Consolidation settlement of floating-column-improved soft clayey deposit

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The interaction behaviour between a floating column and the surrounding soil, with or without a surface cement stabilised slab, has been investigated by laboratory model tests as well as finite-element analysis (FEA) using an axisymmetric unit cell model. Based on the test and the FEA results, a method for calculating the consolidation settlement–time curve of floating-column-improved soft clayey subsoil has been developed. This method has been modified from the earlier method of Chai and Pongsivasathit by treating a part of the column improved layer with a thickness of \( H_c \) as an unimproved layer in settlement calculation to consider the effect of possible penetration of the column into the soft soil layer below the column. An explicit equation for calculating the value of \( H_c \) has been proposed as a function of area improvement ratio \((\alpha)\), depth improvement ratio \((\beta)\), load intensity \((p)\) and the undrained shear strength \((s_u)\) of the soft clayey soil. The validity of the method has been checked using the laboratory model test results as well as four field case histories in Japan.

**Notation**

- \( A_c \): area of the column
- \( A_e \): area of the unit cell which represents a column and its improvement area
- \( c' \): effective cohesion of soil
- \( c_v \): coefficient of consolidation
- \( c_{v1} \): coefficient of consolidation for layer \( H_{1c} \)
- \( c_{v2} \): coefficient of consolidation for layer \( H_{2c} \)
- \( D_{c} \): constrained moduli of the column
- \( D_{c1} \): constrained moduli of the column of the sub-layer \( H_{1i} \)
- \( D_{s} \): constrained moduli of the surrounding soil
- \( D_{s1} \): constrained moduli of the surrounding soil of the sub-layer \( H_{1i} \)
- \( d_c \): diameter of column
- \( d_e \): diameter of unit cell which represents a column and its improvement area
- \( d_s \): diameter of smear zone
- \( E \): Young’s modulus
- \( E_i \): Young’s modulus in the corresponding subsoil layer
- \( e_0 \): initial void ratio
- \( e_{0i} \): initial void ratio in the corresponding subsoil layer
- \( H \): thickness of soft clayey layer
- \( H_{k} \): thickness of the upper layer for the \( s(t) \) calculation
- \( H_{1} \): thickness of the upper layer for the \( U(t) \) calculation
- \( H_{1c} \): thickness of subsoil layers in \( H_1 \)
- \( H_{2} \): thickness of the lower layer for the \( s(t) \) calculation
- \( H_{2c} \): thickness of subsoil layers in \( H_2 \)
- \( H_{20} \): initial thickness of lower layer
- \( k_c \): hydraulic conductivity of the column
- \( k_h \): hydraulic conductivity in the horizontal direction
- \( k_{v} \): hydraulic conductivity of the smear zone
- \( k_v \): hydraulic conductivity in the vertical direction
- \( k_{v1} \): coefficient of permeability for layer \( H_{1c} \)
- \( k_{v2} \): coefficient of permeability for layer \( H_{2c} \)
- \( L_{s} \): length from the end of the column to a point at which the settlement ratio \((SR)\) satisfies a pre-specified criterion
- \( LR \): length ratio
- \( LR' \): length ratio including the thickness of a slab
- \( M \): slope of the critical state line in \((q, p')\) plot

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1. Introduction

Deep cement mixing, normally forming stronger and stiffer columns in the ground, is a widely used soft clayey ground improvement method (e.g. Broms and Boman, 1979; Bergado et al., 1994). Recently, to reduce the construction cost and minimise the impact on the ground environment, a method of improving soft clayey deposits by floating soil-cement columns, with or without a cement stabilised slab on the ground surface, is increasingly being used (Chai et al., 2009; Shen et al., 2001). One of the important aspects in the design of such a system is the calculation of the consolidation settlement of the improved soft deposit.

Chai et al. (2009) studied the interaction behaviour between the floating column–slab system and the surrounding soft soil by finite-element analysis (FEA) using an axisymmetric unit cell model. The results of FEA revealed that the relative penetration of the column into the surrounding soft clayey soil is influenced by (a) area improvement ratio ($\alpha = A_c/A_s$; $A_c$ being the cross-sectional area of the column and $A_s$ the cross-sectional area of the unit cell which represents a column and its improvement area); (b) depth improvement ratio ($\beta = H_c/H_s$ being the length of the column and $H$ the thickness of soft clayey layer); (c) load intensity ($p$), and (d) the strength and stiffness of the soft deposit. A method was then proposed to calculate the final consolidation settlement of the floating column–slab system considering the effect of $\alpha$ and $\beta$ only. Based on the the Chai et al. (2009) method, Chai and Pongsivasathit (2010) developed a method for calculating the consolidation settlement–time curves of the floating column–slab improved deposit.

There are some shortcomings in the method proposed by Chai et al. (2009). First, it does not include the effect of the load intensity and the undrained shear strength ($s_u$) of the soft soil into the equations for calculating $H_s$ which is the thickness of the part of the column improved layer to be treated as an unimproved layer in settlement calculations. As a result, it will overpredict the settlement for a stiffer deposit under a lower surcharge load (for example 50 kPa), and underpredict the settlement for a very soft deposit under a higher surcharge load ($p > 150$ kPa) (Chai et al., 2009). Second, in engineering practice there are cases where the soft deposit is only improved by column inclusions and without a cement-stabilised slab on the ground surface, is non-linear. As a result, the method under-predicts the settlement for $\alpha < 15\%$.

