Lowland Technology International 2017; 19 (2): 99-110
International Association of Lowland Technology (IALT): ISSN 1344-9656

Research Paper

Settlement-time curve calculation of soil-cement column and slab improved soft clay deposit

S. Pongsivasathit [1], P. Voottipruex [2] and J.C. Chai[3]

ARTICLE INFORMATION

Article history:

Received: 27 June 2017
Received in revised form: 21 August 2017
Accepted: 25 August 2017
Publish on: 08 September, 2017

Keywords:

Floating soil-cement column
Embankment
Geotechnical engineering

ABSTRACT

The consolidation behavior of soft clayey deposit improved by a floating soil-column with a cement stabilized slab on the ground surface has been investigated by laboratory model test using a symmetric unite cell model and finite element analysis (FEA). The effects of thickness and undrained shear strength of slab on the relative penetration of column into surrounding soil were quantified. Based on the results, the method proposed Pongsivasathit et al. for calculating the settlement of a floating column improved soft subsoil has been modified. The main modification is the equation for calculating the value of H_c, which is the thickness of a soil layer at the bottom of the column improved zone. And in settlement calculation, H_c layer has been treated as an unimproved layer. The effectiveness of proposed method has been verified by comparing with the measurement results of field case histories and laboratory model tests.

1. Introduction

In the construction of various structures on highly compressible, saturated soft clay deposit, low bearing capacity and large settlement are common problems to deal with. The deep cement-soil mixing (DCM) is one of the most suitable methods to improve the clay deposit (e.g. Broms and Boman, 1979; Bergado et al., 1994). Recently, to reduce the construction cost and minimize the impact on the ground environment, a method of improving soft clay deposits by floating soil-cement columns, with or without a cement stabilized slab on the ground surface, is increasingly being applied in the engineering practice (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.

The Japanese Institute of Civil Engineering (JICE, 1999) proposed a design method for calculating the final consolidation settlement of the floating soil-cement column improved soft subsoil and it was widely used in Japan. In this method, the relative penetration is dependent on the area improvement ratio ($\alpha = A_c/A_e$, A_c and A_e are the cross-section area of the column and the cross-section of the unit cell which represents a column and its improvement area) only. The main compression layer means that the compression is calculated by the properties of the soft soil alone without considering the effect of the columns. However, in many cases, the calculated values did not agree with the field data (Chai et al., 2009).

Using the results of laboratory model test and finite element analysis (FEA), Chai et al. (2009) proposed a method for calculating the final consolidation settlement

[1] Assistant Professor, Department of Civil Engineering, Faculty of Engineering, Rajamangala University of Technology Thanyaburi, THAILAND, supasit.p@en.rmutt.ac.th
[2] Professor, Department of Teacher Training in Civil Engineering, King's Mongkut University of Technology North Bangkok, THAILAND, pnv@kmutnb.ac.th
[3] Professor, Department of Civil Engineering and Architecture, Saga University, 1 Honjo Saga 840-8502, JAPAN, chai@cc.saga-u.ac.jp
Note: Discussion on this paper is open until April 2018

of the floating column improved soft deposit. This method can consider the effect of α and the depth improvement ratio ($\beta = H_L/H$, H_L is the length of the column and H is the thickness of the soft deposit excluding the slab). Then, Chai and Pongsivasathit (2010) proposed a method for calculating the consolidation settlement-time curve of the floating column improved clay subsoil. Regarding the degree of the consolidation of the system, it was calculated by the double soil-layer consolidation theory, and the methods for evaluating the equivalent hydraulic conductivity (k) and the coefficient of volume compressibility (m_v) of the part of the column improved layer were proposed. However this method did not include the effect of the load intensity and the undrained shear strength (s_u) of the soft soil into the equations for calculating value of H_c, which was the thickness of the part of the column improved layer to be treated as an unimproved layer in settlement calculations. As a result, it would over-predict the settlement for a stiffer deposit under a lower surcharge load (for example 50 kPa), and under-predict the settlement for a very soft deposit under a higher surcharge load (p > 150 kPa) (Chai et al., 2009).

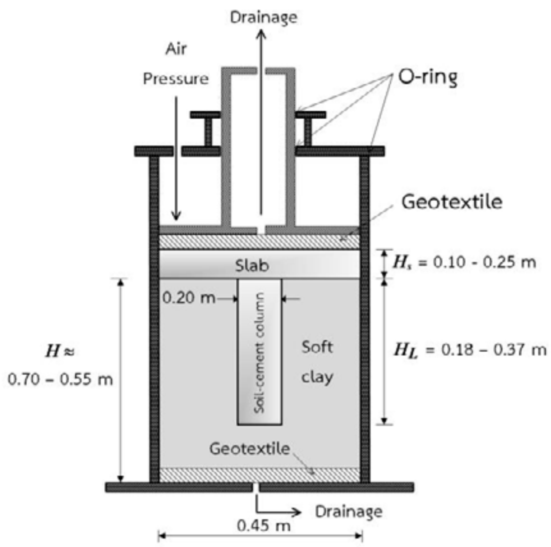
Pongsivasathit et al. (2013) investigated the effect of p and s_u on the behavior of a floating column improved clay deposit by FEA, and the equation for calculating H_c by Chai et al. (2009) was modified. Regarding the FEA of Pongsivasathit et al. (2013), the slab thickness was maintained at 1 m. As a result, the sections of the slab spanning between the columns under a distributed surcharge load would tend to settle more than those immediately above the column. However, the difference settlements of slab at the center of column with the middle point of two adjacent columns would reduce with increasing of the slab thickness due to less flexibility more rigidity. For the engineering practice, there were cases where the thickness of slab was applied over 1 m depth. In this case, this method overestimated the final consolidation settlement (Pongsivasathit et al., 2013).

In this paper, laboratory model tests as well as further FEA were conducted to investigate the interaction behavior of the columns, slab and surrounding soft clay soils. The thickness of the slab was varied. Based on the results, a modified method for predicting the settlement–time curve of floating-column-slab improved soft clay soil had been proposed. The proposed method had been applied to four cases of laboratory model tests and two field case histories in Japan, and its usefulness was verified.

2. Laboratory model test

2.1 Equipment and materials used

The cylindrical model has a diameter of 0.45 m and a height of 0.8 m. The sketch of test set-up is illustrated in **Fig. 1(a)** and the picture of the device is shown in **Fig. 1(b)**. Reconstituted Bangkok clay was used. The properties of the soil are listed in **Table 1**. Cement used was Portland Cement Type I, a typical cement used for ground improvement in Thailand. Non-woven geotextile with a thickness of 3 mm (under zero confining pressure) was used as a drainage material at the bottom and the top of the model ground. The geotextile was made of polypropylene and weighted about 140 g/m^3. To accelerate the rate of consolidation during pre-consolidation process, a mini-prefabricated vertical drain (mini-PVD) was installed at the middle of the model ground during the pre-consolidation process. The mini-PVD was made by folding the geotextile in 3 layers with a cross-section of 30 mm by 9 mm.



