Shirasu' ground.

RESUME: Une méthode rationale pour évaluer la résistance du fût du pieu de béton fabriqué sur place dans la terre sable est presentée osant une base géomechanique. L'angle de frottement à l'etat des contraintes critique est démontré d'etre effective comme paramètre pour déterminer le frottement lateral. Une équation empirique pour la coefficient d'étrainte latérale a été proposée considerant le méchanisme entre la terre et le pieu. L'applicabilité de la modèle est verifiée osant un nouvaux essai.

1 INTRODUCTION

The total axial bearing capacity of a pile derives from the sum of the end bearing soil resistance and side skin friction between pile and soil. However, because relatively large displacements are required to mobilize the end bearing capacity then in practical designs within the limits of allowable displacement, the main part of the vertical bearing capacity of a pile is from the skin friction. Its precise evaluation therefore is very important. The standard penetration test (SPT) blowcount 'N' value is widely used to semi-empirically evaluate pile skin friction. However, since it relies on using previously collected data, it does not directly reflect the individual ground characteristics and frictional characteristics between pile and soil.

These days cast-in-place piles are frequently used in urban areas because of noise and vibration considerations. In this study, a rational geotechnical method for the evaluation of the skin friction of a pile is proposed relating to the soil characteristics under critical state conditions. Its usefulness is verified through the use of a new in-situ pile skin friction test and a full scale pile loading test in volcanic ash sandy 'Shirasu' ground.

2 GEOTECHNICAL METHOD FOR EVALUATION OF SKIN FRICTION

2.1 *Basic Evaluation Equation*

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:

$$f_\delta = c'_\delta + \sigma'_h \tan\phi'_\delta \quad (1)$$

where f_s is the skin friction at unit length of piles, c'_δ and ϕ'_δ are the adhesion and friction parameters between pile and soil, and σ'_h is the effective lateral 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 c'_δ and ϕ'_δ and that for the in-situ effective stress σ'_h. The following section discusses skin friction characteristics with references to the mobilized mechanism.

2.2 *Design Parameters Concerning Soil Properties*

In cast-in-place piles the maximum skin friction is generally mobilized after a displacement equal to approximately 2% of the pile diameter (see Figure 6). On the other hand in practical design the axial pile capacity is estimated for a settlement of approximately 10% of the pile diameter. Therefore in the case of a practical design the settlement criteria for the mobilization of the maximum skin friction are exceeded. When assuming that the mobilized mechanism of skin friction between pile and soil is essentially due to the shear failure in a thin layer of soils surrounding the pile, it is reasonable and rational to use the strength parameters at the critical state corresponding to the sufficiently large displacements, such that:

$$c'_\delta = 0 \quad (2); \qquad \phi'_\delta = \phi'_{cv} \quad (3)$$

where, ϕ'_{cv} is a friction angle at the critical state. If this friction angle is used for the soil, it is independent of density and is unique. In addition it is important to point out that, if this is used, the soil strengths minimum value is assured which is very useful.

2.3 *Mobilized Lateral Effective Soil Stress*

The lateral effective stress σ'_h in Eq.(1) is important in the evaluation of pile skin friction. This lateral stress is influenced by the contact stress between the pile and soil, the soil deposition environment, stress history and soil properties. The mobilization of the skin friction is dependent on the lateral effective stress σ'_h and this in turn is dependent on the overburden pressure σ'_v, as shown in the following equation:

$$\sigma'_h = K\sigma'_v \quad (4)$$

Therefore, the estimation of the lateral effective stress depends itself on the evaluation of the coefficient of lateral effective stress K. For this it is necessary to consider the soil properties, relative displacement of pile and soil, and the depositional history of the soil.