In this paper, laboratory model tests as well as further FEA were conducted to investigate the interaction behaviour between the floating column and the surrounding soft clayey soil. Based on the results, a modified method for predicting the settlement–time curve of floating-column-improved soft clayey soil with or with-
out a surface cement-stabilised slab has been proposed. The proposed method has been applied to four field cases in Japan, and its usefulness is demonstrated.

2. **Laboratory model tests**

2.1 Set-up and test procedure

Laboratory model tests were conducted to investigate the behaviour of a model floating soil-cement column improved ground. The cylinder mould is 0·45 m in diameter and 0·9 m in height and made of PVC. The test set-up is shown in Figure 1. The two types of soil sample used were reconstituted Ariake clay, and Ariake clay–sand mixture (mixed soil). The properties of the soils are listed in Table 1. The undrained shear strength of the soils was measured by both laboratory vane shear test (vane: 20 mm in diameter and 40 mm in height) and unconfined compression test using reconstituted soil samples under a consolidation pressure of about 38 kPa (a pressure used to make model ground for the model test). The \( s_u \) values from both tests are very close and the values shown in Table 1 are average ones.

The cement used for making the model column was US10, a typical cement used for ground improvement in Japan. US10 is a type of cement produced by Ube Material Industries, Japan, using waste concrete as part of the raw material. Its compression strength is almost the same as commonly used Portland cement.

As for the properties of the model column, based on the results of unconfined compression tests, the Young’s modulus of the model column was estimated as \( 6 \times 10^4 \) kPa in the case of the Ariake clay soil and \( 1 \times 10^5 \) kPa for the mixed soil, and the Poisson’s ratio \((\nu)\) was assumed as 0·2. Non-woven geotextile with a thickness of about 3 mm (under zero confining pressure) was used as drainage material at the bottom and the top of the model ground. The geotextile was made of polypropylene and weighed about 130 g/m\(^2\).

To accelerate the process of consolidation during the preloading (making model ground) process, a mini-prefabricated vertical drain (mini-PVD) was installed in the middle of the model ground. The mini-PVD was made by folding the geotextile in three layers with a cross-section of 30 mm by 9 mm.

The detailed procedures of testing can be found in Chai and Pongsivasathit (2010) and the main steps are briefly explained as follows.

2.1.1 Test set-up and pre-consolidation

First, three layers of the geotextile were put at the bottom of the model as drainage layers. A thin layer of silicon grease was painted on the inside wall of the mould to reduce friction. Then, the remoulded soil sample with a water content higher than its liquid limit (about 120\% and 60\% for the Ariake clay and the mixed soil respectively) was put in the mould layer by layer. Piezometers were installed at locations of 0·2 m, 0·4 m and 0·55 m height from the bottom. When the thickness of the soil sample reached 0·78 m, the mini-PVD was installed (pushed in by a stainless steel rod) in the centre of the model, and three layers of the geotextile were placed on the top of the model. Then the loading system as shown in Figure 1 was set up, and air pressure of 40 kPa was applied. Considering the effect of the

The properties of the soils used in laboratory model tests are as follows:

<table>
<thead>
<tr>
<th>Soil type</th>
<th>( w_L: ) %</th>
<th>( w_P: ) %</th>
<th>( \gamma_t: ) kN/m(^3)</th>
<th>( \nu )</th>
<th>( \kappa )</th>
<th>( \lambda )</th>
<th>( \epsilon_0 )</th>
<th>( s_u: ) kPa</th>
<th>( k_h: \times 10^{-4} ) m/day</th>
<th>( k_v: \times 10^{-4} ) m/day</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ariake clay</td>
<td>110</td>
<td>60</td>
<td>15·5</td>
<td>0·3</td>
<td>0·033</td>
<td>0·331</td>
<td>3·20</td>
<td>10·0</td>
<td>3·651</td>
<td>3·651</td>
</tr>
<tr>
<td>Mixed soil</td>
<td>58</td>
<td>36</td>
<td>15·9</td>
<td>0·3</td>
<td>0·015</td>
<td>0·149</td>
<td>1·52</td>
<td>13·1</td>
<td>2·740</td>
<td>2·740</td>
</tr>
</tbody>
</table>

Note: \( w_L \) is liquid limit; \( w_P \) is plastic limit; \( \gamma_t \) is total unit weight; \( \lambda \) is the slope of virgin compression line in \( e-\ln p' \) plot (\( p' \) being the mean effective stress); \( \kappa \) is the slope of unloading-reloading line in \( e-\ln p' \) plot; \( \epsilon_0 \) is initial void ratio; \( k_h \) and \( k_v \) are hydraulic conductivity in the horizontal and the vertical directions respectively.

Table 1. Properties of soils used in laboratory model tests

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Figure 1. Set-up of the laboratory model test
shaft fixed on the piston, the pre-consolidation pressure \((p_0)\) was about 38 kPa. The consolidation was under a two-way drainage condition. During the test, the settlement at the top of the model ground and the excess pore pressure at three different locations were monitored.

