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Simplified method for predicating consolidation settlement of soft ground improved by floating soil−cement column

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Abstract: A simplified method is presented for predicting consolidation settlement of soft ground improved by floating soil−cement column on the basis of double soil-layer consolidation theory. Combining the axisymmetric consolidation model and equal strain assumption, the governing equation was derived for the consolidation of clayey subsoil reinforced by soil−cement column. By modifying the boundary condition of the interface between the improved layer and underlying layer on seepage and pore-water pressure, the analytical solution of consolidation of soft ground improved by floating soil−cement column was developed under depth-dependent ramp load. The results of the parameter analysis of consolidation behavior show that the consolidation rate is closely related with the depth replacement ratio by the column and the permeability of upper layer. The influence of column−soil constrained modulus ratio and radius ratio of the influence zone to the column on consolidation is also affected by depth replacement ratio. The column−soil total stress ratio increases with time and approaches the final value accompanied with the dissipation of excess pore water pressure.

Key words: axisymmetric consolidation model; floating soil−cement column; consolidation; depth-dependent ramp load; analytical solution

1 Introduction

Soil−cement column, normally generated by deep mixing or jet grouting methods, is widely used for soft ground improvement [1−4]. Composite methods are commonly used to calculate the settlement of soft ground improved by column-type inclusions [5−8]. Also there have been some attempts to consider the penetration of columns into the underlying soft soil [9−11]. However, studies on the consolidation of soft ground improved by soil−cement column are still limited.

The consolidation theories for soft ground improved by columns are developed on the basis of those for sand-drained well foundation. Significant studies have been made in developing the consolidation theory for soft ground improved by granular column, such as stone column and sand column [12−14]. Nevertheless, these theories are not very suitable for soil−cement column improved ground because the permeability of cement admixed clay is much lower compared with sand drain or stone column. HOU and GONG [15] revealed that the permeability of soil−cement is almost 3 orders lower than the corresponding subsoil.

Double soil-layer consolidation theory is often cited for consolidation of soft ground improved by soil−cement column [16−17]. CHAI [18] calculated the consolidation of floating soil−cement column improved soft soil by the double soil-layer consolidation theory and also proposed the methods for evaluating the equivalent hydraulic conductivity and the coefficient of volume compressibility of the part of the column improved layer. However, the calculation of equivalent hydraulic conductivity and coefficient of volume compressibility mainly depended on some empirical methods and seemed to lack enough evidences. On the other hand, it is more rational to assume that the stress increment is a function of both time and depth. But in the existing methods, it is difficult to consider the variation of time and depth at the same time.

BERGADO et al [8] derived a new governing equation considering the interdependent consolidation of the soil−cement column and its surrounding clay in a typical unit cell. LU et al [19] proposed the governing

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equation for consolidation of composite ground with impervious pile based on the axisymmetric consolidation model. But these studies do not take into account the consolidation of underlying layer and may be not suitable for floating column case. Therefore, the boundary condition of the interface between the improved layer and underlying layer on seepage and pore water pressure needs to be modified due to the fact that the soil−cement column has much lower permeability.

In this work, a simplified method is proposed for predicting the consolidation settlement of soft ground improved by floating soil−cement column based on double soil-layer consolidation theory. The work begins with the governing equation derivation for the soil−cement column improved layer. Then, the boundary condition of the interface between the improved layer and underlying layer is modified. The corresponding analytical solution is derived under depth-dependent ramp load. Finally, the consolidation behavior is investigated.

2 Basic equation and solution

2.1 Basic equation

If soft ground was reinforced by column in a sufficiently large area, the axisymmetric consolidation model can simulate the real performance of the ground very well. The soft ground reinforced by floating soil−cement column was idealized as shown in Fig. 1. In order to obtain a simplified analytical solution, the following assumptions were made:

1) The equal vertical strain assumption was adopted, that is, the column and the surrounding soil only deform vertically and have an equal strain at any depth.

2) No flow and consolidation was considered within the soil−cement column. The column−soil interface was assumed to be an impervious boundary.

3) The stress increment in soft ground reinforced by soil−cement column was assumed to be a function depending on the vertical depth z and the elapsed time t, i.e. $\sigma=\sigma(z, t)$.

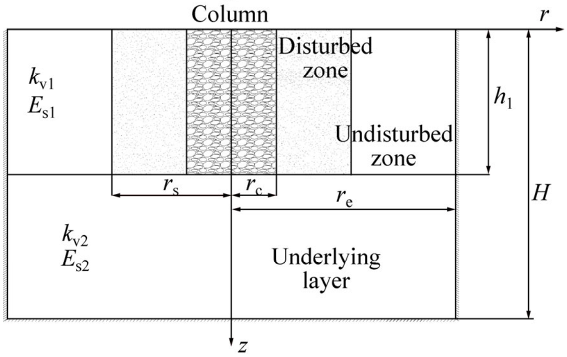

Fig. 1 Schematic diagram of consolidation of soft ground with floating soil−cement column

4) The horizontal permeability coefficient $k_{v1}(r)$ and constrained modulus $E_{s1}(r)$ of the soil in disturbed zone varied with respect to the radial distance away from the column.

