Settlement prediction for soft ground improved by columns

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A simple method of calculating the consolidation settlement of soft subsoil improved by a floating cement-stabilised column–slab system subjected to embankment loading has been proposed, and the method has been applied to three case histories in Fukuoka, Japan. The proposed method adopts the methodology of treating a part of the soil–cement column improved layer with a thickness of Hc as an unimproved layer and calculating its compression using the properties of the soft soil only.

Based on coupled finite-element analysis results using a unit cell model, two bilinear functions with variables of the improvement area ratio (c) and the improvement depth ratio (β) by the columns have been proposed to determine the value of Hc. Both the proposed method and the method currently used in Japan (proposed by the Japanese Institute of Construction Engineering (JICE)) were used to calculate the settlement (compression) of the soft layers for three case histories in Fukuoka, Japan. Comparing the calculated results with the field values shows that the proposed method yielded much better predictions than that of the JICE method. It is suggested that the proposed method can be used to design the improvement of soft clayey ground using the floating cement-stabilised column–slab system.

NOTATION

- D: constrained modulus (M/LT²)
- E: Young’s modulus (M/LT²)
- c₀: initial void ratio
- H: thickness of a soft deposit (L)
- Hc: thickness of a part of soil–cement column improved layer near the end of the column treated as an unimproved layer in settlement calculation (L)
- Hl: length of a column (L)
- Hw: thickness of a slab (L)
- Hz: thickness of an unimproved layer (L)
- k v, k h: hydraulic conductivities in vertical and horizontal directions, respectively (LT⁻¹)
- Lc: length of part of a column from the end of the column to a specified point (L)
- LR: length ratio
- M: slope of critical state line in p’–q plots where p’ is mean effective stress and q is deviator stress
- n: stress concentration ratio
- OCR: over consolidation ratio
- p: distributed load (M/LT²)
- q₀: unconfined compression strength (M/LT²)
- RE: relative error
- RS: relative settlement (L)
- SR: settlement ratio
- Wl: liquid limit (%)\n- Wc: natural water content (%)
- Wp: plastic limit (%)
- α: improvement area ratio
- β: improvement depth ratio
- Y: total unit weight (M/LT²)
- δc: settlement of a column at a point considered (L)
- δs: settlement of soil at the periphery of a unit cell at a point considered (L)
- κ: slope of unloading-reloading line in e-In(p’) plot
- λ: slope of virgin consolidation line in e-In(p’) plot
- ν: Poisson’s ratio
- σ′eo: initial effective vertical stress (M/LT²)
- σ′e: yield stress (M/LT²)

1. INTRODUCTION

Deep cement mixing, normally forming soil–cement columns in the ground, is a widely used soft ground improvement method (e.g. Bergado et al. 1994; Broms and Boman, 1979). To reduce the construction cost and minimise the impact on the ground environment, a system has been developed with a low improvement area ratio (α = Ac/A, where Ac is the cross-sectional area of a column and A is the total cross-sectional area of a column-improved area). This system uses ‘floating’ cement-stabilised columns combined with a soil–cement slab constructed at the ground surface (Shen et al., 2001). The method has been applied in several field constructions recently (e.g. FNHO, 2003). The slab is generally formed by shallow ground improvement techniques, either by compacting the soil–cement mixture in layers or by overlapping short cement-stabilised columns. This system will be designated here as a floating column–slab system. To design such a system, one of the important aspects is the calculation of the consolidation settlement (compression) of the improved soft deposit.

Composite methods, such as those proposed by Priebe (1976, 1995) and Balaam et al. (1977), are commonly used to calculate the settlement of soft ground improved by column-type inclusions. However, for ground improved by floating...
columns with a low value of $\alpha$, there will be penetration of the columns into the underlying soft soil layer and the composite methods may underestimate the settlement, a phenomenon similar to that encountered in the problem of calculating the settlement of floating group pile foundation (Terzaghi and Peck, 1967). There are generally two different types of method used to include the effect of the relative penetration into the settlement calculation. One of them, namely the design guide of the Japanese Institute of Construction Engineering (JICE, 1999) treats a part of the column-improved layer (Figure 1) as an unimproved layer with a thickness of $H_u$, and calculates the consolidation compression of this layer using the properties of the soft soil. Another method is based on the stress concentration ratio ($n$), defined as the ratio of the vertical stress in the column and in the surrounding soft soil. Bergado et al. (1994) used the $n$ value at the top of the column to calculate the stress increment on the surrounding soft soil and used this to calculate the consolidation settlement of the surrounding soil. Chai et al. (2002) used the $n$ value at the end of the column to calculate the compression of the soil layer below the column.