(a) A sketch after column and slab installation

(b) Apparatus

Fig. 1. Set-up of the laboratory model test.

Table 1 Properties of soil used in laboratory model tests.

Soil Type	w_L (%)	w_p (%)	γ_t (kN/m^3)	ν	λ	e_0	G_s	k_h ($\times 10^{-5}$ m/day)	k_v ($\times 10^{-5}$ m/day)
Bangkok clay	46.64	35.15	16.321	0.3	0.2496	1.38	2.67	0.432	0.432

Note: w_L is the liquid limit; w_p is the plastic limit; γ_t is the total unit weight; ν is Poisson's ratio; λ is the slope of virgin compression line in e ln p′ plot; e is the void ratio; G_s is the specific gravity; k_h and k_v are the hydraulic conductivity in the horizontal direction and vertical direction, respectively.

Cases tested are listed in **Table 2**. Cases L1, L2, L3 and L4 investigated the effect of slab thickness (H_s). Case L2 and L3 investigated the effect of undrained shear strength of slab ($(q_u)_{slab}$). According to the results of unconfined compression test, the values of $(q_u)_{slab}$ were 400 kPa and 600 kPa for the soil samples mixed by cement of 16.42% and 23.47 % by dry weight, respectively.

2.2 *Preconsolidation stage*

Firstly, 3 layers of the geotextile were put at the bottom of the model as drainage layers. A thin layer of grease was painted on the inside wall of the model to reduce the friction. Then, the soil sample was put in the model layer by layer. Regarding the initial water content of the soil sample, it was about 68%. When the thickness of the soil reached about 800 mm, the mini-PVD was installed (pushed in by a stainless steel rod) in the center of the model, and 3 layers of the geotextile were placed on the top of the model ground. Then the loading system as shown in **Fig. 1** was set-up, and air-pressure of 40 kPa was applied. The consolidation was under two-way drainage condition. During the consolidation process, the settlement at the top of the model ground was monitored.

2.3 *Soil-cement column and slab installation*

After the degree of primary consolidation reached about 90%, the pre-consolidation was stopped and the loading system was dismounted and the mini-PVD was withdrawn. At this point, the remaining thickness of soil sample was about 630 mm. In order to investigate the effect of slab, the area improvement ratio and depth improvement ratio were fixed at 20% and 70% respectively. At the middle of the model ground, a hole with a diameter of 200 mm and a depth of about 440 mm was made by an auger. For Case L2, L3 and L4, the soil surface was excavated with a pre-designed depth (100 mm – 250 mm). At the bottom of the hole, a small hole was left after withdrawal of the mini-PVD, which was carefully filled with the clay. Then, the soil excavated was mixed with cement (16.6% and 23.47 % by dry weight). For ease of mixing with the cement, the water content of the excavated soil was adjusted to about 68%. In Case L1 (without slab case), the mixture was put into hole to form a model column. In Case L2, L3 and L4 (with slab case), the mixture was split into 2 parts. The first part was put back into the hole to form a model column. The latter was spread on the surface with a pre-designed depth to form the model slab. To avoid the air-bubbles trapped in the column and the slab, the mixture was carefully compacted layer by layer by a steel-rod. Then, the model was left for 4 weeks to cure the column and the slab before the consolidation test.

2.4 *Consolidation test stage*

Three (3) layers of geotextile were installed on the top of the model ground, and the loading system was set-up again. Air-pressure of 120 kPa was applied. During the test, the settlement at the top of the model ground was monitored.

2.5 *Comparisons of test result*

The measured and the calculated settlement by the method of Pongsivasathit et al. (2013) are compared in **Table 2**. In **Table 2**, the relative error (RE) is defined as the percentage of the difference between the calculated and the measured settlement divided by the measured settlement (Pongsivasathit et al., 2013). Based on the results in **Table 2**, for Case L1 (without slab case), the method of Pongsivasathit et al. (2013) resulted in a RE of only about 3.72%. For Case L2, L3 and L4, this method over-predicted the settlement considerably. The comparisons of settlement-time curves on the effect of slab thickness are illustrated in **Fig. 2**. Obviously, the increase of H_s significantly reduced the settlement of the model ground. Comparing Case L2 with Case L3, it can be seen that increase of $(q_u)_{slab}$ reduced slightly the settlement of model ground as shown in **Fig. 3**.

Therefore, the interaction behavior between the soil-cement column with a surface cement stabilised slab and surrounding soft soil was further investigated by FEA focusing on the effect of H_s and $(q_u)_{slab}$.

Table 2 Cases studied and comparison of measurements with calculated values from Pongsivasathit et al. (2013).

Case	Cement by dry weight (%)		Thickness of slab H_s (mm)	α (%)	β (%)	Settlement (mm)		RE (%)
	Column	Slab				$(s_f)_{mea}$	$(s_f)_{cal}$	
L1	16.42	16.42	0	20	70	28.50	29.60	3.72
L2	16.42	16.42	100	20	70	26.97	28.83	6.90
L3	16.42	23.47	100	20	70	25.21	28.54	13.21
L4	16.42	23.47	250	20	70	22.92	25.64	11.87

Note: $(s_f)_{mea}$ and $(s_f)_{cal}$ are the calculated and measured final settlement respectively.

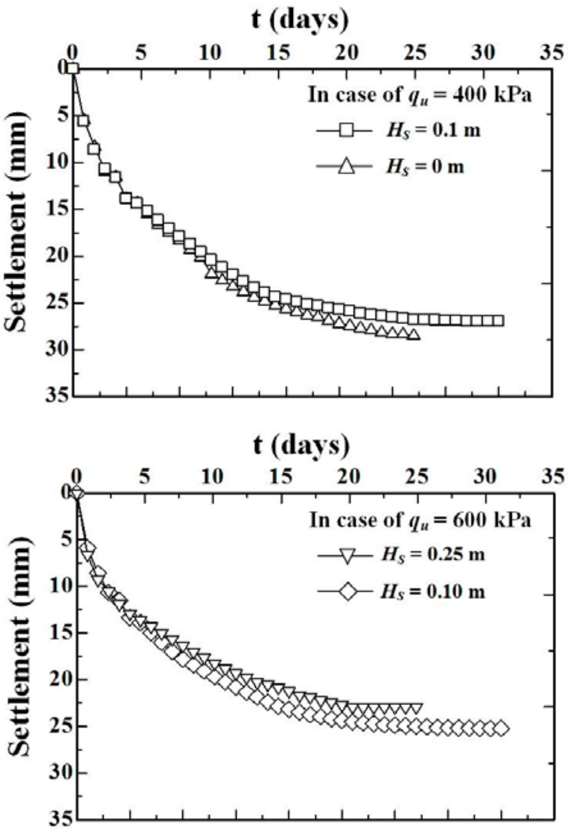


Fig. 2. The effect of slab thickness.