5) Other assumptions were the same as those used in Terzaghi's one-dimensional consolidation theory.

First, we focused on the governing equation of consolidation for the soil−cement column improved layer (for $0\le z\le h_1$).

At any time, both the soil−cement column and the surrounding soil share the total stress, i.e.

$$\pi\left(r_e^2-r_c^2\right)\bar{\sigma}_{s1}+\pi r_c^2\bar{\sigma}_c=\pi r_e^2\sigma \tag{1}$$

where $\bar{\sigma}_c$ and $\bar{\sigma}_{s1}$ are the average total stresses within the column and surrounding soil (for $0\le z\le h_1$), respectively.

The equal strain assumption yields

$$\frac{\bar{\sigma}_{s1}-\bar{u}_{s1}}{\overline{E_{s1}(r)}}=\frac{\bar{\sigma}_c}{E_c}=\varepsilon_z=\varepsilon_v \tag{2}$$

where $\bar{u}_{s1}=\dfrac{1}{\pi(r_e^2-r_c^2)}\displaystyle\int_{r_c}^{r_e}2\pi r u_{s1}(r)\mathrm{d}r$ is the average excess pore water pressure within the surrounding soil (for $0\le z\le h_1$); $\overline{E_{s1}(r)}$ is the average constrained modulus of surrounding soil; E_c is the constrained modulus of soil−cement column; ε_z, ε_v is the vertical strain and volumetric strain at any depth of the surrounding soil as well as the column.

From Eqs. (1) and (2), ε_v can be derived as

$$\varepsilon_v=\varepsilon_z=\frac{n^2\sigma-\left(n^2-1\right)\bar{u}_{s1}}{\overline{E_{s1}(r)}\left(n^2-1+Y\right)} \tag{3}$$

where $n=r_e/r_c$ is the radius ratio and $Y=E_c\big/\overline{E_{s1}(r)}$ is the constrained modulus ratio of the soil−cement column to the surrounding soil.

The rate of the vertical strain can be obtained as

$$\frac{\partial\varepsilon_v}{\partial t}=-\frac{\left(n^2-1\right)\dfrac{\partial\bar{u}_{s1}}{\partial t}-n^2\dfrac{\partial\sigma}{\partial t}}{\overline{E_{s1}(r)}\left(n^2-1+Y\right)} \tag{4}$$

As described in the introduction, the permeability of soil−cement column is much lower than the subsoil. So, we consider no flow and consolidation within the soil−cement column in the assumption 2). That means that no radial flow will occur in the soft ground improved by soil−cement column. The governing equations can be given by the following principle of mass conservation:

$$\frac{\partial\varepsilon_v}{\partial t}+\frac{k_{v1}}{\gamma_w}\frac{\partial^2\bar{u}_{s1}}{\partial z^2}=0 \tag{5}$$

where k_{v1} is the vertical permeability of the soil (for

$0 \le z \le h_1$); γ_w is the unit weight of water.

Substituting Eq. (4) into Eq. (5) yields:

$$\frac{\partial \bar{u}_{s1}}{\partial t} - A\frac{\partial^2 \bar{u}_{s1}}{\partial z^2} = \frac{n^2}{n^2-1}\frac{\partial \sigma}{\partial t} \tag{6}$$

$$A = \frac{k_{v1}\overline{E_{s1}(r)}\left(n^2-1+Y\right)}{\gamma_w\left(n^2-1\right)} = \frac{\left(n^2-1+Y\right)\bar{c}_{v1}}{n^2-1}\frac{\delta y}{\delta x} \tag{7}$$

where A is the equivalent consolidation coefficient of the soft ground improved by soil−cement column; $\bar{c}_{v1} = k_{v1}\overline{E_{s1}(r)}/\gamma_w$ is the consolidation coefficient of soft soil.

Equation (6) is the governing equation of consolidation for the soil−cement column improved layer. The equivalent consolidation coefficient A is mainly related to the vertical permeability of the soil and the constrained modulus of soil−cement column and soil.

Then, we turn to the consolidation governing equation of the underlying layer (for $h_1 \le z \le H$). As revealed by previous research [20], load proportion of pile tip is relative small for soil−cement column. So, we can ignore the penetration of columns into the underlying soft soil for simplification here. Then, the Terzaghi's one-dimensional consolidation theory can be adopted for the underlying layer consolidation as

$$\frac{\partial \bar{u}_{s2}}{\partial t} - c_{v2}\frac{\partial^2 \bar{u}_{s2}}{\partial z^2} = \frac{\partial \sigma}{\partial t} \tag{8}$$

where $\bar{u}_{s2} = \frac{1}{\pi r_e^2}\int_0^{r_e} 2\pi r u_{s2}(r)\mathrm{d}r$ is the average excess pore water pressure within the underlying layer; $c_{v2} = k_{v2}E_{s2}/\gamma_w$ is the consolidation coefficient of the underlying layer; k_{v2} and E_{s2} is the vertical permeability and constrained modulus of the soil (for $h_1 \le z \le H$).