The method of treating a part of the column-improved layer (of thickness $H_l$ near the end of the column as an unimproved layer is simple and it has been widely adopted in Japan. However, there are still debates on how to determine the thickness of $H_u$. The method proposed by JICE (1999) suggests that if $\alpha \geq 30\%$ only the unimproved layer is considered as the main compressive layer; while for $\alpha < 30\%$, one-third of the improved layer near the end of the column needs to be treated as an unimproved layer, which is the same as the concept proposed by Terzaghi and Peck (1967) for calculating the settlement of a floating pile group. Obviously this is a rough estimation and it is impossible to obtain good agreement between the calculated settlements and field measurements for all possible $\alpha$ values using this approach. Another shortcoming of the method is that it does not consider the effect of the depth of improved soil (relative to the thickness of the soft deposit) in determining the value of $H_u$. The methods based on the use of $n$ value are rational, but how to correctly estimate the value of $n$ for a floating column–slab system remains a challenge. For a floating column–slab system, the value of $n$ could vary along the column depending upon the interaction behaviour between the column and the surrounding soil.

In this study, the interaction behaviour between the floating column–slab system and the surrounding soft soil is investigated by finite-element analysis (FEA) using a unit cell model (a single column and its improvement area). Based on the FEA results, a method has been proposed for determining the thickness ($H_u$) of a part of the column-improved layer near the end of the column which is then regarded as an unimproved layer for the purpose of settlement calculation. Finally, the proposed method has been used to calculate the consolidation settlement (compression) of three case histories observed in Fukuoka, Japan (FNHO, 2003). By comparing the predicted values with the field measured data, discussions are made on the usefulness of the method.

### 2. FINITE-ELEMENT ANALYSES AND RESULTS

#### 2.1. Model and parameters

The axisymmetric unit cell model used in the finite-element study is illustrated in Figure 1 together with the boundary conditions adopted. The ranges of the geometric variables and loading intensity used are listed in Table 1. The loading rate adopted was 1 kPa/day (from local construction experience). In the analyses, the soft clayey soil was represented by the modified cam clay (MCC) model (Roscoe and Burland, 1968) and the column and slab as well as the sand layer were modelled as linear elastic materials. Two types of soft subsoil conditions were adopted: one was very soft, like the Ariake clay deposit in Saga, Japan (model ground-1), and another was a soft clayey

![Figure 1. Unit cell model for floating column–slab improved soft subsoil](image)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Base value</th>
<th>Range</th>
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</thead>
<tbody>
<tr>
<td>Thickness of soft layer, $H$: m</td>
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<td>–</td>
</tr>
<tr>
<td>Radius of column, $r$: m</td>
<td>0.5</td>
<td>–</td>
</tr>
<tr>
<td>Length of column, $H_l$: m</td>
<td>8.0</td>
<td>4.0–10.0</td>
</tr>
<tr>
<td>Thickness of slab, $H_s$: m</td>
<td>1.0</td>
<td>–</td>
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<tr>
<td>Area improvement ratio, $\alpha$: %</td>
<td>30</td>
<td>10–40</td>
</tr>
<tr>
<td>Distributed load, $p$: kPa</td>
<td>110 (5.5 m high embankment)</td>
<td>50–160</td>
</tr>
</tbody>
</table>

Table 1. The ranges of geometric variables considered
deposit, like the clayey deposit in Shanghai, China (model ground-2). The assumed model parameters are given in Table 2. An unconfined compression strength ($q_u$) of the cement-stabilised column and the slab was assumed as 550 kPa, and the Young’s modulus ($E$) was estimated as 100$q_u$ (Kitazume, 1996).