2.1.2 Soil-cement column installation

After the degree of pre-consolidation reached about 90%, judged on the measured shape of the settlement–time curves and excess pore pressures, the test was stopped, the loading system was dismounted and the mini-PVD was withdrawn. Then at the centre of the model ground, a hole with a diameter of 150–246 mm was made by an augur to a pre-designed depth. A frame was used to fix the position of the augur to make sure the hole could be made in the correct location. At the bottom of the hole a small hole was left after withdrawal of the mini-PVD, which was carefully filled with the clay. The soil removed by the augur from the model ground was mixed with the cement (16-6% by dry weight) and put back into the hole to form the model soil-cement column. For ease of mixing with the cement, the water content of the excavated soil was adjusted to about 120% and 60% for the Ariake clay and the mixed soil respectively. Then the model was left for 2 weeks to cure the column before the further consolidation test. According to laboratory unconfined compressive test results of cement-stabilised Ariake clay samples at different curing times, the strength with 2 weeks curing time is more than 90% of the strength with 28 days curing time (Taguchi et al., 2007). To shorten the time period for one test, 2 weeks curing time was adopted before the further consolidation test.

2.1.3 Consolidation test

After 2 weeks curing, three layers of geotextile were installed on the top of the model ground, and the loading system was set up again. Then a pressure of 38 kPa was applied first for about 1 day to ensure firm contact between the piston and the top of the model ground and to bring the model ground to a normally consolidate state, and then the pressure was increased to 95 or 190 kPa and the consolidation test was started. During the test, settlement at the top of the model ground and excess pore water pressures were monitored.

In total five cases were tested with conditions as listed in Table 2.

2.2 Comparison of measured and calculated values

Chai et al. (2009) proposed a method to calculate the final consolidation settlement of floating column–slab system improved clayey ground, which treats a part of the soil-cement column improved layer, with thickness \(H_c\) (see Figure 2), as an unimproved layer and to calculate its compression using the properties of the soft soil only. The core of the method is the proposed equation for calculating the value of \(H_c\), which is a function of \(\alpha\) and \(\beta\).

![Figure 2. Floating-column-improved soft clayey subsoil](image)

<table>
<thead>
<tr>
<th>Case</th>
<th>Soil type</th>
<th>(\alpha): %</th>
<th>(\beta): %</th>
<th>(p_0): kPa</th>
<th>(P): kPa</th>
<th>Settlement: mm</th>
<th>RE: %</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1</td>
<td>Ariake clay</td>
<td>11</td>
<td>70</td>
<td>38</td>
<td>95</td>
<td>35.26</td>
<td>31.21</td>
</tr>
<tr>
<td>L2</td>
<td>Ariake clay</td>
<td>30</td>
<td>50</td>
<td>38</td>
<td>95</td>
<td>37.86</td>
<td>37.16</td>
</tr>
<tr>
<td>L3</td>
<td>Ariake clay</td>
<td>11</td>
<td>70</td>
<td>38</td>
<td>190</td>
<td>68.38</td>
<td>59.95</td>
</tr>
<tr>
<td>L4</td>
<td>Mixed soil</td>
<td>30</td>
<td>50</td>
<td>38</td>
<td>95</td>
<td>27.32</td>
<td>27.93</td>
</tr>
<tr>
<td>L5</td>
<td>Mixed soil</td>
<td>30</td>
<td>70</td>
<td>38</td>
<td>95</td>
<td>18.98</td>
<td>19.22</td>
</tr>
</tbody>
</table>

Note: \(p\) is consolidation pressure; \((s)_{\text{mea}}\) and \((s)_{\text{cal}}\) are the calculated and measured final settlement respectively.

Table 2. Cases studied and comparison of measurements with calculated values from Chai et al. (2009)
The measured final settlements are compared with the calculated values by the Chai et al. (2009) method. Although the applied surcharge pressure was 95 kPa, it was increased to that level from a pressure of about 38 kPa. Therefore, the increment was only 57 kPa. Then considering the degree of consolidation (of about 90%) under 38 kPa, an incremental consolidation pressure of about 61 kPa was evaluated for calculating the final settlement. In the case of a 190 kPa surcharge load, the incremental consolidation pressure was about 156 kPa. The results are listed in Table 2. In the table, the relative error (RE) is defined as the percentage of the difference between the calculated and the measured settlement divided by the measured settlement (Chai et al., 2009). Based on the results in Table 2, the following comments can be made.

(a) For the model tests using Ariake clay (L1, L2 and L3), the Chai et al. (2009) method resulted in a good prediction of the final settlement for L2 (α = 30%) but under-estimated the settlement for L1 and L3, which had a small α value of 11%. Further, the under-estimation increased with the increase in load intensity (comparing L1 and L3).

(b) Although relatively small, the calculation method over-estimated the final settlement for L4 and L5 that used the mixed soil, which is stronger and stiffer than the reconstituted Ariake clay.