So, we can propose the governing equation of consolidation for the soft ground improved by floating soil−cement column.

$$\begin{cases} \dfrac{\partial \bar{u}_{s1}}{\partial t} = A\dfrac{\partial^2 \bar{u}_{s1}}{\partial z^2} + \dfrac{n^2}{n^2-1}\dfrac{\partial \sigma}{\partial t}, \quad 0 \le z \le h_1 \\ \dfrac{\partial \bar{u}_{s2}}{\partial t} = c_{v2}\dfrac{\partial^2 \bar{u}_{s2}}{\partial z^2} + \dfrac{\partial \sigma}{\partial t}, \quad h_1 \le z \le H \end{cases} \tag{9}$$

The vertical total stress increase caused by the external load is assumed to vary linearly with depth and time and remained unchanged after time t_0 (shown in Fig. 2).

$$\sigma(z,t) = \begin{cases} \left(\sigma_0 + \dfrac{\sigma_1-\sigma_0}{h_1}z\right)\min(1,\dfrac{t}{t_c}), 0 \le z \le h_1 \\ \left[\sigma_1 + \dfrac{\sigma_2-\sigma_1}{H-h_1}(z-h_1)\right]\min(1,\dfrac{t}{t_c}), h_1 \le z \le H \end{cases} \tag{10}$$

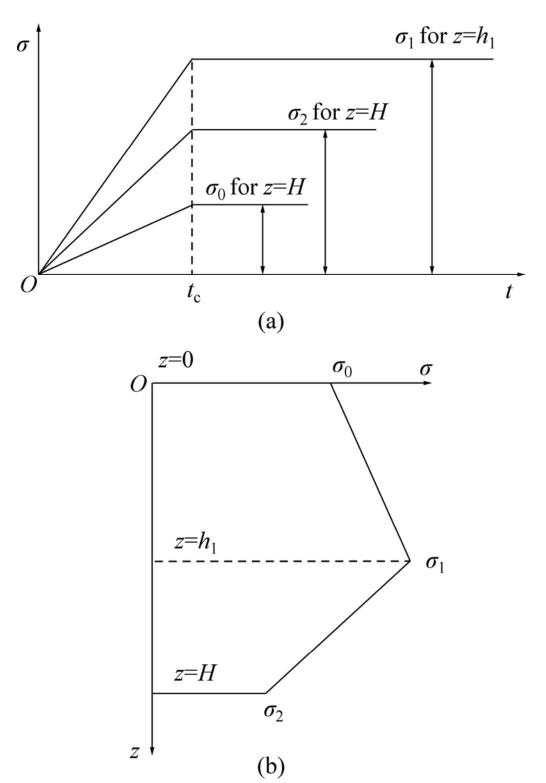

Fig. 2 Variation of vertical total stress increment: (a) Variation with time; (b) Variation with depth

where σ_0, σ_1 and σ_2 are the vertical total stress increment at $t=t_c$ and $z=0$, $z=h_1$ and $z=H$, respectively.

As shown in Fig. 1, the top surface of soft ground improved by soil−cement column is assumed to be pervious and the bottom surface is impervious (i.e. PTIB). Thus, the vertical boundary condition can be written as

$$\begin{cases} z=0: \ \bar{u}_{s1}(z,t)=0 \\ z=H: \ \dfrac{\partial \bar{u}_{s2}(z,t)}{\partial z}=0 \end{cases} \tag{11}$$

As aforementioned, soil−cement column usually has much lower permeability. There is a need to modify the boundary condition of the interface between the improved layer and underlying layer on seepage and pore-water pressure. The seepage boundary condition is stated.

$$(1-m)k_{v1}\frac{\partial \bar{u}_{s1}}{\partial z}\Big|_{z=h_1} = k_{v2}\frac{\partial \bar{u}_{s2}}{\partial z}\Big|_{z=h_1} \tag{12}$$

where $m=1/n^2$ is the replacement ratio of soil−cement column.

The soil−cement column is assumed to make no pore-water pressure. The pore-water pressure boundary condition is derived by the total stress and effective stress equilibrium conditions.

$$\pi\left(r_e^2-r_c^2\right)\bar{\sigma}_{s1} + \pi r_c^2\bar{\sigma}_c = \pi r_e^2\bar{\sigma}_{s2} \tag{13}$$

$$\pi\left(r_e^2-r_c^2\right)\left(\overline{\sigma}_{s1}-\overline{u}_{s1}\right)+\pi r_c^2\overline{\sigma}_c=\pi r_e^2\left(\overline{\sigma}_{s2}-\overline{u}_{s2}\right) \quad (14)$$

where σ_{s1} and σ_{s2} are the average total stresses within the soil for improved layer and underlying layer, respectively.