It was assumed that the groundwater level was 1.0 m below the ground surface. Referring to the soft Ariake clay deposit in Saga, Japan, it was assumed that the soil layers from the ground surface to 4.0 m depth had an over-consolidation ratio (OCR) varying from 4.0 to 1.2, and below it a uniform OCR value of 1.1. The program used for this modelling is a modified version of Crisp (Britto and Gunn, 1987). Eight-node quadrilateral elements were used to represent all materials. At the column–soil and the slab–soil interfaces, one layer of solid elements with a thickness of 50 mm was used, which could simulate a shear band 25 mm thick.

2.2. Finite-element analysis results

As the main purpose of the analysis is to investigate the interaction behaviour between the column–slab system and the surrounding soft soil, the results presented in this section are focused on the relative settlement between the column and the surrounding soil. For ease of discussion, the following terms are introduced.

| 1 | (a) Improvement depth ratio: $\beta = H_t / H$ |
| 2 | (b) Settlement ratio: $SR = \delta_s / \delta_c$ |
| 3 | (c) Relative settlement: $RS = \delta_s - \delta_c$ |
| 4 | (d) Length ratio: $LR = L_s / H_t$ |

where $H$ is the overall thickness of the soft clayey layer (excluding the slab), as shown in Figure 2, $H_t$ is the length of the column, $L_s$ is the length from the end of the column to a point at which $SR$ or $RS$ satisfies a pre-specified criterion, $\delta_c$ is the settlement of the column at a point considered, and $\delta_s$ is the settlement of the soil at the periphery of the unit cell (corresponding to the middle point between two adjacent columns) at the same elevation selected for measuring $\delta_c$. It is considered that $LR$ is a direct indicator of the degree of relative movement between the column and the surrounding soft soil, and can form the basis for determining the thickness ($H_t$ in Figures 1 and 2) of the part of the column improved layer to be treated as an unimproved layer in the settlement calculation. Larger values of $LR$ mean there is relative settlement between the column and the surrounding soil along a longer portion of the column, and a larger portion of the improved layer needs to be treated as an unimproved layer in the settlement calculation. From the FEA results, the effects of $\alpha$, $\beta$, the assumed ground properties and the loading intensity ($p$) on $LR$ were investigated. The settlement values from the FEA correspond to 2 years elapsed time after load application, at which time the primary consolidation is essentially complete. Unless otherwise described, the results presented are from the FEA using the conditions assumed in model ground-1.

(a) Effect of $\alpha$. As shown in Figure 3, $\alpha$ has a strong influence on $LR$. In cases assuming the criterion $SR = 1.0$, when $\alpha = 10\%$, $LR > 0.8$, and when $\alpha = 40\%$, $LR$ reduced to

<table>
<thead>
<tr>
<th>Depth: m</th>
<th>Soil layer</th>
<th>Young’s modulus $E$: kPa</th>
<th>Poisson’s ratio, $\nu$</th>
<th>$\kappa$</th>
<th>$\lambda$</th>
<th>$M$</th>
<th>$\varepsilon_0$</th>
<th>$\gamma_f$: kN/m$^3$</th>
<th>$k_h$</th>
<th>$k_v$</th>
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<td>0.0–2.0</td>
<td>Surface soil</td>
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<td>0.052 (0.013)*</td>
<td>0.52 (0.13)</td>
<td>1.2</td>
<td>3.30 (1.54)</td>
<td>13.6 (16.5)</td>
<td>3.0 (1.5)</td>
<td>2.0 (1.0)</td>
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<tr>
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<td>Clay-1</td>
<td>0.30</td>
<td>0.065 (0.022)</td>
<td>0.65 (0.22)</td>
<td>1.2</td>
<td>3.30 (1.75)</td>
<td>13.6 (16.0)</td>
<td>3.0 (1.5)</td>
<td>2.0 (1.0)</td>
<td></td>
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<tr>
<td>4.0–8.0</td>
<td>Clay-2</td>
<td>0.30</td>
<td>0.065 (0.022)</td>
<td>0.65 (0.22)</td>
<td>1.2</td>
<td>3.10 (1.54)</td>
<td>13.8 (16.5)</td>
<td>2.1 (1.2)</td>
<td>1.4 (0.8)</td>
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</tr>
<tr>
<td>8.0–12.0</td>
<td>Clay-3</td>
<td>0.30</td>
<td>0.052 (0.022)</td>
<td>0.52 (0.22)</td>
<td>1.2</td>
<td>2.60 (1.36)</td>
<td>14.4 (17.0)</td>
<td>1.1 (1.0)</td>
<td>0.7 (0.5)</td>
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<td>12.0–15.0</td>
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<td>30 000</td>
<td>0.25</td>
<td>18.0</td>
<td>290</td>
<td>14.5 (16.0)</td>
<td>The same as the corresponding soil layer</td>
<td></td>
<td></td>
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<td>Column</td>
<td>55 000</td>
<td>0.20</td>
<td>14.5 (16.5)</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Slab</td>
<td>55 000</td>
<td>0.20</td>
<td>14.5 (16.5)</td>
<td></td>
<td></td>
<td></td>
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</tbody>
</table>