For cast-in-place piles the range for K is normally $K_0 \leqq K \leqq K_p$, where K_0 is the coefficient of earth pressure at rest and K_p is the passive earth pressure coefficient. Now let us consider the characteristics of K-value:

1) Near the ground surface the effective overburden stress is small and thus the dilation 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-value approaches the passive value.
2) The overburden pressure increases with depth from the ground surface. At the end bearing depth because of the confinement as a result of the dilation due to shear, at rest earth pressure conditions are developed.
3) The decrease of K with depth is affected by the depositional environment. In order to evaluate these properties, the following equation is available,

$$K = \left\{1 - \left(\frac{Z}{L}\right)^{\alpha}\right\} K_p + \left(\frac{Z}{L}\right)^{\alpha} K_0 \tag{5}$$

where, α is the parameter related to the depositional environment of ground including the stress histories and aging effects, and based on the definition, the range of α is $0 \leqq \alpha \leqq 1$. Z is the depth and L is the length of the pile. Here, considering the failure mode assumed, it is natural to evaluate K_p-value as a function of ϕ'_{cv} in the following equation:

$$K_p = \frac{1 + \sin\phi'_{cv}}{1 - \sin\phi'_{cv}} \tag{6}$$

Based on the fact that K_0-value is mobilized under non-plastic deformation condition, Ochiai (1976) derived K_0-value is given by $K_0 = 1 - \sin\phi'_{cv}$. In this study, the following extended equation is presented to evaluate the K-value in Eq.(5):

$$K_0 = \left(1 - \sin\phi'_{cv}\right)\sqrt{OCR} \tag{7}$$

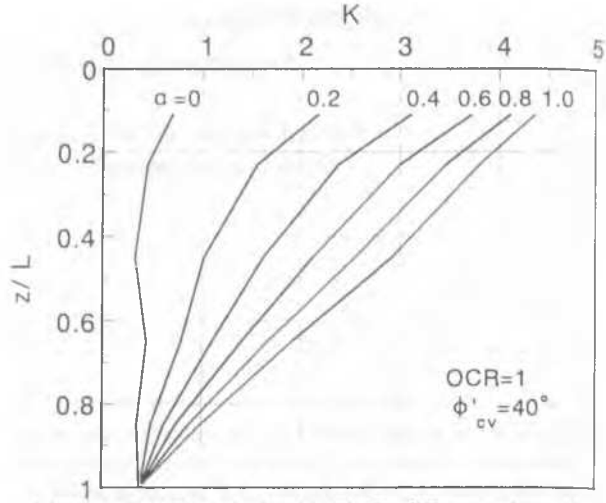

Figure 1. Variation of K with Z/L for different values of the parameter α

where, "OCR" in Eq.(7) means an overconsolidation ratio. It is shown that this type of equation is effective for normally and overconsolidated ground. Based on the above considerations, Eq.(1) is finally reduced as;

$$f_s = K\sigma'_v \tan\phi'_{cv} \tag{8}$$

Figure 1 shows the variation of K with Z/L for different values of the parameter α, using equation (5) with OCR = 1 and ϕ'_{cv} = 40°. It is important to point out that Eq. (5) represent $K = K_p$ when Z=0 and $K=K_0$ when Z=L, and also that for the special case of α =0, $K= K_0$ irrespective of Z/L. It can be seen that the value of α representing the depositional environment has a large effect on the variation of the K value.

3. VERIFICATION FROM IN-SITU TESTS

3.1 *Design Parameters*

3.1.1 *Ground conditions*

Figure 2 shows the soil profiles and the various soil parameters with depth for Shirasu sediments in which the in-situ pile test