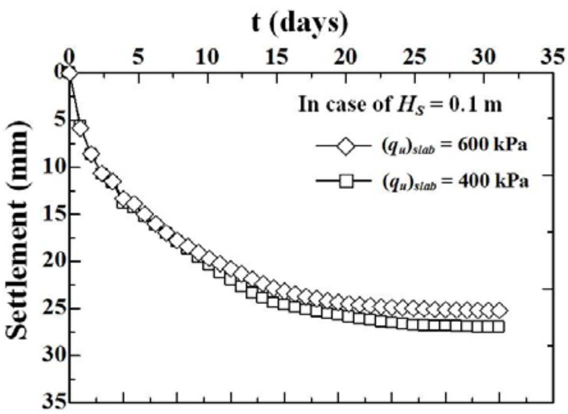


Fig. 3. The effect of $(q_u)_{slab}$.

3. Finite element analysis model and parameters

The axisymmetric unit cell model used in the FEA is illustrated in **Fig. 4** together with the boundary conditions adopted. The FEA was carried out using PLAXIS 2D version 8.0 (Brinkgreve, 2002). Fifteen-noded triangular elements were used to simulate the model ground model and the mesh is shown in **Fig. 5**.

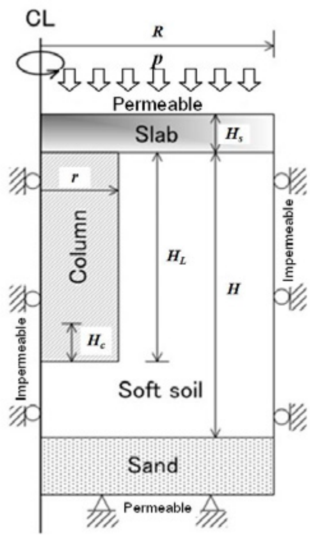


Fig. 4. Unit cell model for floating column improved soft soil.

The soft clay soil was represented by the Soft-Soil model (Brinkgreve, 2002). The column was modeled as a linear elastic material and the slab was simulated by bi-linear elastic model with Mohr-Coulomb failure criterion. The interface between soil-cement column and surrounding soil was simulated by a joint element with the R_{inter} value of 0.9. R_{inter} is the strength reduction factor for interfaces. Referring Pongsivasathit et al. (2013), the assumed thickness of soft layer (H) and slab (H_s) were 15 m and 1.0 m, respectively. The other assumed model parameters are given in **Table 3**. Regarding α and β, they were fixed at 20% and 70%, respectively. The diameter of column was maintained at 1.0 m. So, the diameter of unit cell was 2.236 m according to $\alpha = A_c/A_e$.

The unconfined compression strength (q_u) of the soil-cement column was assumed as 550 kPa, and the Young's modulus (E) was estimated as $100q_u$ and the Poisson's ratio of 0.2. It was assumed that the groundwater level was set at the bottom of the slab. Referring to the site investigation results of soft Bangkok clay in the Bangkok Plain, Thailand (Horpibulsuk et al., 2007), it was assumed that the soil layers from the ground surface to 4.0 m depth had an over-consolidation ratio (OCR) value varying from 3.2 to 1.1, and below it was a normally consolidated layer. The unit weight of soft soil deposit was assumed as 13.6 kN/m^3.

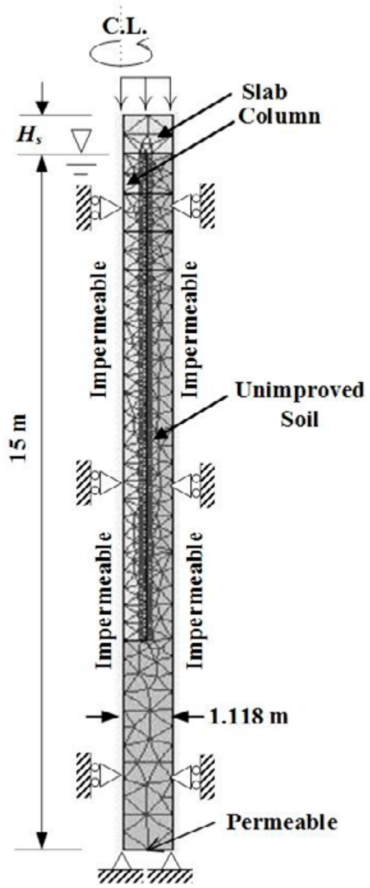


Fig. 5 Finite element model.

4. Finite element analysis results

4.1 Column-Slab System

The presentation of FEA results is focused on the relative settlement between the column and the surrounding soil. For ease of presentation, the parameters called the settlement ratio (SR) (Chai et al., 2009) and the length ratio (LR) (Pongsivasathit et al., 2013) were used with definition as follows

$$SR = \frac{\delta_s}{\delta_c} \quad [1]$$

$$LR = \frac{L_s}{(H_L + H_s)} \quad \text{(prototype)} \quad [2]$$

where L_s is the length from the end of the column to a point on the column at that elevation. The settlement of the soft soil (δ_s) at the middle between two adjacent columns equals a pre-defined portion (for example 0.95) of the settlement of the column (δ_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. Larger value of LR means there is a larger portion of the improved layer needs to be treated as an unimproved layer in the settlement calculation.

4.2 Effect of the undrained shear strength of slab

To consider the effect of strength and stiffness of slab on LR, the undrained shear strength of the slab ($(s_u)_{slab}$) was varied from 50 kPa to 750 kPa but the value of α, β and p were fixed as 20%, 70% and 100 kPa, respectively. In FEA, to vary $(s_u)_{slab}$ value, only c' and E ($E = 100q_u$) were changed and all other parameters remained the same. The relationship between $(s_u)_{slab}$ and LR was compared in **Fig. 6.** The $(s_u)_{slab}$ slightly effects on the LR value. That means that for the range investigated the strength of the slab has only minor effect on the relative movement of column with the surrounding soil.