From Eqs. (13) and (14), it can be derived as

$$(n^2-1)\overline{u}_{s1}=n^2\overline{u}_{s2} \quad (15)$$

At the initial moment, no vertical deformation occurs in either the soil−cement column or the surrounding soil. Therefore, the following initial condition can be derived from Eq. (3):

$$t=0:\ \overline{\sigma}_{s1}=\overline{u}_{s1},\ \overline{\sigma}_{s2}=\overline{u}_{s2},\ \overline{\sigma}_c=0 \quad (16)$$

2.2 Analytical solution

Equation (9) is the governing equation of consolidation for the soft ground improved by floating soil−cement column. It can be seen that this equation is similar to the consolidation solution of double soil layers that provided by ZHU and YIN [21] in format. To solve it, the following parameter transformations are made:

$$\hat{u}_{s1}=(n^2-1)\overline{u}_{s1},\ \hat{u}_{s2}=n^2\overline{u}_{s2},\ \hat{\sigma}=n^2\sigma \quad (17)$$

Substituting Eq. (17) into Eqs. (9), (11), (12), (15), (16) yields:

$$\begin{cases}\dfrac{\partial\hat{u}_{s1}}{\partial t}=A\dfrac{\partial^2\hat{u}_{s1}}{\partial z^2}+\dfrac{\partial\hat{\sigma}}{\partial t},\ 0\le z\le h_1\\ \dfrac{\partial\hat{u}_{s2}}{\partial t}=c_{v2}\dfrac{\partial^2\hat{u}_{s2}}{\partial z^2}+\dfrac{\partial\hat{\sigma}}{\partial t},\ 0\le z\le h_1\end{cases} \quad (18)$$

$$\begin{cases}z=0:\ \hat{u}_{s1}(z,t)=0\\ z=H:\ \dfrac{\partial\hat{u}_{s2}(z,t)}{\partial z}=0\end{cases} \quad (19)$$

$$\hat{\sigma}(z,t)=\begin{cases}n^2\left(\sigma_0+\dfrac{\sigma_1-\sigma_0}{h_1}z\right)\min(1,\dfrac{t}{t_c}),0\le z\le h_1\\ n^2\left[\sigma_1+\dfrac{\sigma_2-\sigma_1}{H-h_1}(z-h_1)\right]\min(1,\dfrac{t}{t_c}),h_1\le z\le H\end{cases} \quad (20)$$

$$k_{v1}\left.\frac{\partial\hat{u}_{s1}}{\partial z}\right|_{z=h_1}=k_{v2}\left.\frac{\partial\hat{u}_{s2}}{\partial z}\right|_{z=h_1} \quad (21)$$

$$u_{s1}=u_{s2} \quad (22)$$

In addition, the following dimensionless parameters are defined as follows

$$p=\frac{\sqrt{(k_{v2}m_{v2})}-\sqrt{(k_{v1}\hat{m}_{v1})}}{\sqrt{(k_{v2}m_{v2})}+\sqrt{(k_{v1}\hat{m}_{v1})}}=\frac{1-\dfrac{k_{v1}}{k_{v2}}\sqrt{\dfrac{c_{v2}}{A}}}{1+\dfrac{k_{v1}}{k_{v2}}\sqrt{\dfrac{c_{v2}}{A}}};$$

$$q=\frac{h_1\sqrt{c_{v2}}-h_2\sqrt{A}}{h_1\sqrt{c_{v2}}+h_2\sqrt{A}};$$

$$\alpha=\frac{h_1\sqrt{c_{v2}}}{h_1\sqrt{c_{v2}}+h_2\sqrt{A}};$$

$$\beta=\frac{h_2\sqrt{A}}{h_1\sqrt{c_{v2}}+h_2\sqrt{A}};$$

$$T_v=\frac{Ac_{v2}t}{(h_1\sqrt{c_{v2}}+h_2\sqrt{A})^2};$$

$$T_c=\frac{Ac_{v2}t_c}{(h_1\sqrt{c_{v2}}+h_2\sqrt{A})^2} \quad (23)$$

Referring to ZHU and YIN [21], the solution for Eq. (18) can be readily obtained as

$$\hat{u}(z,T_v)=\sum_{m=1}^{+\infty}T_m(T_v)Z_m(z) \quad (24)$$

where

$$T_m(T_v)=\begin{cases}\dfrac{b_m}{\lambda_m^3T_c}\left[1-\exp(-\lambda_m^2T_v)\right],\ T_v\le T_c\\ \dfrac{b_m}{\lambda_m^3T_c}\left[1-\exp(-\lambda_m^2T_c)\right]\exp\left[-\lambda_m^2(T_v-T_c)\right],\\ \quad T_v>T_c\end{cases} \quad (25)$$