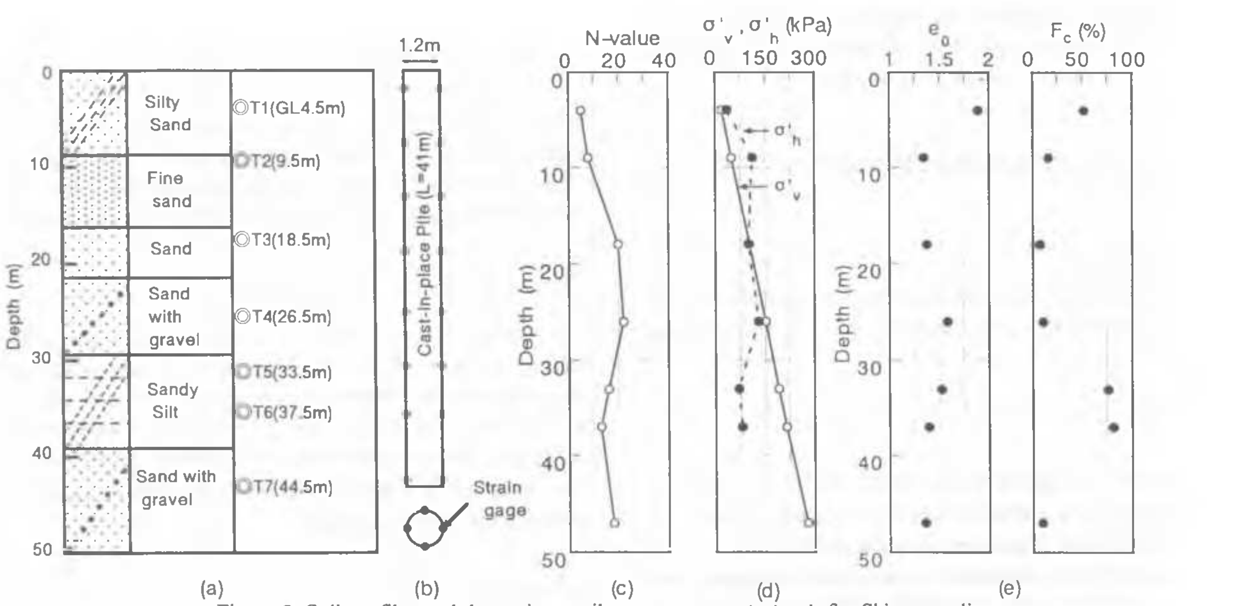

Figure 2 Soil profiles and the various soil parameters with depth for Shirasu sediments

Table1. Fundamental properties of sandy ground with depth

Site	Depth (m)	ρ_s (g/cm³)	γ' (gf/cm³)	D_{50} (mm)	Fc (%)	e_0	σ_v' (kPa)	σ_h' (kPa)	N-value	ϕ_{cv}' (°)
T1	4	2.484	0.51	0.067	52.5	1.892	20.5	36.0	5	40.7
T2	9	2.436	0.61	0.170	16.2	1.345	48.6	112.0	8	43.0
T3	18	2.450	0.61	0.290	7.5	1.373	103.7	100.0	20	41.1
T4	26	2.484	0.58	0.290	10.7	1.582	151.1	130.0	22	41.0
T5	33	2.418	0.56	0.023	76.4	1.520	191.0	71.0	16	40.1
T6	37	2.436	0.60	0.019	81.4	1.393	214.2	81.0	13	39.2
T7	47	2.660	0.70	0.320	9.6	1.359	279.4	—	18	38.0

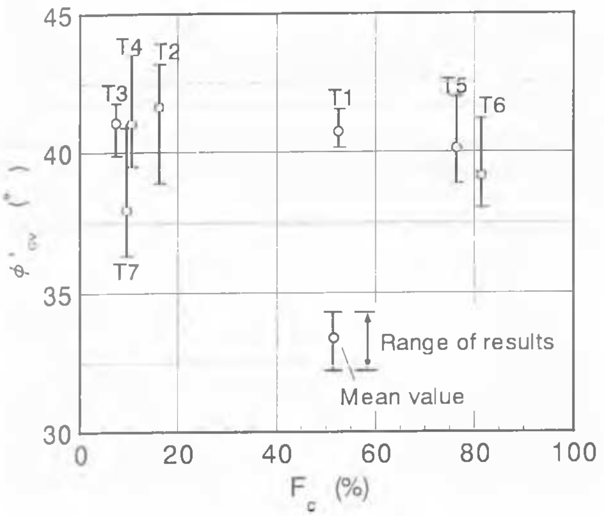

Figure 3. Relationship between ϕ'_{cv} and the fines ratio F_c for the samples taken from each depth