All these results confirmed the tendency reported by Chai et al. (2009). Further, the Chai et al. (2009) method is only applicable for a column slab system, but in the field there are cases where the soft deposit was only improved by column inclusions (e.g. Igaya et al., 2010). Therefore, the interaction behaviour between the soil-cement column and the slab was assumed as a linear elastic material. The assumed thickness of the soft layer is 1 m. The diameter of the column is 1 m. The other parameters were assumed by referring to the site investigation results of soft Ariake clay in the Saga Plain, Japan (Hino et al., 2008) and the parameter values adopted by Chai et al. (2009); the values are given in Table 3. The unconfined compression strength (qc) of the soil-cement column and the slab was assumed as 550 kPa, the Young’s modulus (E) was estimated as 100qc and the Poisson’s ratio at 0.2. It was assumed that the groundwater level was 1·0 m below the ground surface. The program used for the FEA is Plaxis 2D V8.

The purpose of the analysis was to identify the interaction mechanism between the column and the surrounding soft soil as well as to quantify it. Thus, the results focus on the relative settlement between the column and the surrounding soil. For ease of presentation, a parameter called length ratio (LR) (Chai et al., 2009) is used with definition as follows

\[ LR = \frac{L}{H} \]

where \( L \) is the length from the foundation column to a point on the column at that elevation. The settlement of the soft soil \( \delta_s \) at the middle between two adjacent columns equals a pre-defined portion (for example 0·95) of the settlement of the column \( \delta_c \). In other words, LR is a measure of the percentage length of the column which has considerable relative movement (pre-defined criterion) with the surrounding soft soil. A larger value of LR means a longer portion of the column has considerable relative movement with the surrounding soil. \( \delta_s/\delta_c \) is designated as the settlement ratio (SR) (Chai et al., 2009).

3.1.1 Numerical results

3.1.1.1 EFFECT OF THE CEMENT-STABILISED SURFACE SLAB

With the same improvement depth, the length of the column for the with-slab case will be smaller than for the without-slab case. Then for a given improvement depth, for both cases, the LR value will be different. To make a direct comparison of with- and without-slab cases under the same improvement depth, the following parameters are introduced for the with-slab case

\[ \beta' = \frac{H_s + H_f}{H + H_s} \]

\[ LR' = \frac{L_s}{H_s + H_f} \]

where \( H_s \) is the thickness of the slab and \( H_f \) is the thickness of the soft layer below the slab.

The relationships between \( \beta \) (or \( \beta' \)) and LR (or LR') are compared in Figure 5. It can be seen that the without-slab case has a smaller LR value, which means a smaller relative penetration of the column into the surrounding soil. The difference is reduced with increase of \( \beta \) (or \( \beta' \)). The reasons for less relative penetration of the column are: (a) less stress concentra-
The effect of $\alpha$ is shown in Figure 7. When $\alpha$ is larger than 20%, the LR of the without-slab system differs little from LR of the with-slab system. However, when $\alpha$ is lower than 20%, the difference between the two systems increases with reducing $\alpha$ value.

### 3.1.1.2 EFFECT OF $p$ AND $s_u$ VALUES OF THE SURROUNDING SOIL

For a given soil condition, the relative penetration of the column into the underneath soft soil will increase with the increase of the magnitude of the load at the end of the column, while for a given load, the weaker the surrounding soil the larger the relative penetration. Therefore, it is considered that the ratio of $p$ and $s_u$ ($p/s_u$) can be a parameter to represent the influence of $p$ and $s_u$ on the relative penetration of the column. Since the $s_u$ value of a deposit varies with depth, we propose to use the value at the elevation of the end of the column, which will have the most significant influence on the penetration of a column into the surrounding soft soil. For the Soft-Soil model (Brinkgreve, 2002) used, the $s_u$ value can be calculated by the following equations

4. $s_u = \frac{1}{2^\Lambda+1} Mp\delta(OCR)^\Lambda$

5. $p\delta = \left(\frac{M^2 + \eta^2}{M^2}\right)^\Lambda (p' + c' \cot \phi')$

6. $\eta = \frac{q}{p' + c' \cot \phi'}$
where $p_{90}$ is the effective mean stress on isotropic consolidation line corresponding to the current yielding stress state, $p'$ is mean effective stress, $q$ is deviatoric stress, $M$ is the slope of the critical state line in $(q, p')$ plot, $\eta$ is stress ratio, and $\Lambda$ equals $(1 - \kappa \lambda)$.

For the case of $\alpha = 20\%$ with slab: Without slab:

- For $SR = 1.00$: $LR = 1.00$
- For $SR = 0.95$: $LR = 0.95$
- For $SR = 0.90$: $LR = 0.90$

Figure 5. Comparison between with and without-slab cases on the effect of $\beta$

Figure 6. Deformation modes of column–slab and floating-column systems

Figure 8 shows the effect of $p/s_u$ on $LR$, an index for the relative penetration of the column. In the case of varying $p$, the $s_u$ at the end of the column was fixed. In the case of varying $s_u$, the constant $p$ value of 100 kPa was applied. The change of $s_u$ was achieved by varying cohesion, $c'$, in Equation (5). For $SR = 1.00$, $LR$ increases with increase of $p/s_u$. However, when $p/s_u > 8$, the...

