Estimation of Time Dependent Consolidation Settlement on Floating Type Improved Ground

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1. Introduction

In the case of constructing superstructures such as embankment on deep soft clay layer, a combined technique is being applied widely for reducing the settlement as well as construction cost. This technology of combining the float-type cement-treated columns and surface stabilization is perceived as one of the effective and acceptable methods for improving the soft clay ground.
Figure 1 shows the concept of floating type ground improvement. As shown in Fig. 1, this type of technique can be useful for deep soft soil layer considering that it can reduce the settlement. In practical design for this structural form, it is necessary to predict the total settlement of the ground in relation to the degree of improvement. In fact, the saturated soft ground usually displays strong nonlinear, low strength and high compressibility. Especially, the deformation of soft soil may get considerable with time elapsed. Under such circumstance, it is of practical significance to properly consider the time dependent behavior of soil deformations in the interaction analysis of columns and surrounding soil. In this paper, for investigating the time dependent consolidation settlement, a hypothesis of skin friction that around the columns' surface is adopted (R. Ishikura, 2009), and based on the hypothesis, a rational model for evaluating time dependent skin friction is proposed. Meanwhile, a series of ring shear tests and laboratory model tests using kaolin clay were conducted. Based on the comparison of calculated and experiment results, the validity and rationality of this time dependent skin friction model is verified.

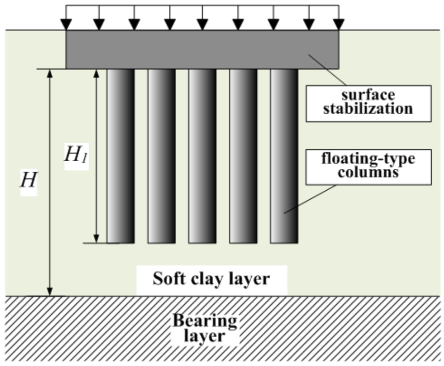

Fig. 1 Concept of floating type ground improvement

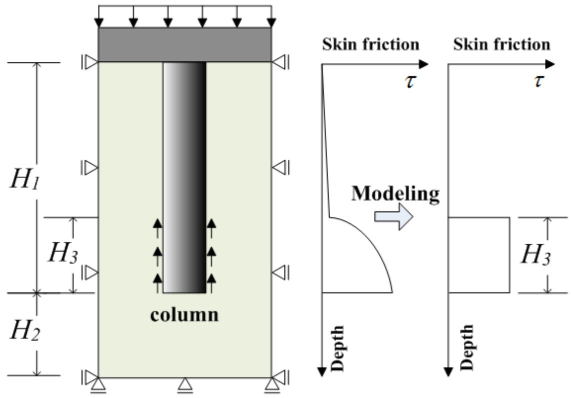

Fig. 2 Hypothesis of skin friction around the surface of columns

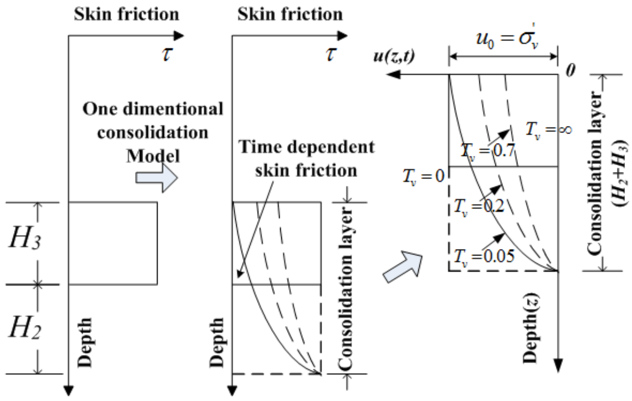

Fig. 3 Illustration of time dependent skin friction model

2. Skin friction during consolidation

2.1 Full mobilized skin friction length

Loading on ground surface would cause compression and consolidation settlement. Pile-soil relative displacement increases subsequently, action exerting on interface skin friction is reinforced, resulting in augment onto the interface skin friction. The combined foundation transfers the load to the pile group via the surface stabilization, hence soft clay between the friction piles in the upper part of the combined foundation is enclosed.
Figure 2 shows the hypothesis of skin friction around the surface of columns. Based on field measurement, it shows that the excess pore water pressure increased gradually from the ground surface, and increased significantly at a depth until pile end. The pile soil interface resistance can be expressed as Equation (1).

$$\tau = K\sigma_v^{'} \tan\delta^{'} \tag{1}$$

where, τ is the interface skin friction, K is the coefficient of horizontal effective stress, σ_v' is the effective overburden pressure, and δ' is the effective friction angle of pile soil interface.

2.2 Time dependent skin friction model

For normal consolidated saturated soft clay, after applying overburden load, the effective stress in soft clay increased with time elapsed due to the excess pore water pressure dissipation. The skin friction at random time (t) and depth (z), can be calculated by Equation (2).

$$\tau(z,t) = K\sigma_{z,t}^{'} \tan\delta^{'} \tag{2}$$

where, $\sigma_{z,t}' = u_0 - u_{z,t}$, u_0 is the initial excess pore water pressure, $u_{z,t}$ is the excess pore water pressure at random time (t) and depth (z). As shows in Figure 3, the excess pore water pressure equal to effective overburden pressure initially, and then decreased with time until reached to zero.