* The numbers in parentheses are for model ground-2.

Table 2. Assumed model parameters
about 0·15. Also, the influence of $\alpha$ is non-linear. The rate of reduction on $LR$ with increase of $\alpha$ is much higher for $\alpha < \sim 20$.

(b) Effect of $\beta$. The variation of $LR$ with $\beta$ is given in Figure 4. For $SR = 1.0$, $LR$ reduced with increasing $\beta$. With the increase of the relative improvement depth, the thickness of the unimproved layer is reduced, and also for the assumed soft deposit, its stiffness and strength increase with depth. All these factors tend to reduce the relative penetration of the column into the surrounding soft soil. However, for $SR = 0.9$, $LR$ was unchanged or even slightly increased with the increase of $\beta$. This is because both $SR$ and $LR$ are relative values. With an increase in the improvement depth, the settlement of the system becomes small and a small difference of settlements can result in a $SR$ value more than 0.9. It has been verified that using the relative settlement ($RS = \delta - \delta_c$) (with a limit such as 20 mm) instead of a ratio, $LR$ reduced with the increase of $\beta$ (Figure 5).

(c) Effect of loading intensity ($p$). $LR$ increased non-linearly with increasing $p$ (Figure 6). This can be explained by the observation that an increase in $p$ increases the yielded zone between the column and the surrounding soil, resulting in greater relative penetration of the column into the soil.

(d) Effect of soil type. A line for $SR = 0.95$ obtained using the conditions of model ground-2 (generally stronger than model ground-1) is included in Figure 3 for comparison. It can be seen that $LR$ reduced with an increase in the stiffness and the strength of the soft soil. This means the stronger the clayey deposit, the smaller the relative penetration of the column, and thus in the settlement calculation, the smaller the $H_c$ value.

3. PROPOSED METHOD AND COMPARISON OF CALCULATED VALUES WITH FEA RESULTS

3.1. Method for determining $H_c$ value

The FEA results from the unit cell model indicate that $LR$, and therefore the thickness of the part of the column-improved layer ($H_c$) to be treated as an unimproved layer, is a function of $\alpha$, $\beta$, loading intensity ($p$) and soil type. However, the effects of the loading intensity ($p$) and soil type are more complicated and interactive, and the data from this study are not sufficient for proposing a generally valid equation to include the separate effects of these two factors on determining $H_c$. For simplicity, at present it is assumed that $H_c$ is a function of $\alpha$ and $\beta$ only, as follows

$$H_c = H_c f(\alpha)f_1(\beta)$$
Based on the results in Figures 3 to 5 and for ease of use, bilinear functions have been chosen for \( f(\alpha) \) and \( f_1(\beta) \), as shown in Figures 7(a) and (b), respectively. These functions can be written mathematically as follows:

\[
f(\alpha) = \begin{cases} 
\frac{8}{15} - \frac{\alpha}{0.75} & (10\% \leq \alpha \leq 40\%) \\
0 & (\alpha > 40\%) 
\end{cases}
\]

\[
f_1(\beta) = \begin{cases} 
1.62 - 1.6\beta & (20\% \leq \beta \leq 70\%) \\
0.5 & (70\% < \beta \leq 90\%) 
\end{cases}
\]

It is considered that the ranges for \( \alpha \) and \( \beta \) adopted cover most practical situations. Figures 3 to 5 only provide the information about the variation of LR with \( \alpha \) and \( \beta \), but they do not provide any information on what kind of criterion for SR and/or RS can be used to determine \( H_c \). The numbers in Equations 6 and 7 have been determined by trial and error under the condition that the calculated settlements provide a good fit to the FEA results using the unit cell model. This implies that the method may best fit a soil condition somewhere between model ground-1 and 2 under a surcharge load around 100 kPa.