Table 3. Assumed model parameters

<table>
<thead>
<tr>
<th>Depth: m</th>
<th>Soil layer</th>
<th>$v$</th>
<th>$E$: kPa</th>
<th>$\kappa$</th>
<th>$\lambda$</th>
<th>$c'$: kPa</th>
<th>$\phi'$: deg</th>
<th>OCR</th>
<th>$\gamma_i$: kN/m$^3$</th>
<th>$e_0$</th>
<th>$k_h$: $\times 10^{-8}$ m/s</th>
<th>$k_v$: $\times 10^{-8}$ m/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0–4.0</td>
<td>Clay-1</td>
<td>0.3</td>
<td>0.065</td>
<td>0.65</td>
<td>5–20</td>
<td>30</td>
<td>4–1.1</td>
<td>13.6</td>
<td>3.3</td>
<td>3.0</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>4.0–16.0</td>
<td>Clay-2</td>
<td>0.3</td>
<td>0.065</td>
<td>0.65</td>
<td>5–20</td>
<td>30</td>
<td>1</td>
<td>13.6</td>
<td>3.3</td>
<td>3.0</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>Column</td>
<td>0.2</td>
<td>55 000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>The same as the corresponding soil layer</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slab</td>
<td>0.2</td>
<td>55 000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Note: $c'$ is the effective cohesion of soil; $\phi'$ is the effective internal friction angle.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
4. Modification of the Chai et al. (2009) method

4.1 Modification of the effect of $\alpha$

Based on the results reported by Chai et al. (2009) and the numerical and laboratory model test results from this study, the function $f(\alpha)$ has been modified as follows

$$f(\alpha) = \begin{cases} 0.75 - 2.5(\alpha) & (\alpha \geq 0.20) \\ 0.4 - 1.0(\alpha) & (0.20 < \alpha < 0.45) \\ 0 & (\alpha > 0.45) \end{cases}$$

The comparison of the modified function with the original one for a column–slab system is given in Figure 9. Referring to the results in Figures 5 and 7, it is further proposed that for the without-slab case, the effect of $\alpha$ can be expressed by a function $f_1(\alpha)$ as follows

$$f_1(\alpha) = f(\alpha)(0.775 + 0.5\alpha)(0.10 \leq \alpha \leq 0.45)$$

Most empirical constants in Equations 7 and 8 were back-fitted from the numerical results.

4.2 Considering the effect of $p/s_u$ and $s_u$

To include $p$ and $s_u$ in the equation for evaluating the $H_c$ value, the following approach was taken.

(a) Find an $H_c$ value (by trial and error) which can result in the same settlement as the measured value from the model test or from FEA for column–slab cases, designated as $H_{c(meas)}$ and $H_{c(FEA)}$ respectively.

(b) Determine $H_c'$ as follows

$$H'_c = H_c f(\alpha) g(\beta)$$

where $g(\beta)$ is a function expressing the effect of $\beta$ and can be expressed as follows (Chai et al., 2009)

$$g(\beta) = \begin{cases} 1.62 - 1.6\beta & (0.2 \leq \beta \leq 0.7) \\ 0.5 & (0.7 \leq \beta \leq 0.9) \end{cases}$$

(c) Then a ratio of $H_{c(meas)}/H'_c$ or $H_{c(FEA)}/H'_c$ can be obtained. Define a dimensionless parameter, $\gamma$, as
where $p_a$ is the atmospheric pressure. The relationship between the ratio of $H_{\text{limeal}}/H_c^c$ or $H_{\text{limeal}}/H_c^c$ and $\gamma$ is shown in Figure 10. The FEA values are scattered. A possible reason is that the proposed functions of $f(\alpha)$ and $g(\beta)$ are approximations of the real situation. It has been confirmed that for fixed values of $\alpha$ and $\beta$, the $H_{\text{limeal}}/H_c^c \sim \gamma$ relationship is much less scattered.

(d) Finally, an empirical equation has been proposed to consider the effect of $\gamma$ on the calculated $H_c$ value as follows:

\begin{equation}
H_c = \frac{H_{\text{limeal}}}{H_c^c} = 0.27\ln(\gamma) + 0.41
\end{equation}

Then it is proposed that:

\begin{enumerate}
  \item $H_c = H_c f(\alpha)g(\beta)h(\gamma)$ for with-slab case
  \item $H_c = H_c f(\alpha)g(\beta)h(\gamma)$ for without-slab case
\end{enumerate}

5. Method for calculating the settlement–time curve

5.1 Degree of consolidation

Chai and Pongsivasathit (2010) presented a method for determining the average degree of consolidation ($U(t)$) of floating-column-improved soft soil deposit using a two-soil layer consolidation theory (Zhu and Yin, 1999) as illustrated in Figure 11.

\begin{equation}
\gamma = \frac{p_p^{1.5}}{s_u^{0.5}}
\end{equation}

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure10.png}
\caption{Figure 10. Variation of $H_{\text{limeal}}/H_c^c$ and $H_{\text{limeal}}/H_c^c$}
\end{figure}