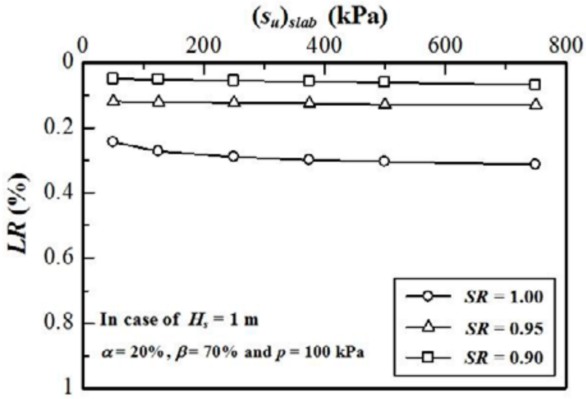


Fig. 6. Relationship between LR and $(s_u)_{slab}$.

4.3 Effect of the thickness of the slab

In FEA, H_s was varied from 1 m to 5 m. The other factors were fixed as: α = 20%, β = 70%, p = 100 kPa and $(s_u)_{slab}$ = 250 kPa. The adopted soil properties of the model ground are listed in **Table 3**. The groundwater level was set at 1 m below the ground surface. The relationships between H_s and LR are compared in **Fig. 7**.

Table 3 Assumed model parameters for FEA of prototype condition.

Depth (m)	Soil layer	ν	E (kPa)	κ	λ	c (kPa)	ϕ (°)	K_0^{NC}	OCR	e_0	k_h ($\times10^{-8}$ m/s)	k_v ($\times10^{-8}$ m/s)
0-4.0	Clay-1	0.3	-	0.065	0.65	1	30	0.5	4-1.1	3.3	3.0	2.0
4.0-16.0	Clay-1	0.3	-	0.065	0.65	1	30	0.5	1	2.9	3.0	2.0
Column		0.2	55,000	-	-	-	-	-	-	The same as the corresponding soil layer		
Slab		0.2	$100q_u$	-	-	50-750	0	-	-			

Note: κ is the slope of unloading–reloading line in e ln p′ plot; c is the cohesion; ϕ is the angle of friction; K_0^{NC} is the coefficient of earth pressure at-rest for normal consolidation state; k_v is the hydraulic conductivity in the vertical direction; k_h is the hydraulic conductivity in the horizontal direction.

It has been found that a thicker slab has a smaller LR value which means a smaller relative penetration of the column into the surrounding soil. The reasons for the less relative penetration of the column are (a) less stress concentration at the top of the column and (b) less surface settlement of the surrounding soil.

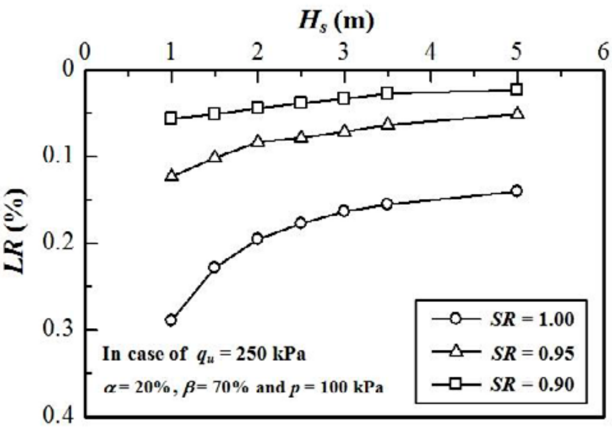


Fig. 7. Relationship between LR and H_s.

5. Modification of Pongsivasathit et al. (2013) Method

A new function as i(ω) was defined for considering the effect of slab thickness. Here, ω is the ratio of H_s and H in percent. To include the ω value into the equation for evaluating the value of H_c, the following approach was taken.

1) Find an H_c value (by trial and error) which can result in the same settlement as the measured value ($H_{c(mea)}$) from the model test or from the results of FEA ($H_{c(FEA)}$). In FEA, H_s was varied from 1.0 m to 5.0 m and the thickness of soft soil deposit (H) from 15 m to 22 m.

2) Referring Pongsivasathit et al. (2013), the H_c values are calculated by using the following equation;

$$H_c = H_L \times f(\alpha) \times g(\beta) \times h(\gamma) \quad [3]$$

$$f(\alpha) = \begin{cases} 0.75 - 0.025(\alpha) & (\alpha \le 20\%) \\ 0.45 - 0.010(\alpha) & (20\% < \alpha \le 45\%) \\ 0 & (\alpha > 45\%) \end{cases} \quad [4]$$

$$g(\beta) = \begin{cases} 1.62 - 0.016\beta & (20\% \le \beta \le 70\%) \\ 0.5 & (70\% \le \beta \le 90\%) \end{cases} \quad [5]$$

$$h(\gamma) = \frac{H_{c(FEA)}}{H_c} = 0.27\ln(\gamma) - 0.41 \quad [6]$$

$$\gamma = \frac{p.p_a^{1.5}}{s_u^{2.5}} \quad [7]$$

where p_a is the atmosphere pressure and s_u is the undrained shear strength of soft soil deposit at the end of column which can be calculated as follows:

$$s_u = \frac{M_{MC}}{2} p'_{MC} (OCR_e)^{\Lambda} \quad [8]$$

$$p'_{MC} = p'_e \left(\frac{M^2 + \eta_e^2}{M^2 + M_{MC}^2} \right)^{\Lambda} \quad [9]$$

$$OCR_e = \left(\frac{\frac{\sigma'_v OCR(1+2K_0^{nc})}{3} + c' \cot\phi'}{p'_e} \right) \quad [10]$$

$$\eta_e = \frac{q}{p'_e} \quad [11]$$

where p'_e is the equivalent initial mean stress; q is the deviator stress; η_e is the stress ratio; M is the slope of critical state line (CSL), (in the soft soil model (**Fig. 8**), which is mainly a function of coefficient of at-rest earth pressure at normally consolidated state, K_0^{nc}); and M_{MC} is the slope of Mohr-coulomb criteria in p′ – q plane; p'_{MC} is the equivalent mean stress on M_{MC} line; Λ =1– κ/ λ; λ, κ are the slopes of virgin compression and swelling lines in e – lnp′ plot (e is the voids ratio and p′ is the effective mean stress) respectively; OCR_e is the equivalent over consolidation ratio; c′ is the effective cohesion and ϕ′ is the effective friction angle of the soft subsoil; σ'_v is the initial vertical effective overburden pressure.

3) Referring Pongsivasathit et al.(2013), the thickness of the slab was fixed at 1 m. So, the ratio of $H_{c(FEA)}$ with H_c for H_s = 1 m would be calculated, given here as ($H_{c(FEA)}/H_c$). This value would be used to compare with a ratio of $H_{c(FEA)}$ with H_c for the different H_s.