In the settlement calculation, the compression of the unimproved layer \( (H_L) \) and the part of the column improved layer \( (H_c) \) is calculated using just the properties of the soft soil. The compression of the improved layer above the \( H_c \) layer \( (H_c - H_u) \) is computed using an area-weighted average value of the constrained modulus \( (D) \) of the column and the surrounding soft soil deforming under one-dimensional (1-D) conditions, which is the same as for the JICE method. However, under embankment loading, the load-spreading effect in the ground can be considered using Osterberg’s (1957) solution, which is a plane strain solution for the vertical stress increment in homogeneous ground caused by an embankment load.

### 3.2. Comparison of calculated values with the FEA results

Figure 8 compares the calculated settlement variation with \( \alpha \) to that obtained using the FEA approach and the unit cell model \( (\beta = 73\% \text{ and } p = 110 \text{ kPa}) \). Generally, the results calculated by the proposed method match the FEA results well. There is some over-prediction for the conditions of model ground-2, and under-prediction for the conditions of model ground-1 at lower values of \( \alpha \). Also, for model ground-1, the values calculated using the method of JICE (1999) are included to allow comparison with the values predicted by the proposed method. It can be seen that for \( \alpha < 30\% \), the JICE method over-predicts the settlement significantly. Figure 9 shows the comparison of the variation of settlements with \( \beta \) for the case where \( \alpha = 20\% \). Although the proposed method predicted a larger settlement for lower values of \( \beta \), generally the agreement between the calculated and the FEA results is good.
increasing $\alpha$ value, and even at lower values of $\beta$, the difference between the values predicted by the proposed method and the FEA results becomes smaller. The JICE method over-estimates the settlement and the over-estimation increases with increasing values of $\beta$. Figure 10 shows a comparison of the predicted and the FEA results for different loading intensity ($p$). This figure shows that the larger the value of $p$, the more non-linear is the variation of the settlement with $\alpha$. Using the assumed bilinear function $f(\alpha)$ can not simulate this effect well, and as a result, the predictions under-estimate the settlement for $p = 160$ kPa at lower values of $\alpha$.

4. APPLICATION OF THE PROPOSED METHOD TO THREE CASE HISTORIES

4.1. General description

In 2002, along the national road No. 208, at Shaowa-Biraki, Fukuoka, Japan, three test embankments (referred to here as case-1, -2 and -3) were constructed on soft clayey silt/silty clay deposits, each of which had been improved by the installation of either floating cement-stabilised columns with an overlying cement-stabilised slab or else simply a cement-stabilised column system (FNHO, 2003). The plan dimensions and relative locations of the three cases are shown in Figure 11. The embankment for case-1 had a length of about 64 m, while for case-2 it was 68.8 m. Case-3 involved the construction of an embankment behind a retaining wall and the embankment had a length of about 38 m. For all three cases, the cement-stabilised columns were constructed by the dry jet mixing method (Chai et al., 2005), and the diameter of the columns was about 1-0 m and they were arranged in a square pattern. However, the length of the columns, the improvement area ratio ($\alpha$) and the design strength of the columns were varied from case to case. The thickness of the slab and the method for constructing the slab were also different in each case. The backfill used was decomposed granite soil and the compacted backfill had a total unit weight of approximately 19-0 kN/m$^3$.