The methods for determining $k_{v1}$ and $m_{v1}$ for the upper layer (Figure 11), and the thicknesses ($H_{1c}$ and $H_{2c}$) of both layers are as follows:

\begin{enumerate}
  \item $m_{v1} = \frac{1}{\alpha D_c + (1 - \alpha)D_k}$
  \item $k_{v1} = \left(1 + \frac{2.5H_{1c}^2 k_h}{\mu k_s d^2 k_v}\right)$
\end{enumerate}

where $D_c$ and $D_k$ are the constrained moduli of the column and the surrounding soil, $H_{1c}$ is the thickness of the upper layer, and $\mu$ can be expressed as follows (Hansbo, 1981):

\begin{equation}
\mu = \ln \frac{n}{s} + \frac{k_h}{k_s} \ln(s) - \frac{3}{4} + \frac{8H_{1c}^2 k_h}{3d^2 k_c}
\end{equation}

where $n = d_c/d_s$, $s = d_c/d_s$ ($d_c$, $d_s$ and $d_e$ are diameter of column, smear zone and unit cell which represents a column and its improvement area, respectively), $k_c$ and $k_v$ are the hydraulic conductivities of the column and the smear zone, respectively. It should be noted that even for $k_c = k_v$, $k_{v1}$ is larger than $k_v$ because a flow mode toward the column is assumed in calculating the value of $k_{v1}$. For a soil–cement column formed by an in situ deep mixing method, the authors suggest not considering the effect of the smear zone when applying Equation 17 to a soil-cement column.

Considering the higher stiffness, and therefore the coefficient of consolidation of a stabilised slab, it is proposed to exclude the slab from $H_c$, which implies that the bottom of the slab is permeable. So, for the thickness of the upper layer and the lower layer, Chai and Pongsivasathit (2010) proposed (by trial and error) that $H_{1c} = H_c - H_c/2$. As for $H_{2c}$, to consider the relatively large
strain in the unimproved layer, it has been proposed to use the average thickness (before and after consolidation) of the lower layer – that is, \( H_{2o} = H_{20} - s_f / 2 \). \( H_{20} \) is the initial thickness of the lower layer and \( s_f \) is the final consolidation settlement of the system (assuming that most compression is from the lower layer). Note that the thicknesses of the layers for the degree of consolidation are different from those for the settlement calculation.

### 5.2 Settlement–time curve

Generally, the settlement consists of two parts: the compression of the column improved layer \((s_1)\) with a thickness of \(H_1 = H_k - H_{k1}\) and the compression of the unimproved layer plus \(H_{k1}\) layer \((s_2)\) with a total thickness of \(H_2\). The equations for calculating \(s_1\) and \(s_2\) values are as follows:

18.  
\[
s_1(t) = \sum_{i=1}^{n} \frac{\Delta p_{i1} H_{1i} U_{1i}(t)}{D_{ci} \alpha + (1 - \alpha) D_{ki}}
\]

19.  
\[
s_2(t) = \sum_{i=1}^{n} H_{2i} \frac{\lambda_i}{1 + e_{0i}} \ln \left[ 1 + \frac{\Delta p_{ci} \lambda_i}{\sigma_{0vi} U_{2i}(t)} \right]
\]

where \(H_{1i}\) and \(H_{2i}\) are the thickness of subsoil layers in \(H_1\) and \(H_2\) respectively; \(U_{1i}(t)\) and \(U_{2i}(t)\) are the average degree of consolidation of the subsoil layers in \(H_1\) and \(H_2\) at time \(t\) respectively; \(\sigma_{0vi}\) is the initial vertical effective stress in the sub-layer \(H_{2i}\); \(e_{0i}\) is the initial void ratio; \(\lambda_i\) is the slope of virgin compression line in \(e - \ln p'\) plot in the corresponding subsoil layer; \(\Delta p_{ci}\) and \(\Delta p_{v1}\) are the total vertical stress increments in sub-layer \(H_{1i}\) and \(H_{2i}\) respectively; \(D_{ci}\) and \(D_{ki}\) are the constrained moduli of the column and the surrounding soil of sub-layer \(H_{2i}\) respectively and they can be calculated as follows:

20.  
\[
D_{ci} = \frac{E_i(1 - \nu_i)}{(1 + \nu_i)(1 - 2\nu_i)}
\]

21.  
\[
D_{ki} = \frac{(1 + e_{0i}) \sigma_{0vi}}{\lambda_i}
\]

where \(E_i\) is Young’s modulus, \(\nu_i\) is Poisson’s ratio and \(\sigma_{0vi}\) is the average effective vertical stress in the corresponding subsoil layer. In Equations 19 and 21, \(\lambda_i\) is the slope of the unloading-reloading line in the \(e - \ln p'\) plot, is used instead of \(\lambda_i\), in case the subsoil is in an overconsolidated state – that is, \([\sigma_{0vi} + \Delta p_{ci} U_{ci}(t)]\) or \([\sigma_{0vi} + \Delta p_{v1} U_{v1}(t)]\) is less than \(p_k\) (the consolidation yield stress).

Finally, the settlement (total compression), \(s(t)\), can be expressed as follows:

22.  
\[
s(t) = s_1(t) + s_2(t)
\]

Considering the complicated expression for calculating the excess pore pressure distribution in a two-layer system, it was found that using the average degree of consolidation to all sub-layers, an acceptable settlement–time curve can be predicted, and it is suggested this be used for most practical purposes.

### 6. Application of the proposed method to laboratory results

The parameters used for calculation are listed in Tables 2 and 4. The calculated results are compared with the measured ones in Figure 12.