was performed. Figures 2(a) and (b) show a summary of the geological conditions in relation to the length of the load test pile. Figures 2(c) and (d) show the results of the standard penetration together with the effective overburden pressure σ'_v and horizontal stress σ'_h determined as a yield stress from pressuremeter tests. Figure 2(e) shows the variation of initial void ratio e_0 and the proportion of fines F_c less than 75μm.

From this data it can be seen that: 1) the N values lie between 4 and 22, and there is no clear bearing stratum at a depth of 50m, 2) the measured effective overburden pressure increases almost linearly with depth, however there is no clear relationship between effective horizontal stress σ'_h and depth, which seems to be remarkably influenced by the stress history, 3) the initial void ratio e_0 lies between 1 and 1.5 and the proportion of fines F_c lies between 5 and 80%. These values vary considerably for different depths. The fundamental properties of this ground related to the depth are summarized in table 1.

3.1.2 *Strength parameters*

Drained triaxial compression tests were carried out on undisturbed samples taken from locations T_1 to T_7 (as shown in Figure 2(a)) using a triple tube sampler in order to determine the critical state strength parameters. The tests were carried out at a range of confining pressures of 50-400kPa and a strain rate of 0.05% per min. Figure 3 shows the relationship between ϕ'_{cv} and the fines ratio F_c for the samples taken from T_1 to T_7, in which ϕ'_{cv} are determined or extrapolated from the stress-dilatancy relationships of each sample. From this data it can be seen that except for T_7 which is below the base of the pile, the mean values of ϕ'_{cv} lie in the range of 39° to 42° with a variation

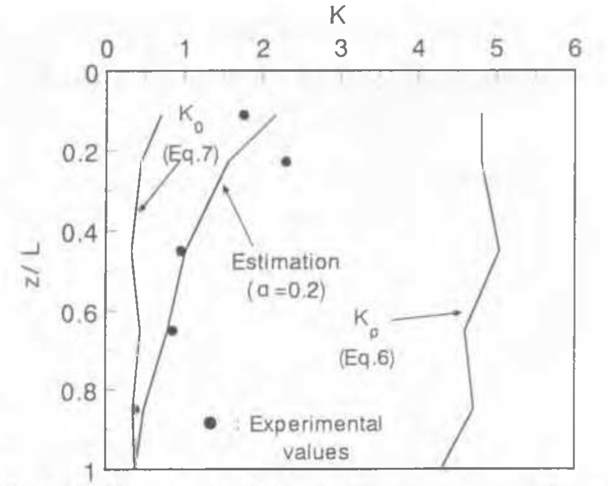

Figure 4. Comparison of lateral effective stress coefficients obtained from pressuremeter test results and computed ones

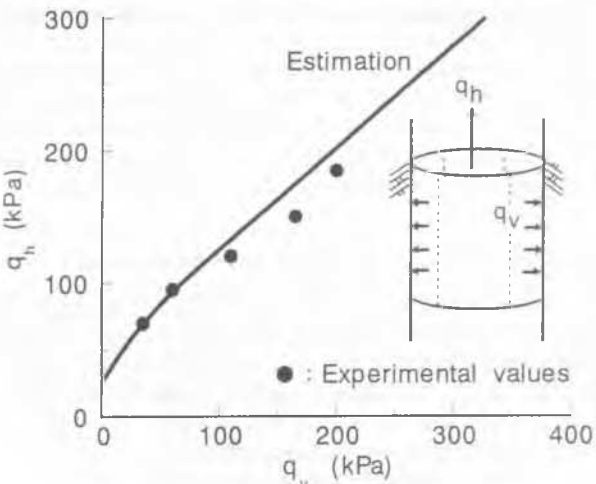

Figure 5. Comparison of bearing stresses values q_h obtained from in-situ friction tests at location T_4 and computed values

of only 3°, irrespective of various fine contents F_c.