The total skin friction caused by soft soil consolidation can be computed as below:

$$f(t) = K\tan\delta^{'}(u_0 H_3 - \int_0^{H_3} u_0 \Omega(z,t)) \tag{3}$$

$$\Omega(z,t) = \sum_{n=0}^{\infty} \frac{2}{M} \sin\frac{Mz}{H} \exp(-M^2 T_v) \tag{4}$$

where, M=(2n+1)π/2, n=0,1,2···, T_v=$C_v t/H_2$, H=H_2+H_3, t is consolidating time and H is the length of the drainage path.

For this floating-type pile improved technique, there will be penetration of the piles into the underlying soft clay layer, a part of pile improved portion (H_1) can be treated as unimproved portion, as shown in Figure 2, full mobilized skin friction length H_3, a phenomenon similar to that encountered in the problem of calculating the settlement of floating group pile foundation (Terzaghi and Peck, 1967). The unimproved portion length (H_3) is the key point for estimating the consolidation settlement. Based on a homogenization method (Omine and Ochiai, 1992; R. Ishikura, et.al., 2009) in which the stress concentration ratio is considered, the length of H_3 can be calculated.

2.3 Influence of roughness coefficient

In general, the skin friction resistance is mainly determined by the slip surface, which is influenced by pile shaft and ground soil relative roughness. Equation (2) can be changed considering roughness coefficient as following,

$$\tau(z,t) = K\sigma'_{z,t}(R\tan\delta') \quad (5)$$

where, R is the interface roughness coefficient, φ' is the effective friction angle of soft soil. When R=1, it means the slip surface is in the soil mass, $\varphi' = \delta'$; and when R<1, $\varphi' > \delta'$, it means soil roughness is large than pile shaft roughness, the slip surface will generate on the pile soil interface. The relationship between shin friction and roughness coefficient can be investigated detailedly using ring shear tests.

3. Comparison of calculated and model test results

3.1 Laboratory model tests

The model tests were conducted under the plane strain condition, which is comprised of two parts. Firstly, the specimen was prepared under a pre-consolidation pressure of 40kPa using a bellofram cylinder until the end of primary consolidation. During the test, the settlement at the top of the model ground and the vertical pressure were monitored. Secondly, after pre-consolidation finished, a model pile composed of aluminum and have 30 mm in wide D, 100mm in depth and 200mm in length H_1, was inserted in the model ground for the consolidation settlement test. The vertical pressure was increased stepwise from 20kPa to 40kPa, 40kPa to 80kPa and 80kPa to 160kPa by using a bellofram cylinder.

3.2 Results comparison

The ground soil and model foundation size parameters are listed in Table 1and 2. According to Equation (6), the consolidation settlement can be computed:

$$S = m_{vs} H \overline{\sigma'_t} \quad (6)$$

where m_{vs} is the coefficient of volume compressibility of soil, H is the consolidating layer length, and $\overline{\sigma'_t}$ is the average effective vertical pressure at random time (t) . H can be obtained using homogenization method. And the comparison results are shown in Fig. 4 and Fig. 5. $\bar{\tau}$ is the average skin friction, p_0 and Δp are the initial and incremental vertical pressure, and L is the distance between two columns.

4. Conclusions

The experimental results are used to verify the skin friction model and calculation method. The conclusion of this study can be summarized as follows：

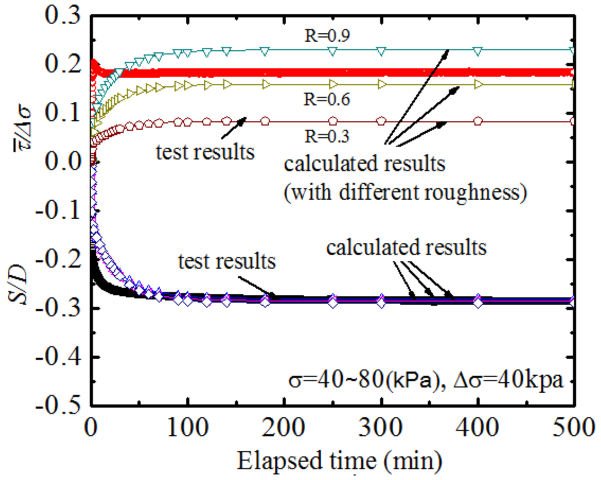

Fig. 4 Comparison between calculated and model test results under 80kPa

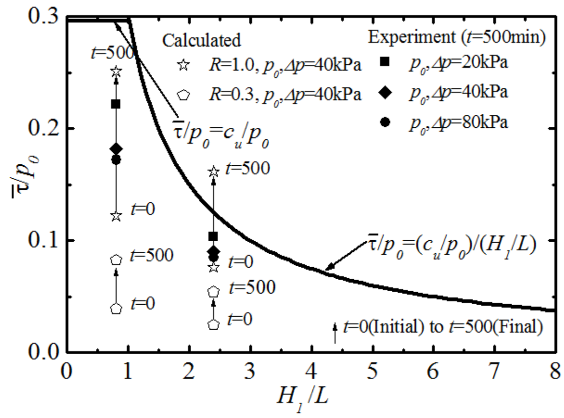

Fig. 5 Comparison between formulation of upward skin friction and experiment results

Table 1. Soil parameters

φ'(°)	C_c	e_0	K_0	γ_{sat} (kN/m^3)	OCR
31	0.294	1.79	0.48	18	1

Table 2. Laboratory model size parameters

H_1(m)	H_2(m)	L	R
0.2	0.07	0.25	0.9,0.6,0.3

1) For this technique, a part of pile improved portion (H_1) will be treated as unimproved portion (H_3), and can be equivalent as full mobilized skin friction length.
2） The skin friction is influenced greatly by roughness coefficient. From the comparison results, when R≈0.65, the calculated results fit well with experimental results.

References

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