4.1.1. Case-1. The cross-sectional geometry of the embankment and the thickness of the soft layer as well as the thickness of the slab and the length of the cement-stabilised columns for this case are illustrated in Figure 12. The design compressive strength ($q_u$) of the column was 700 kPa. Based on laboratory test results, for this case the amount of the cement mixed with the soil in the field was 140 kg/m$^3$. The $\alpha$ value was 21-7% and $\beta$ was 76%. The thickness of the slab for this case was 0-5 m with a design value of $q_u$ of 300 kPa and the amount of the cement used was 80 kg/m$^3$. The slab was constructed by mixing the cement with the surface soil using a mixing machine and compacting it by a bulldozer. The embankment construction started on 5 August 2002 and finished on 30 October 2002 and the final embankment thickness was 8-0 m (FNHO, 2003).

4.1.2. Case-2. The cross-section of case-2 is shown in Figure 13. The design value of $q_u$ of the columns was 700 kPa, and the amount of the cement used was 100 kg/m$^3$. The $\alpha$ value was 9%, and for this case, the length of the columns varied...
from 5.5 m ($\beta = 85\%$) at the left-hand side of Figure 13 (for about two-thirds of the total width) to 5.0 m ($\beta = 77\%$) at the right side (for about one-third of the total width). This case had a cement-stabilised slab with a thickness of 2.5 m, which was constructed by overlapped cement-stabilised columns 3.0 m in diameter. The design value of $q_u$ of the slab was 300 kPa and the amount of the cement used was 80 kg/m$^3$. Construction of the embankment started on 16 August 2002 and finished on 6 November 2002, and the total thickness of the embankment fill was 8.3 m (FNHO, 2003).

4.1.3. Case-3. The longitudinal cross-section of case-3 is illustrated in Figure 14(a) and the transversal cross-section at the settlement measuring locations (S-1 and S-2 in Figure 14 (a)) is shown in Figure 14(b). The design $q_u$ value of the cement-stabilised columns was 400 kPa, and the amount of the cement used was 130 kg/m$^3$. The length of the columns was 5.0 m with an $\alpha$ value of 30% and $\beta$ of 47%. For this case, there was no cement-stabilised slab constructed and only a sand mat about 0.5 m thick was placed on the ground surface. The slope of the embankment was reinforced and had a steep slope angle ($V:H = 1:0.5$). Construction of the embankment started on 1 July and ended on 28 October 2002, and resulted in an overall fill thickness of 10.0 m (FNHO, 2003).
4.2. Soil properties and calculated settlement (compression)

4.2.1. Case-1. The soil profiles as well as some physical and mechanical properties measured on samples recovered at borehole No. 1 (Figure 11), which was adjacent to the site of case-1, are shown in Figure 15 (FNHO, 2003). The thickness of the soft layer was about 9.0 m, and the groundwater level was about 1.1 m below the ground surface. Curves showing the measured settlement \((S)\) against elapsed time \((t)\) at the measurement points S-1 and S-2 (Figure 12) are depicted in Figure 16. Essentially, the settlement difference between points S-1 and S-2 represents the measured compression \((\tilde{S})\) of the soft layer improved by the floating column–slab system. Based on the soil properties presented in Figure 15, the parameters used in the settlement calculation are listed in Table 3.

Regarding the over-consolidation ratio (OCR), first the values were calculated using the consolidation yield stress \((\sigma_y^c)\) and the initial vertical effective stress \((\sigma_{vo})\) in the ground, and then OCR values of 1.9 and 1.4 were selected for the layers above and below 5.0 m depth, respectively. The Young’s moduli of the columns and the slab were assumed as 100 times the corresponding design value of \(q_u\) (700 kPa for the column and 300 kPa for the slab) and Poisson’s ratio of the column and slab was assumed to be 0.2. The stress increments in the ground under the embankment load were calculated by Osterberg’s solution (Osterberg, 1957).

The compressions of the soft layers were calculated by the method suggested by JICE and by the proposed method. Both measured and calculated values are listed in Table 4. The measured value is corresponding to the last measured data showing in Figure 16. In the table, the relative error \((RE)\) is defined as follows

\[
RE = \frac{(\Delta S)_{\text{cal}} - (\Delta S)_{\text{mea}}}{(\Delta S)_{\text{mea}}} \times 100(\%)
\]

where \((\Delta S)_{\text{cal}}\) and \((\Delta S)_{\text{mea}}\) are the calculated and measured amount of compression of the soft layer. It can be seen that both methods over-estimated the amount of compression, but the proposed method yielded a much better result with an \(RE\) value of 10.5% compared with 57.0% for the JICE method (JICE, 1999).