3.1.3 *The coefficient of lateral earth pressure and the parameter α*

The variation of the lateral effective stress coefficient $K=\sigma'_h/\sigma'_v$ can be seen in Figure 4, where σ'_v and σ'_h values have been taken from Figure 2(d). Also in this figure the lines are drawn for K, K_p and K_0 using equations (5) to (7) respectively and assuming a value of α=0.2. For these soils the coefficient K lies between the K_0 and K_p values. Using equation (5) the variation of K with depth can be accurately determined. It is also suggested that using a value of α of approximately 0.2 is a valid assumption.

3.2 *A Comparison of In-Situ Friction Test Results*

The newly devised friction testing equipment is a circular borehole device divided into four loading sections (shear plates) as is simply shown in Figure 5. The loading plate was pulled up with a constant vertical stress q_v and the shear stress q_h measured. The details of this equipment have been reported by Maeda et. al (1996). Substituting Eq.(5) into Eq.(8) and considering $q_v=K_o\,\sigma'_v$, the computed values of q_h are given by

$$q_h = \left[\left\{1-\left(\frac{Z}{L}\right)^{\alpha}\right\}K_p\sigma'_v + \left(\frac{Z}{L}\right)^{\alpha} q_v\right]\tan\phi'_{cv} \qquad (9)$$

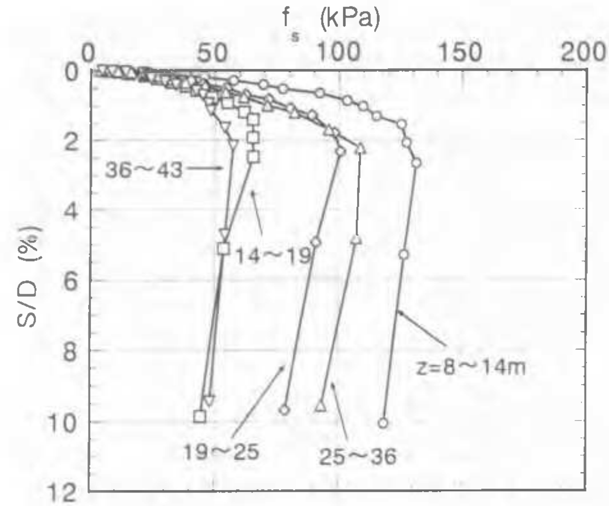

Figure 6. Relationship between the skin friction and the pile normalized settlement obtained from a full scale pile loading test

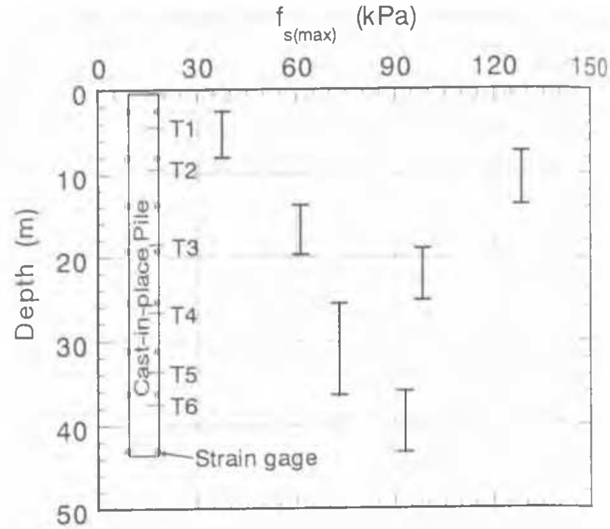

Figure 7. Relationship between maximum skin friction $f_{s(max)}$ and the depth measured from a full scale pile loading test