For the case of PTIB, the constant b_m in Eq. (25) and the function $Z_m(z)$ in Eq. (24) can be derived:

$$b_m=\left\{\left[\frac{\hat{m}_{v1}h_1}{\alpha\sin(\lambda_m\alpha)}\right]n^2\sigma_0+\left(\frac{\hat{m}_{v1}h_1}{\alpha^2\lambda_m}\right)n^2(\sigma_1-\sigma_0)+\left[\frac{m_{v2}h_2[\cos(\lambda_m\beta)-1]}{\beta^2\lambda_m\cos(\lambda_m\beta)}\right]n^2(\sigma_1-\sigma_2)\right\}\Bigg/\left\{\left[\frac{\hat{m}_{v1}h_1}{2\sin^2(\lambda_m\alpha)}\right]+\left[\frac{m_{v2}h_2}{2\cos^2(\lambda_m\beta)}\right]\right\} \quad (26)$$

$$Z_m(z)=\begin{cases}\dfrac{\sin[\lambda_m\alpha(z/h_1)]}{\sin(\lambda_m\alpha)},\ 0\le z\le h_1\\ \dfrac{\cos\{\lambda_m\alpha(z/h_1)\}}{\sin(\lambda_m\alpha)},h_1\le z\le H\end{cases} \quad (27)$$

The constant λ_m is the mth positive root of the following equation for variable θ:

$$\cos\theta-p\cos(q\theta)=0 \quad (28)$$

Substituting Eq. (17) into Eq. (24), the analytical solution for consolidation of soft ground improved by floating soil−cement column can be derived as

$$\begin{cases}\overline{u}_{s1}=\dfrac{1}{n^2-1}\displaystyle\sum_{m=1}^{+\infty}T_m(T_v)Z_m(z),\ 0\le z\le h_1\\ \overline{u}_{s2}=\dfrac{1}{n^2}\displaystyle\sum_{m=1}^{+\infty}T_m(T_v)Z_m(z),\ h_1\le z\le H\end{cases} \quad (29)$$

where $T_m(T_v)$, b_m, $Z_m(z)$ can be expressed as Eqs. (25), (26), (27), respectively.

The average degree of consolidation is defined as the ratio of the settlement at time t to the final settlement of the soft ground improved by floating soil−cement column is defined

$$U(T_v)=S_t/S_\infty \tag{30}$$

First, the average pore water pressure at the cross-sectional areas of the soil−cement column improved layer:

$$\bar{\bar{u}}_{s1}=\frac{\bar{u}_{s1}\pi(r_e^2-r_c^2)}{\pi r_e^2}=\frac{\bar{u}_{s1}(n^2-1)}{n^2} \tag{31}$$

The settlement of soft ground improved by floating soil−cement column at time t is expressed:

$$S_t=\int_0^H \varepsilon(z,t)\mathrm{d}z=\int_0^{h_1}\left[\sigma(z,t)-\bar{\bar{u}}_{s1}(z,T_v)\right]\hat{m}_{v1}\mathrm{d}z+\int_{h_1}^{h_2}\left[\sigma(z,t)-\bar{u}_{s2}(z,T_v)\right]m_{v2}\mathrm{d}z \tag{32}$$

The final settlement when the pore water pressure elapsed to zero is expressed as

$$S_t=\int_0^H \varepsilon(z,t=\infty)\mathrm{d}z=\int_0^{h_1}\sigma(z,t=\infty)\hat{m}_{v1}\mathrm{d}z+\int_{h_1}^{h_2}\sigma(z,t=\infty)m_{v2}\mathrm{d}z \tag{33}$$

So, the average degree of consolidation for the case of PTIB is derived as

$$U(T_v)=\min(1,\frac{T_v}{T_c})-\sum_{m=1}^{\infty}2\hat{m}_{v1}h_1T_m(T_v)/\left\{\lambda_m\alpha\sin(\lambda_m\alpha)n^2\cdot\left[\hat{m}_{v1}h_1(\sigma_0+\sigma_1)+m_{v2}h_2(\sigma_1+\sigma_2)\right]\right\} \tag{34}$$

The solution of consolidation for the case of PTPB (the top and bottom surfaces of soft ground are assumed to be pervious) can be derived by similar method. So, it is omitted here.

3 Parametric analysis of consolidation behavior

In this section, parametric analysis is performed to investigate the consolidation behavior of soft ground improved by floating soil−cement column. In the analysis, we introduce $\tilde{T}_v=c_{v2}t/H^2$ as the time axis for general significance which differs from $T_v=Ac_{v2}t/(h_1\sqrt{c_{v2}}+h_2\sqrt{A})^2$ defined in past section. For simplified analysis, the variation of constrained modulus of soil in disturbed zone is ignored and it is assumed that the $E_{s1}(r)=E_{s1}$.