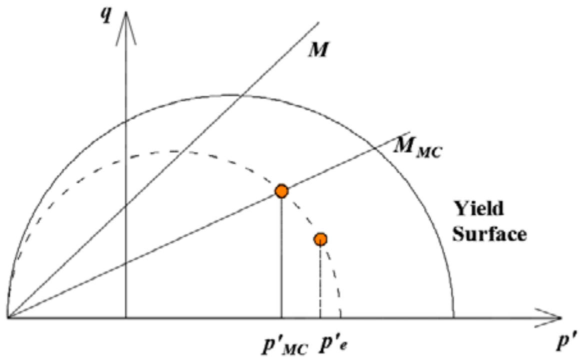


Fig. 8. Yield surface of the soft soil model in p′-q plane.

4) Calculate a ratio of $H_{c(FEA)}$ with H_c for the other H_s, given here as $(H_{c(FEA)}/H_c)'$.

The relationship between the ratio of $(H_{c(FEA)}/H_c)'/(H_{c(FEA)}/H_c)$ and ω are plotted in **Fig. 9**. The FEA values are scattered. A possible reason is that the proposed functions of f(α), g(β) and h(γ) are approximations of the real situation. Also, the ratios of $H_{c(mea)}$ with H_c are plotted in **Fig. 9**. So, an empirical equation has been proposed to consider the effect of ω on the calculated H_c value as follows:

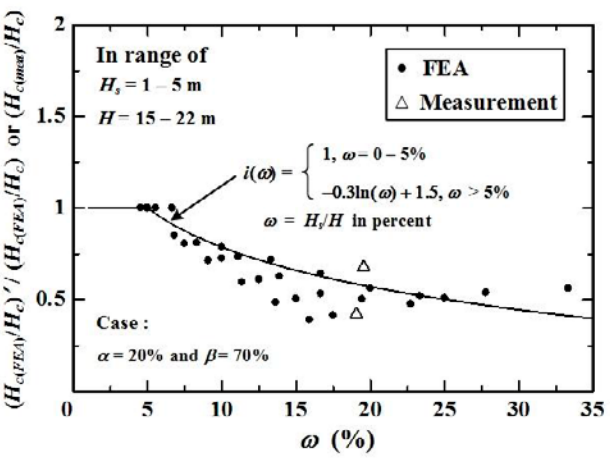


Fig. 9. Variation of $(H_{c(FEA)}/H_c)'/(H_{c(FEA)}/H_c)$.

$$i(\omega) = \begin{cases} 1 & (0 < \omega \le 5\%) \\ -0.31\ln(\omega) + 1.5 & (\omega > 5\%) \end{cases} \quad [12]$$

Then, the new equation for calculating H_c has been proposed as:

$$H_c = H_L \times f(\alpha) \times g(\beta) \times h(\gamma).i(\omega) \quad [13]$$

6. Method for calculating the settlement – time curve

6.1 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 **Fig. 10**.

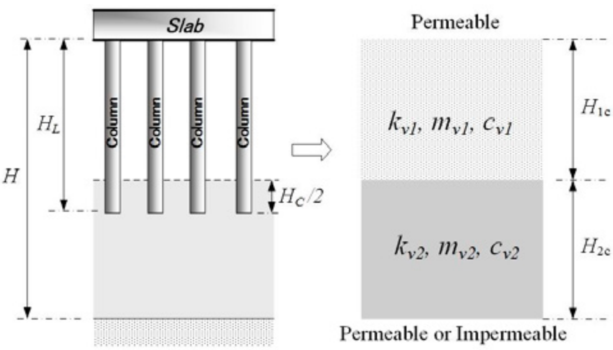


Fig. 10. Two-layer model for calculating the degree of consolidation.

The methods for determining k_{v1} and m_{v1} for the upper layer (Fig. 10), and the thicknesses (H_{1c} and H_{2c}) of both layers are as follows

$$m_{v1} = \frac{1}{\alpha D_c + (1-\alpha) D_s} \quad [14]$$

$$k_{v1} = \left(1 + \frac{2.5H_{1c}^2}{\mu d_e^2}\frac{k_h}{k_v}\right) \quad [15]$$

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

$$\mu = \ln\frac{n}{s} + \frac{k_h}{k_s}\ln(s) - \frac{3}{4} + \frac{8H_{1c}^2 k_h}{3d_c^2 k_c} \quad [16]$$

where n = d_e/d_c, s = d_s/d_c (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_s 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 16 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. For the thickness of the upper layer and the lower layer in calculating the degree of consolidation, Chai and Pongsivasathit (2010) revealed that the thickness of layer-1 can be between H_L and $(H_L - H_c)$. By comparing with the results of finite element analysis (FEA) using a unit cell model, and by trial and error, they found that $H_1 = H_L - H_c/2$ can yield a good result. Another point is that due to the large consolidation strain within layer-2 (unimproved), the thickness of layer-2 will be quite different before and after consolidation. To approximately consider this kind of large deformation phenomenon, they proposed to use average thickness of layer-2 for calculating the degree of consolidation as $H_2 = H_{20} - s_f/2$. Where H_{20} is the initial thickness of layer-2 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.

6.2 Settlement-time curve

Generally, the settlement consists of two parts: the compression of the column improved layer (s_1) with a thickness of $H_1 = H_L - H_c$ and the compression of the unimproved layer plus H_c layer (s_2) with a total thickness of H_2: The equations for calculating s_1 and s_2 values are as follows

$$s_1(t) = \sum_{i=1}^{n} \left(\frac{\Delta p_{1i} H_{1i} U(t)}{D_{ci}\alpha + (1-\alpha)D_{si}} \right) \quad [17]$$

$$s_2(t) = \sum_{i=1}^{n} \left[H_{2i} \frac{\lambda_i}{1+e_{0i}} \ln\left(1 + \frac{\Delta p_{2i}}{\sigma'_{vi}} U(t) \right) \right] \quad [18]$$

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; σ'_{vi} is the initial vertical effective stress in the sublayer H_{2i}; e_{0i} is the initial void ratio; λ_i is the slope of virgin compression line in e-lnp′ plot in the corresponding subsoil layer; Δp_{1i} and Δp_{2i} are the total vertical stress increments in sub-layer H_{1i} and H_{2i}, respectively; D_{ci} and D_{si} are the constrained moduli of the column and the surrounding soil of sublayer H_{1i} respectively and they can be calculated as follows

$$D_{ci} = \frac{E_i(1-v_i)}{(1+v_i)(1-2v_i)} \quad [19]$$

$$D_{si} = \frac{(1+e_i)\sigma'_{avi}}{\lambda_i} \quad [20]$$

where E_i is Young's modulus, ν_i is Poisson's ratio and σ'_{avi} is the average effective vertical stress in the corresponding subsoil layer. In Equations 18 and 20, κ_i is the slope of the unloading-reloading line in the e-lnp′ plot, is used instead of λ_i, in case the subsoil is in an overconsolidated state – that is, $[\sigma'_{vi} + \Delta p_{1i}U_{1i}(t)]$ or $[\sigma'_{vi} + \Delta p_{2i}U_{2i}(t)]$ is less than p_c (the consolidation yield stress).