4.2.2. Case-2. The soil profiles and the soil properties at the site of case-2 (borehole No. 2 in Figure 11) are shown in Figure 17. The thickness of the soft layer was about 8.0 m, and the groundwater level was about 0.7 m below the ground surface. The measured settlement curves at points S-1 and S-2 (Figure 13) are depicted in Figure 18. The parameters adopted for the settlement (compression) calculation were determined using the same method as for case-1, and these values are listed in Table 3. Both the calculated and the measured compressions of the soft layer are given in Table 4. Again, the proposed method yielded a better predicted result with an \(RE\) value of 17.2% compared with 42.9% for the JICE method.

![Figure 16. Measured settlement–time curves of case-1](image-url)

<table>
<thead>
<tr>
<th>Case</th>
<th>Depth of groundwater: m</th>
<th>Soil properties</th>
<th>Depth (m)</th>
<th>(\lambda)</th>
<th>(\varepsilon_0)</th>
<th>(\gamma_v): (kN/m³)</th>
<th>OCR</th>
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<td>Case-1</td>
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<td>2.4</td>
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<td>15.46</td>
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<td>1.4</td>
<td>0.21</td>
<td>1.97</td>
<td>15.26</td>
<td>2.5 (2.55)</td>
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<td>1.95</td>
<td>15.44</td>
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<td></td>
<td>5.4</td>
<td></td>
<td>0.18</td>
<td>1.63</td>
<td>15.10</td>
<td>1.5 (1.38)</td>
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<tr>
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<td>1.83</td>
<td>15.75</td>
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<tr>
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<td>1.52</td>
<td>16.66</td>
<td>1.6 (2.37)</td>
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</table>

* Numbers in parentheses are calculated from consolidation yield stress \((\sigma_y^c)\) and initial vertical effective stress \((\sigma_{vo})\) in the ground. The numbers not in parentheses are adopted in settlement calculation.

Table 3. Soil properties for settlement calculation at sites of case-1, -2 and -3
4.2.3. Case-3. The soil profiles and the soil properties at the site of case-3 (borehole No. 3 in Figure 11) are shown in Figure 19. The thickness of the soft layer was about 10.6 m at the settlement measuring point, and the groundwater level was about 0.9 m below the ground surface. The measured settlement curves are given in Figure 20. This case did not have a cement-stabilised slab, but considering the improvement area ratio of 30%, and a sand mat as well as compacted decomposed granite soil backfill above the columns, the relative settlement between the columns and the surrounding soil at ground surface should be very limited, and the same calculation methods were used to calculate the compression of the soft layer. The computed values are compared with the measured settlements in Table 4. For this case, the $\Delta S$ value was 30%, and the method of JICE under-evaluated the $\Delta S$ value with an $RE$ value of $4.8\%$. The proposed method yielded a slight over-prediction ($RE = 2.3\%$).

5. CONCLUSIONS

A simple method of calculating the consolidation settlement of soft subsoil improved by a system of floating cement-stabilised columns and a surface slab, and subjected to embankment loading has been proposed based on finite-element analysis results using a unit cell model (a single column and its improvement area). The proposed method was applied to three case histories.
The proposed method. In this method, part of the cement-stabilised column improved layer near the end of the columns, with a thickness of $H_c$, is treated as an unimproved layer, and its compression is computed using the properties of soft soil only. Two bilinear functions with variables of the improvement area ratio ($\alpha$) and the improvement depth ratio ($\beta$) have been proposed to determine the value of $H_c$.

Validity of the method. The proposed method was applied to the calculation of the consolidation settlement (compression) of soft layers for three case histories in Fukuoka, Japan, in which the soft soil layers were improved by floating cement-stabilised column–slab systems. Comparison of predictions of this method with the field measured values shows that the proposed method yielded satisfactory predictions. It is suggested that the proposed method can be used to design the improvement of soft clayey soil using cement-stabilised column–slab systems.

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REFERENCES


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