The proposed method yields a good prediction of the test data for the cases using both the Ariake clay and the mixed soil. The calculated final consolidation settlement of case L1 is less than the measured settlement due to the poor quality of the column around its bottom end, which was verified by the post-test investigation. For case L3 (higher loading case), the calculated result by the proposed method is much better than that by the method of Chai and Pongsivasathit (2010). For this case, the value of \(h(t)\) is about 1-27 and if the effect of \(p\) and \(s_a\) is not considered, the \(H_k\) value will be underestimated.

### 7. Application of the proposed method to case histories

The modified method has been applied to four case histories – three from Fukuoka, Japan (Fukuoka cases), and one from Saga, Japan (Saga case).

<table>
<thead>
<tr>
<th>Case</th>
<th>(\lambda)</th>
<th>(e_0)</th>
<th>(c_1: \times 10^{-2} ) m²/day</th>
<th>(c_2: \times 10^{-2} ) m²/day</th>
<th>(k_1: \times 10^{-4} ) m/day</th>
<th>(k_2: \times 10^{-4} ) m/day</th>
<th>(H_{k1}: m)</th>
<th>(H_{k2}: m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1</td>
<td>0.331</td>
<td>2.383</td>
<td>10.71</td>
<td>0.34</td>
<td>1.38</td>
<td>1.02</td>
<td>0.39</td>
<td>0.24</td>
</tr>
<tr>
<td>L2</td>
<td>0.287</td>
<td>1.72</td>
<td>10.17</td>
<td>0.35</td>
<td>1.37</td>
<td>1.02</td>
<td>0.31</td>
<td>0.34</td>
</tr>
<tr>
<td>L3</td>
<td>0.149</td>
<td>1.240</td>
<td>10.72</td>
<td>0.34</td>
<td>1.38</td>
<td>1.02</td>
<td>0.41</td>
<td>0.25</td>
</tr>
<tr>
<td>L4</td>
<td>0.017</td>
<td>0.86</td>
<td>56.08</td>
<td>0.32</td>
<td>1.59</td>
<td>0.65</td>
<td>0.32</td>
<td>0.35</td>
</tr>
<tr>
<td>L5</td>
<td>0.345</td>
<td>0.25</td>
<td>43.25</td>
<td>0.30</td>
<td>1.38</td>
<td>0.65</td>
<td>0.45</td>
<td>0.24</td>
</tr>
</tbody>
</table>

| Table 4. The adopted parameters for calculating degree of consolidation |
Ground Improvement

Consolidation settlement of floating-column-improved soft clayey deposit
Pongsivasathit, Chai and Ding

7.1 Fukuoka case histories
The Fukuoka cases were described by Chai et al. (2009). The cross-sections of the three cases are shown in Figures 13–15 and the construction time as well as some of design parameters are given in Table 5. The field measurements indicate that there were settlements below the soft layers for these three cases. Since the proposed method only considers the compression of the soft layer, the settlement differences ($\Delta s$) between measurement points S-1 and S-2 in Figures 13–15 are used to compare with the calculated values. For calculating the degree of consolidation, the Young’s moduli of the columns and the slabs were assumed as 100 times the corresponding design value of unconfined compression strength (except for case-F3), and a Poisson’s ratio of 0.2. For case-F3, the amount of the cement used was between the values of case-F1 and case-F2, but the $q_u$ value is obviously lower. Since there are no field measured data available, and based on local experience, $E = 70\,000\,\text{kPa}$ was assumed and used in calculations. The Zhu and Yin (1999) solution considers linear variation of the total stress increment in a two-layer system as shown in Figure 16. Under embankment load, $\alpha_b$, $\alpha_t$, and $\alpha_2$ can be approximately calculated by the Osterberg (1957) method. Based on the test results using the undisturbed samples from the sites (FNHO, 2003), the other parameters for calculating the degree of consolidation ($U(t)$) are listed in Table 6. $c_v$ and $k_v$ are the values corresponding to the average consolidation pressure.

7.2 Saga case histories
In 2007, in Saga, Japan, a full-scale test embankment was constructed on soft clay (Ariake clay) deposit improved by floating columns. At the test site, the thickness of soft Ariake soil is about 11 m. The embankment had a fill thickness of 6.5 m, with a base dimension of $45.1\times23.7\,\text{m}$, and top dimension of $33.4\times12.0\,\text{m}$ (Igaya et al., 2010). The cross-section of the embankment is shown in Figure 17.

The columns were constructed with a diameter of about 1.2 m and arranged in a square pattern. The spacing was about 1.9 m which results in an $\alpha$ value of about 31%. The length of the column was about $8.5\,\text{m}$ ($\beta = 76\%$). The backfill used was decomposed granite soil and the compacted backfill had a total unit weight of approximately $18.2\,\text{kN/m}^3$. The construction time was about 109 days. The parameters for calculating the degree of consolidation ($U(t)$) are included in Table 6. For the value of Young’s moduli of the column, $E = 100\,000\,\text{kPa}$ was adopted.
Figure 14. Cross-section of case-F2