First, we focus on the influence of depth replacement ratio ($\beta=h_1/H$, h_1 is the length of the column and H is the thickness of the soft ground). Figure 3 shows that the consolidation rate of soft ground improved by floating soil−cement column is highly influenced by the depth replacement ratio. When $0<h_1/H\le1$, the consolidation rate increases with increasing h_1/H. As shown in Fig. 3, when the depth replacement ratio is small ($h_1/H<0.7$ in this example), the consolidation rate is slower than that of the natural ground ($h_1/H=0$). When the depth replacement ratio is large ($h_1/H>0.7$ in this example), the consolidation rate is more rapid than that of the natural ground ($h_1/H=0$). When the soft ground is penetrated by the soil−cement column (i.e. $h_1/H=1$), the average degree of consolidation is at its maximum. We think that the soil−cement column will take a much higher proportion of the external load for its much higher constrained modulus than the surrounding soil. So, it will accelerate the consolidation of the column improved layer. On the other hand, the lower permeability of the soil−cement column will also cut down the drainage path for the underlying layer. So, it will also reduce the consolidation of the underlying layer. The consolidation rate of soft ground improved by floating soil−cement column is comprehensive reflection of consolidation of the column improved layer and underlying layer. When the depth replacement ratio is large, the consolidation acceleration of the column improved layer has more important effect during the consolidation of this system; while, when the depth replacement ratio is small, the consolidation acceleration of the underlying layer has more important effect.

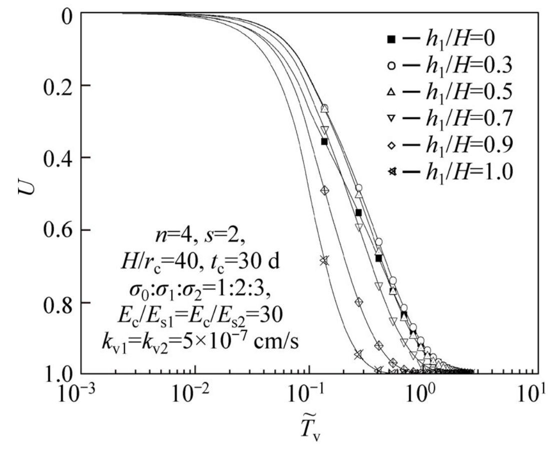

Fig. 3 Influence of depth replacement ratio on consolidation behavior

We turn to discuss the influence of properties of double soil layers. Here, we assume two cases which keep c_{v1}, c_{v2} constant ($c_{v1}=\frac{k_{v1}E_{s1}}{r_w}$, $c_{v2}=\frac{k_{v2}E_{s2}}{r_w}$). As shown in Fig. 4, the average degree of consolidation for the two cases differs much from each other. For the consolidation analysis of soft ground improved by floating soil−cement column, the permeability and

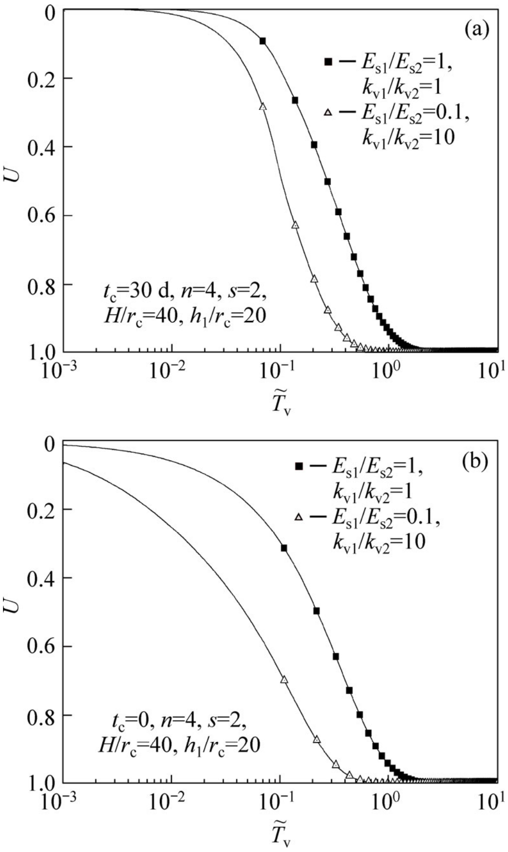

Fig. 4 Influence of permeability of upper layer on consolidation behavior: (a) For t_c=30 d; (b) For t_c=0