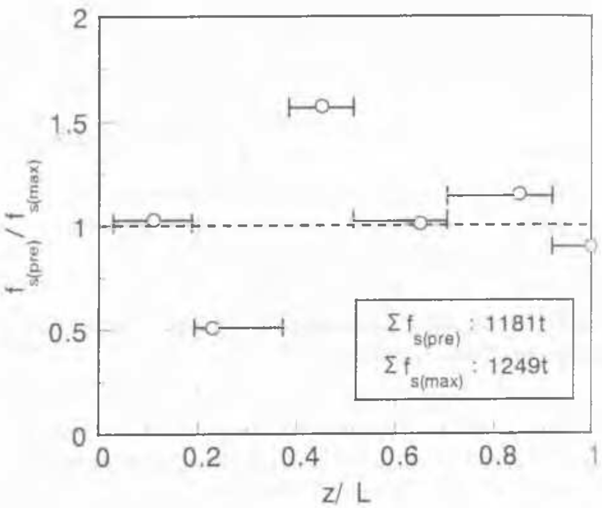

Figure 8. Ratio of the skin friction values estimated using the suggested method and measured skin friction values

Figure 5 shows the data taken from the in-situ friction test at location T_4 (as shown in Figure 2(a)). A comparison can be seen in this plot between the test data and the line estimated using our suggested method. It can be seen from this figure that there seem to be a reasonable fit between the data and the estimated line.

3.3 *Comparison with Full Scale Pile Loading Test Results*

The cast-in-place pile with a diameter of 1.2m and length of 41m was constructed using the overall casing method. It was located in the ground as shown in Figure 2 and subjected to several cycles of axial load testing. Four strain gauges were located at each of the cross sections as shown by the dots in Figure 2(b). The skin friction is found from the axial stress difference.

Figure 6 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 approximately S/D=2%. The relationship between this maximum skin friction $f_{s(max)}$ and the depth is shown in Figure 7. In this case it was not possible to establish a unique relationship between $f_{s(max)}$ and depth. In Figure 8, the ratio of the skin friction values estimated using the suggested method and measured skin friction values $f_{s(pre)}/f_{s(max)}$ are shown plotted against the normalized pile depth Z/L for each of the locations T_1 to T_6. The ratio between estimated and measured values along the length of the pile is around 1.0 suggesting that the estimation method is a valid one. The ratio of estimated and measured skin frictions summed over the length of the pile:

$$\frac{\Sigma f_{s(pre)}}{\Sigma f_{s(max)}} = \frac{1181(ton)}{1249(ton)} = 0.94 \tag{10}$$

gives an even greater precision of estimation.

5 CONCLUSIONS

A new geotechnical method of estimating the skin friction for cast-in-place piles has been presented. Its usefulness has been verified using laboratory tests, in-situ friction tests and full scale pile loading tests. The main conclusions are as follows:

1. It is essential and reasonable to use the critical state values for soil strength when evaluating the pile skin friction mobilized in practical design.
2. Based on the geomechanical consideration, an rational equation has been proposed for the lateral effective stress coefficient used in the determination of skin friction. This equation reasonably represents the changes of lateral effective stress with depth.
3. A method of determining skin friction has been presented and been proved as valid, comparing the calculated results with those of two types of field tests.

REFERENCES

Maeda, Y., H. Ochiai, N. Yasufuku & Y. Morita 1996. Considerations on bearing capacity based on the data from S.B.I.F.T, Faculty of Engineering, Kyushu Kyoritsu University, Research Reports Vol.21 (In Japanese).

Ochiai, H. 1976. The Coefficient of Earth Pressure at Rest of Sands, Domestic Edition of Soils and Foundations, Vol.16, No.2, pp.105-111.