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

$$s(t) = s_1(t) + s_2(t) \quad [21]$$

7. Application of the proposed method to laboratory model tests

The proposed method and the method of Pongsivasathit et al. (2013) are applied to laboratory model tests. The parameters used for calculation are listed in **Tables 2** and **4**. The calculated results by the both methods are compared with the measured ones in **Figs.11(1), 11(2)** and **11(3).**

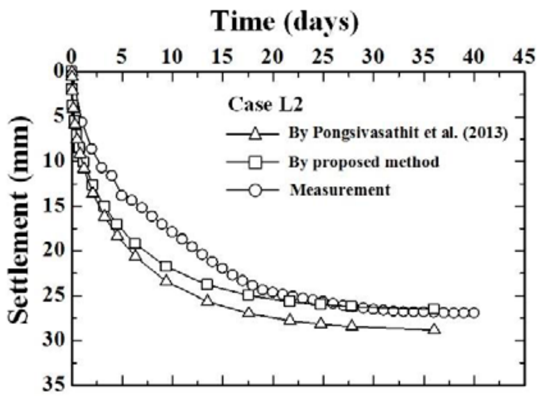


Fig. 11(1). Comparison of results on the settlement–time curves for Case L2.

It shows that the calculated results by the proposed method fit the measured curves better than that by the method of Pongsivasathit et al. (2013). For with slab cases, the values of H_c will be overestimated by Pongsivasathit et al.'s (2013) method. However, the rate of consolidation for the results of proposed method is faster than that for the measurement.

Table 4 The adopted parameters for calculating degree of consolidation.

Case	ω (%)	Method	c_{v1} ($\times10^{-3}$ m^2/day)	c_{v2} ($\times10^{-3}$ m^2/day)	k_{v1} ($\times10^{-5}$ m/day)	k_{v2} ($\times10^{-5}$ m/day)	H_{1c} (m)	H_{2c} (m)
L1	-	A	10.404	1.087	1.111	0.432	0.4106	0.2005
		B	10.404	1.087	1.111	0.432	0.4106	0.2005
L2	19.53	A	10.187	1.087	1.086	0.432	0.3100	0.1886
		B	10.151	1.087	1.082	0.432	0.2989	0.1985
L3	19.05	A	15.104	1.087	1.086	0.432	0.3097	0.2021
		B	15.049	1.087	1.081	0.432	0.2983	0.2123
L4	68.12	A	9.339	1.087	0.955	0.432	0.1774	0.1784
		B	13.554	1.087	0.974	0.432	0.1625	0.1917

Note: A is the proposed method and B is the method of Pongsivasathit et al. (2013)

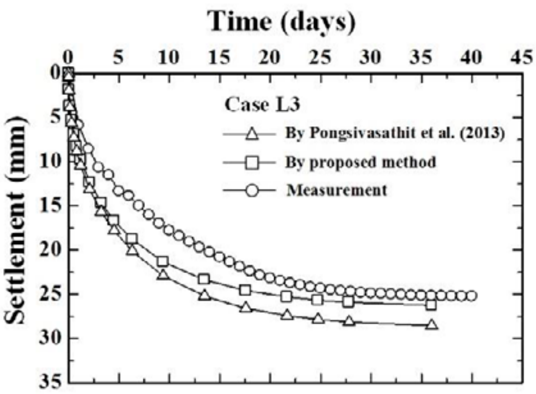


Fig. 11(2). Comparison of results on the settlement–time curves for Case L3.

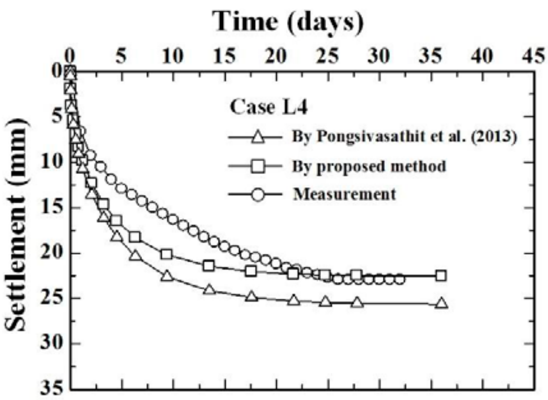


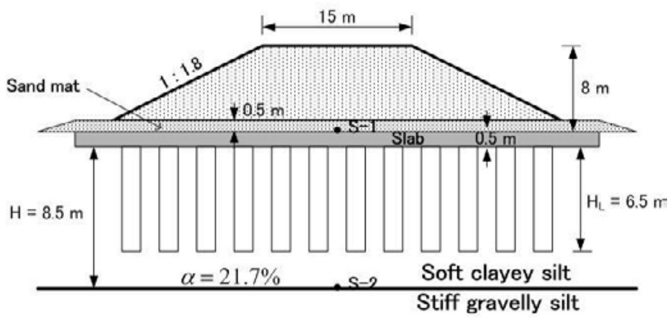
Fig. 11(3). Comparison of results on the settlement–time curves for Case L4.

The possible reason is that the c_v values used in calculations are the average one for whole layer of soft clay. As results, the settlement rate at the early stage for the measurement are slower than that for the results of proposed method.

8. Application of proposed method to case histories

8.1 General Description

Two case histories in Fukuoka, Japan were described by Chai et al. (2009). The cross-sections of the two cases are shown in **Figs.12 - 13** respectively, and the construction time as well as some of design parameters are given in **Table 5**. The field measurements indicate that there was settlement below the soft layers for these two case histories. Since the proposed method only considers the compression of soft layer, the settlement differences (Δs) between points S-1 and S-2 in **Figs. 12 - 13** were used to compare with the calculated values. The Young's moduli of the column and the slab were assumed as 100 times the corresponding design values of q_u and Poisson's ratio of 0.2.



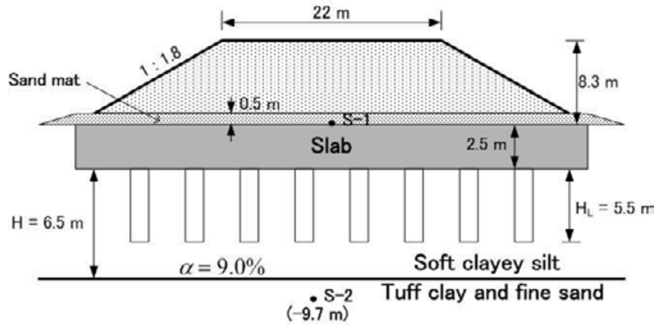


Fig. 13. Cross-section of Case-2.