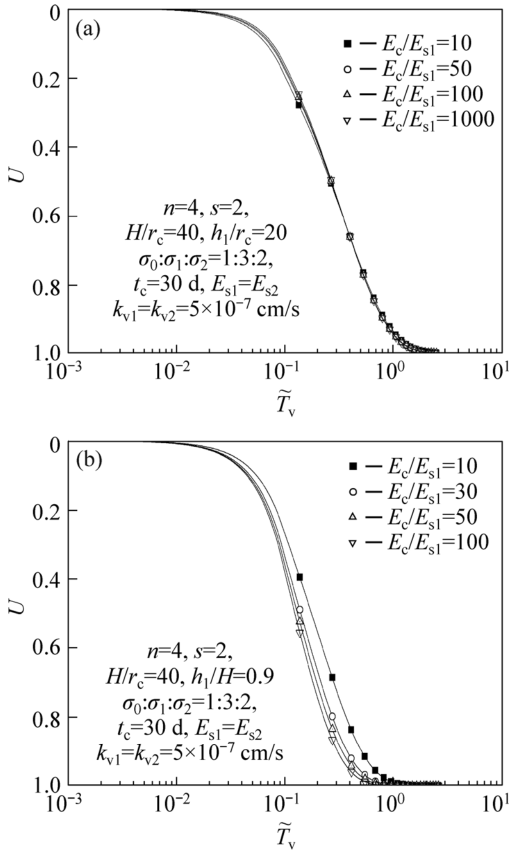

Fig. 5 Influence of constrained modulus ratio of column to soil on consolidation behavior: (a) For h_1/H=0.5; (b) For h_1/H=0.9

compressibility of double soil layers should be taken into consideration at the same time. For further research, it can be seen that the permeability of upper layer plays an important role in PTIB pattern. When we keep c_{v1}, c_{v2} as constants, the larger the permeability of upper layer, the more rapid the consolidation rate. For instantly loading (t_c=0), the regularity is more obvious.

The influence of constrained modulus ratio of the column to soil is investigated in Fig. 5. It can be seen that the influence of constrained modulus ratio on consolidation is also affected by depth replacement ratio. When the depth replacement ratio is small (h_1/H=0.5 in this example), the consolidation rate keeps almost constant for different constrained modulus ratio. When the depth replacement ratio is large (h_1/H=0.9 in this example), the consolidation rate increases with increasing constrained modulus ratio E_c/E_{s1}.

As shown in Fig. 6, the influence of radius ratio (which reflects the relative size of the radius of the influence zone compared with the radius of the column) is also affected by depth replacement ratio. When the depth replacement ratio is small (h_1/H=0.5 in this example), the consolidation rate keeps almost constant. When the depth replacement ratio is large (h_1/H=0.9 in this example), the consolidation rate increases with decreasing radius ratio of the influence zone to the

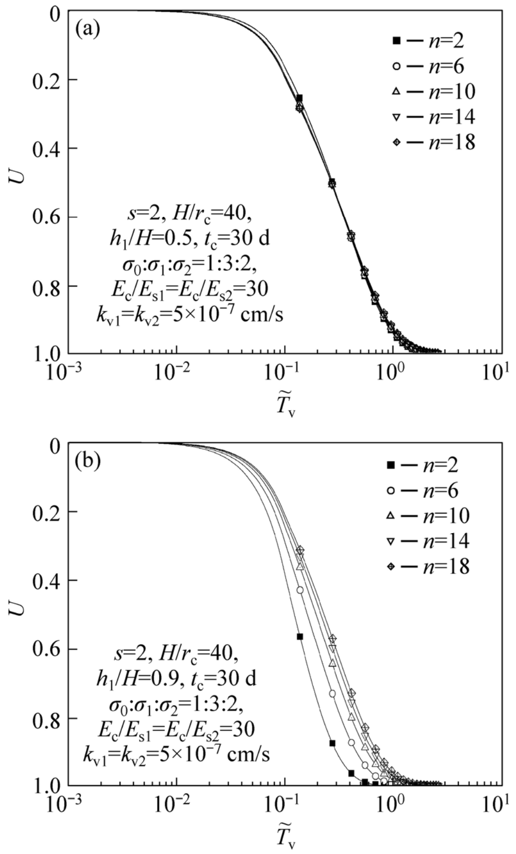

Fig. 6 Influence of radius ratio on consolidation behavior: (a) For h_1/H=0.5; (b) For h_1/H=0.9

column.

As revealed in Fig. 7, the loading period exerts a great influence on the average degree of consolidation. The longer the loading period, the slower the consolidation rate. For instantly loading (t_c=0), the average degree of consolidation is at its maximum.

Figure 8 shows a series of dissipation curves for the excess pore water pressure with different loading periods. It can be seen that the excess pore water pressure dissipates more rapidly with a reduction of the loading period. For instantly loading (t_c=0), the dissipation of the excess pore water pressure is most rapidly. It also reveals that the excess pore water pressure dissipates more rapidly at the column improved layer (for z/H=0.25 in this example) than the underlying layer (for z/H=0.75 in this example) in the PTIB case.