Figure 15. Cross-section of case-F3

<table>
<thead>
<tr>
<th>Case</th>
<th>F1</th>
<th>F2</th>
<th>F3</th>
<th>F4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight of backfill, $\gamma_i$: kN/m$^3$</td>
<td>19</td>
<td>19</td>
<td>19</td>
<td>18.2</td>
</tr>
<tr>
<td>The design compressive strength of column, $q_u$: kPa</td>
<td>700</td>
<td>700</td>
<td>400</td>
<td>600</td>
</tr>
<tr>
<td>The amount of the cement mixed with soil for column: kg/m$^3$</td>
<td>140</td>
<td>100</td>
<td>130</td>
<td>150</td>
</tr>
<tr>
<td>Column length: m</td>
<td>6.5</td>
<td>5.5</td>
<td>5</td>
<td>8.5</td>
</tr>
<tr>
<td>$\alpha$: %</td>
<td>21.7</td>
<td>9</td>
<td>30</td>
<td>31</td>
</tr>
<tr>
<td>$\beta$: %</td>
<td>76</td>
<td>85</td>
<td>47</td>
<td>76</td>
</tr>
<tr>
<td>Thickness of slab: m</td>
<td>0.5</td>
<td>2.5</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>The design compressive strength of slab, $q_u$: kPa</td>
<td>300</td>
<td>300</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>The amount of the cement mixed with soil for slab: kg/m$^3$</td>
<td>80</td>
<td>80</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Time of construction: day</td>
<td>96</td>
<td>90</td>
<td>124</td>
<td>109</td>
</tr>
<tr>
<td>Final embankment thickness: m</td>
<td>8</td>
<td>8.3</td>
<td>10</td>
<td>6.5</td>
</tr>
</tbody>
</table>

Table 5. Some design and geometry parameters for case histories
referring to the test results using the samples retrieved from the field.

7.3 Comparing the results
The comparisons of the settlement curves are given in Figure 18. The calculated results using the Chai and Pongsivasathit (2010) methods are also included in the figures for comparison. It can be seen that the modified method yielded a better fit to the measured results. The lower predicted settlement for case-F1 by the modified method is due to the consideration of the effect of $p$ and $s_u$. The $h(y)$ value is about 0.71. For case-F2, the value of $h(y)$ is less than unity but the modified $f(x)$ function results in a larger value, and the overall effect on the $H_c$ value is about the same as the previous method. For case-F3, the $h(y)$ value is about 1-03. As for case-F4, both the effects of $h(y)$ and the without-slab

<table>
<thead>
<tr>
<th>Case</th>
<th>Fukuoka cases</th>
<th>Saga case F4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H$: m</td>
<td>8-5</td>
<td>6-5</td>
</tr>
<tr>
<td>$H_c$: m</td>
<td>6-23</td>
<td>4-92</td>
</tr>
<tr>
<td>$\sigma_0$: kN/m²</td>
<td>2-12</td>
<td>1-44</td>
</tr>
<tr>
<td>$\sigma_1$: kN/m²</td>
<td>152-00</td>
<td>157-70</td>
</tr>
<tr>
<td>$\sigma_2$: kN/m²</td>
<td>146-96</td>
<td>153-23</td>
</tr>
<tr>
<td>$\alpha$: %</td>
<td>21-7</td>
<td>9-0</td>
</tr>
<tr>
<td>$\beta$: %</td>
<td>76-5</td>
<td>84-6</td>
</tr>
<tr>
<td>$s_u$ at the end of column: kN/m²</td>
<td>22-50</td>
<td>20-00</td>
</tr>
<tr>
<td>Calculated final settlement: m</td>
<td>0-289</td>
<td>0-279</td>
</tr>
<tr>
<td>Layer $H_1$</td>
<td>$c\nu_1$: m²/day</td>
<td>0-596</td>
</tr>
<tr>
<td>$k\nu_1$: $\times 10^{-4}$ m/day</td>
<td>3-387</td>
<td>2-347</td>
</tr>
<tr>
<td>Layer $H_2$</td>
<td>$c\nu_2$: m²/day</td>
<td>0-035</td>
</tr>
<tr>
<td>$k\nu_2$: $\times 10^{-4}$ m/day</td>
<td>2-668</td>
<td>2-022</td>
</tr>
</tbody>
</table>

Table 6. Parameters for calculating the degree of consolidation

Figure 17. Cross-section of the test embankment in Saga, Japan
condition contributed to a smaller predicted settlement (Figure 18(d)) by the modified method.

Generally, the calculations resulted in a slower settlement rate at the early stage for the Fukuoka cases. The possible reason is that the $c_v$ values used correspond to the normally consolidated state, but for the three cases initially the subsoil was in an overconsolidated state.

8. Conclusions

Based on the laboratory model test and finite-element analysis (FEA) results, a modified method for calculating the consolidation settlement–time curve of floating soil-cement column improved soft clayey soil deposit, with or without a surface cement-stabilised slab system, has been proposed. The main modification is in the method for determining the thickness ($H_c$) of a part of the column improved layer near the end of the column, which is treated as an unimproved layer in settlement calculations. The explicit equations for calculating the value of $H_c$ have been proposed as a function of area improvement ratio, depth improvement ratio, load intensity and the undrained shear strength of the soil.

The proposed method was applied to calculate the settlement–time curves of the laboratory model tests and four field case histories in Japan. Comparisons of the measured and calculated results show that the proposed method yielded satisfactory predictions. It is suggested that the proposed method can be used to design the soft ground improvement using floating columns with/without a cement-stabilised slab on the ground surface.

REFERENCES


Chai JC, Miura N, Kirekawa T and Hino T (2009) Settlement


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