8.1 Comparison of results

Referring Pongsivasathit et al. (2013), in macro level, the floating column improved soft clayey subsoil can be considered as a two-layer system, and the degree of consolidation can be evaluated by the corresponding theoretical solutions, proposed by Zhu and Yin (1999). The Zhu and Yin's solution can consider linear variation of the total stress increment in two-layer system as shown in **Fig. 14**. Under embankment load, σ_0, σ_1 and σ_2 can be approximately calculated by Osterberg's (1957) method. Based on the test data 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 are the values corresponding to the average consolidation pressure. The used parameters for settlement calculation are listed in **Table 7**.

Table 5 Some of design and geometry parameters for the 2 case histories

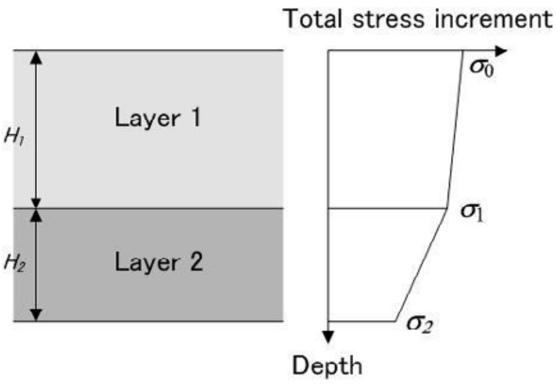
Case	1	2
The design compressive strength of column, q_u (kPa)	700	700
The amount of the cement mixed with soil for column (kg/m^3)	140	100
Column length (m)	6.5	5.5
α (%)	21.7	9
β (%)	76	85
Thickness of slab (m)	0.5	2.5
ω (%)	5.9	38.5
The design compressive strength of slab, q_u (kPa)	300	300
The amount of the cement mixed with soil for slab (kg/m^3)	80	80
Time of construction (day)	96	90
Final embankment thickness (m)	8	8.3

Table 6 Parameters for calculating the degree of consolidation

Case		1	2
H (m)		8.5	6.5
H_{1c}(m)		6.24	5.29
H_{2c}(m)		2.11	1.32
σ_0 (kN/m^2)		152.00	157.70
σ_1 (kN/m^2)		146.96	153.23
σ_2 (kN/m^2)		140.24	150.97
α (%)		21.7	9.0
β (%)		76.5	84.6
s_u at the end of column (kN/m^2)		22.50	20.00
Calculated final settlement (m)		0.287	0.221
Layer H_1	c_{v1} (m^2/day)	0.643	0.177
	k_{v1} ($\times 10^{-4}$ m/day)	3.387	2.347
Layer H_2	c_{v2} (m^2/day)	0.035	0.031
	k_{v2} ($\times 10^{-4}$ m/day)	2.668	2.022

Table 7 Parameter for calculating the consolidation settlement.

Case	Depth (m)	e_0	λ	γ_t (kN/m^3)	OCR
1	0 - 3	1.942	0.195	15.46	1.9
	3 - 5	1.529	0.178	16.39	1.9
	5 - 9	1.818	0.226	15.75	1.4
2	0 - 2.5	1.965	0.208	15.27	2.5
	2.5 - 4	1.946	0.191	15.44	2.5
	4 - 5	1.632	0.191	16.10	1.5
	5 - 7	1.632	0.191	16.10	1.5
	7 - 9	1.832	0.287	15.75	1.5



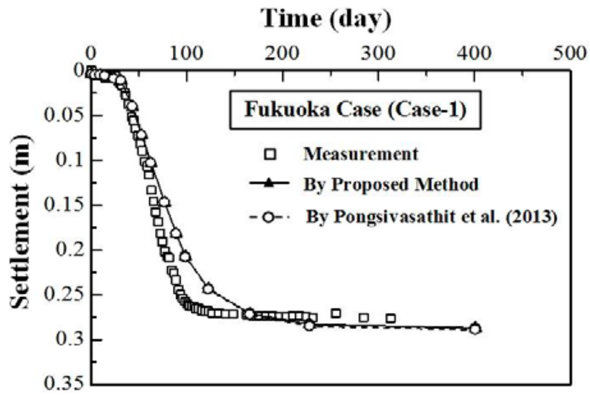


Fig. 15(1). Comparison of calculated results with measurement results for Case-1.

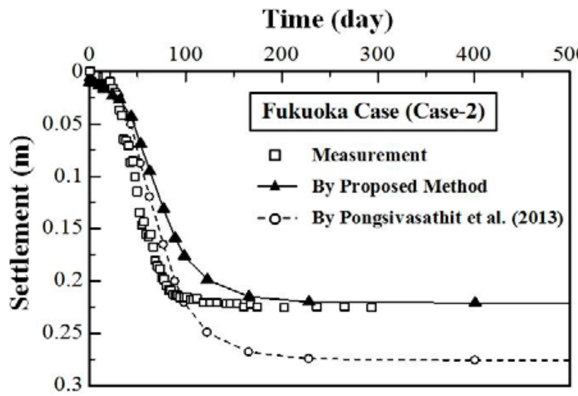


Fig. 15(2). Comparison of calculated results with measurement results for Case-2.

The settlement-time curves of 2 case histories are calculated by using the proposed method and the method of Pongsivasathit et al. (2013). The comparisons of the settlement curves are given in **Figs. 15(1)** and **15(2)**. For Case-1, the value of i(ω) is about 0.951. The overall effect on the H_c value is about the same as the previous method. For Case-2, the value of i(ω) is about 0.373. The proposed method yielded a better simulation. 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 two cases initially the subsoil was in an overconsolidated state.

9. Conclusion

The behavior between the floating column and the surrounding soft soil was investigated on the effect of thickness and the undrained shear strength of the cement-stabilised slab on the ground surface by conducting the laboratory model testing and finite element analysis (FEA). A new parameter has been defined as ω is the ratio of a slab thickness with the thickness of soft clayey deposit. Based on the results, the method to calculate the settlement-time curve of floating soil-cement column improved soft clayey deposit by Pongsivasathit et al. (2013) was modified by adding the function of ω in the equation for calculating the value of H_c. H_c is the thickness of a part of the column improved layer near the end of the column, which is treated as an unimproved layer in settlement calculation. The proposed method was applied to calculate the settlement – time curves of the laboratory model tests and 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 a cement-stabilised slab on the ground surface.