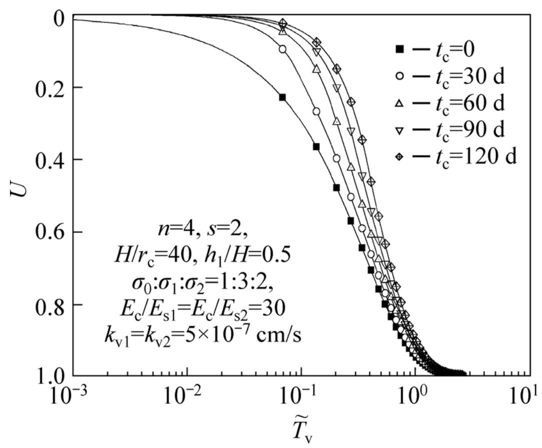

Fig. 7 Influence of loading period on consolidation behavior

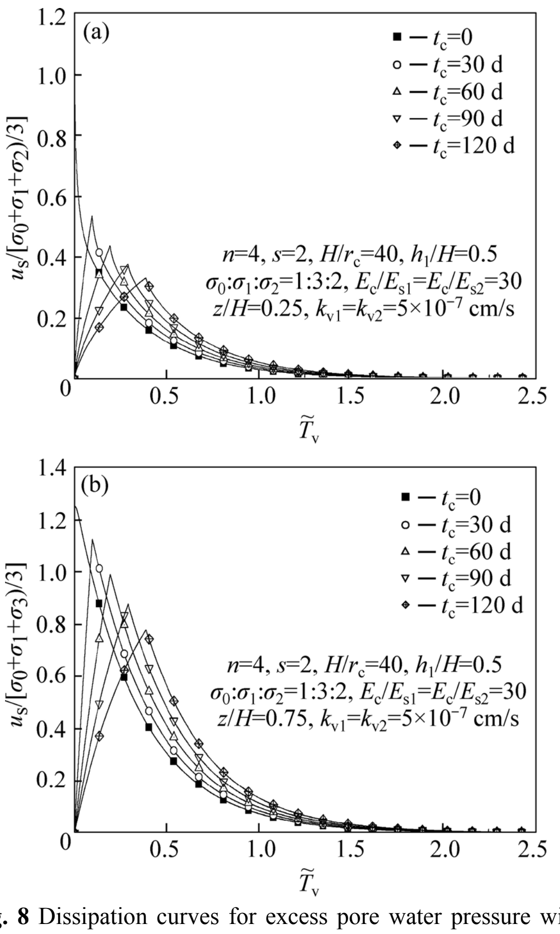

Fig. 8 Dissipation curves for excess pore water pressure with different loading periods: (a) For z/H=0.25; (b) For z/H=0.75

It is found that the column-to-soil total stress ratio at a given depth increases with time and approaches the final value in Fig. 9. So, it means that the stress transfers from the soil to the column during consolidation gradually.

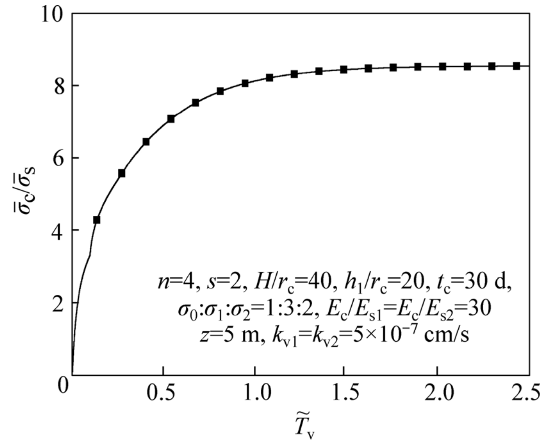

Fig. 9 Development curve of column-to-soil total stress ratio with time at z/H=0.25

4 Conclusions

1) The consolidation behavior is closely related with the depth replacement ratio by the column. The consolidation rate increases with increasing h_1/H. When the depth replacement ratio is small, it is slower than the natural ground. When the depth replacement ratio is large, the reverse turns to be true.

2) The permeability of upper soil layer has a great influence on the consolidation behavior even if the consolidation coefficients of the upper and underlying soil layers keep the same. The larger the permeability of upper layer, the more rapid the consolidation rate.

3) The influence of column-to-soil constrained modulus ratio and radius ratio of the influence zone to the column on consolidation is also affected by depth replacement ratio. When the depth replacement ratio is small, the influence is limited. When the depth replacement ratio is large, the consolidation rate increases with increasing constrained modulus ratio E_c/E_{s1} and decreasing radius ratio of the influence zone to the column.

4) The loading period exerts a great influence on the average degree of consolidation. The longer the loading period, the slower the consolidation rate.

5) The column-to-soil total stress ratio increases with time and approaches the final value gradually accompanied with the dissipation of excess pore water

pressure.

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