Acknowledgements

The authors wish to acknowledge the partial support by the RMUTT Annual Government Statement of Expenditure in 2014. Opinions expressed in this paper are those of the authors and do not necessarily reflect those of the sponsor.

References

Bergado, D.T., Chai, J.C., Alfaro, M.C. and Balasubramaniam, A.S., 1994. Improvement techniques of soft ground in subsiding and lowland environments. Balkema, Rotterdam: pp. 222.

Brinkgreve, R.B.J., 2002. Plaxis 2D. Scientific Manual V.8. Delft, the Netherlands.

Broms, B.B. and Boman, P.O., 1979. Lime column: a new foundation method. Journal of Geotechnical Engineering Division, ASCE, **105** (GT4): 539–556.

Chai, J.C. and Pongsivasathit, S., 2010. A method for calculating consolidation settlements of floating column improved clayey subsoil. Frontiers of Architecture and Civil Engineering in China, **4** (2): 241–251.

Chai, J.C., Miura, N., Kirekawa, T. and Hino, T., 2009. Settlement prediction for soft ground improved by columns. Proceedings of the Institution of Civil Engineers – Ground Improvement **163** (2): 109–119.

FNHO (Fukuoka National Highway Office), 2003. Report on the field monitoring of test embankments at Shaowa-Biraki, Road No. 208, Fukuoka, Part 1. Kyushu Regional Development Bureau, Ministry of Land, Infrastructure, Transport and Tourism, Kyushu, Japan (in Japanese).

Hansbo, S., 1981. Consolidation of fine-grained soils by prefabricated drains. Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, **3**: 677–682.

Horpibulsuk, S., Shibuya, S., Fuenkajorn, K. and Katkan, W., 2007. Assessment of engineering properties of Bangkok clay. Canadian Geotechnical Journal, **44**: 173–187.

Japanese Institute of Construction Engineering (JICE), 1999. Flexible foundation, Foundation structure part. Design Code for Flexible Box Culvert-II, Tokyo: San-kai-dou Press,4: 233 –248 (in Japanese)

Osterberg, J.O., 1957. Influence values for vertical stresses in semi-infinite mass due to embankment loading. Proceedings of the 4th International Conference in Soil Mechanics and Foundation Engineering, **1**: pp 393.

Pongsivasathit, S., Chai, J.C. and Ding, W.Q., 2013. Consolidation settlement of floating-column-improved soft clayey deposit. Ground Improvement, Proceeding of Institute of Civil Engineers, UK, **166** (1): pp. 44-58.

Shen, S.L., Chai, J.C. and Miura, N., 2001. Stress distribution in composite ground of column–slab system under road pavement. First Asian-Pacific Congress on Computational Mechanics. Elsevier Science: pp. 485–490.

Zhu, G. and Yin, J.H., 1999. Consolidation of double soil layers under depth-dependent ramp load. Geotechnique, **49** (3): 415– 421.

Symbols and abbreviations

A_c	Area of the column
A_e	Area of the unit cell which represents a column and its improvement area
c	Cohesion of soil
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_{ci}	Constrained moduli of the column of the sub-layer H_{1i}
D_s	Constrained moduli of the surrounding soil
D_{si}	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_L	Length of the column
H_S	Thickness of slab
Hc	Thickness of the part of the column improved layer to be treated as an unimproved layer in settlement calculations.
H_1	Thickness of the upper layer for the s(t) calculation
H_{1c}	Thickness of the upper layer for the U(t) calculation
H_{1i}	Thickness of subsoil layers in H_1
H_2	Thickness of the lower layer for the s(t) calculation
H_{2i}	Thickness of subsoil layers in H_2
H_{2c}	Thickness of the lower layer for the U(t) calculation
H_{20}	Initial thickness of lower layer
k_c	Hydraulic conductivity of the column
k_h	Hydraulic conductivity in the horizontal direction
k_s	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}
K_0^{NC}	Coefficient of at-rest earth pressure at normally consolidated state
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 including the thickness of a slab
M	Slope of the critical state line in (q, p′) plot
M_{MC}	Slope of Mohr-coulomb criteria in p′ – q plane
m_{v1}	Constrained compressibility for layer H_{1c}
m_{v2}	Constrained compressibility for layer H_{2c}
OCR	Overconsolidation ratio
OCR_e	Equivalent over consolidation ratio

p	Loading intensity or consolidation pressure
p_a	Atmospheric pressure
p_c	Consolidation yield stress
p_0	Pre-consolidation pressure
p'_e	Equivalent initial mean stress
p'_{MC}	Equivalent mean stress on M_{MC} line
p'	Mean effective stress
q	Deviatoric stress
q_u	Unconfined compression strength of the soil-cement column
R_{inter}	Strength reduction factor for interfaces
RE	Relative error
SR	Settlement ratio sf final consolidation settlement of the system
$(s_f)_{cal}$	Calculated final settlement
$(s_f)_{mea}$	Measured final settlement
$s(t)$	Total consolidation settlement at any time
$s_1(t)$	Consolidation settlement at any time for layer H_1
$s_2(t)$	Consolidation settlement at any time for layer H_2
s_u	Undrained shear strength of the soft soil
$U(t)$	Degree of consolidation at any time
$U_{1i}(t)$	Average degree of consolidation of the subsoil layers in H_1 at time t
$U_{2i}(t)$	Average degree of consolidation of the subsoil layers in H_2 at time t
w_L	Liquid limit
w_P	Plastic limit
α	Area improvement ratio
β	Depth improvement ratio
γ_t	Total unit weight
Δp_{1i}	Total vertical stress increments in sub-layer H_{1i}
Δp_{2i}	Total vertical stress increments in sub-layer H_{2i}
δ_c	Settlement of the column at a point considered
δ_s	Settlement of soil at periphery of unit cell at same elevation selected for measuring δ_c
η_e	Stress ratio
κ	Slope of unloading–reloading line in e ln p′ plot
κ_i	Slope of unloading–reloading line in e ln p′ plot in sub-layer H_{2i}
λ	Slope of virgin compression line in e ln p′ plot
λ_i	Slope of virgin compression line in e ln p′ plot in the corresponding subsoil layer
ν	Poisson's ratio
ν_i	Poisson's ratio in the corresponding subsoil layer
σ'_v	Initial vertical effective overburden pressure
σ'_{avi}	Average effective vertical stress in the corresponding subsoil layer
σ'_{vi}	Initial vertical effective stress in the sub-layer H_{2i}
σ_0, σ_1, σ_2	Embankment loads calculated by the Osterberg (1957) method
ω	Ratio of H_s with H
ϕ	Internal friction angle
ϕ'	Effective internal friction angle