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Analysis of 1D large strain consolidation of structured marine soft clays*

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Abstract: Structured soft clays are widely distributed in the coastal regions of China. The characteristics of the natural structure greatly influence the compressibility and permeability of marine soft clays and should be considered in the theory of their consolidation. When a large surcharge load is applied to a structured clay deposit, large strains can be induced in the clay layer due to the high compressibility, where the consolidation process follows the large strain assumption. However, there are few published theories of consolidation in which both the natural structure of marine soft clays and the large strain assumption can be considered simultaneously. In this study, a novel large strain consolidation model for structured marine soft clays was developed by considering the variation of structural yield stress with depth and using different calculation methods for initial effective stress of structured clay deposits in the Lagrangian coordinate system. The corresponding solution was derived by the finite difference method. Finally, the influences of the natural structure of soft clays and different geometric assumptions on consolidation behavior were investigated. The results show that the dissipation rate of excess pore water pressure of structured clays under the large strain assumption is expected to be faster than that under the small strain assumption, and the difference in consolidation behavior between the two assumptions increases with the strain level of natural structured clays. If the strain level in the clay layer is more than 15%, the difference in consolidation behavior between the large and small strain assumptions must be considered.

Key words: Structured soft clays; Large strain consolidation; Structural yield stress; Finite difference method
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1 Introduction

With the rapid economic development in the southeast coastal regions of China, infrastructure construction has increased dramatically in recent years. Structured marine soft clays are widely distributed in such coastal regions (Zeng et al., 2011). Therefore, the consolidation of structured marine soft clays needs to be assessed carefully when the foundations of superstructures or high embankments are designed or constructed in these regions. The properties of marine soft clays are greatly influenced by their natural structure, which can cause a considerable difference in shear strength and compressibility between structured and disturbed clays (Burland, 1990; Chai et al., 2004; Mataic et al., 2016). If the effective stress is greater than the yield stress, the natural structure of marine soft clays would be damaged, leading to a sudden increase in settlement and an obvious reduction in permeability (Xie et al., 2016). Under this condition, by neglecting the influence of

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the natural structure of marine soft clays, analysis using the estimated compressibility and permeability based on disturbed samples could result in erroneous predictions for consolidation settlement (Hong et al., 2012; Hu et al., 2018). Therefore, it is imperative to consider explicitly the influence of the characteristics of the natural structure on the variation in compressibility and permeability of structured marine soft clays.

Wang and Chen (2003) derived an approximate solution for 1D consolidation of structured clays by considering the yield stress as the criterion to distinguish different consolidation behaviors with a constant coefficient of consolidation. Cao et al. (2006) obtained numerical solutions for 1D consolidation of structured clays using nonlinearly varying compressibility and permeability. Chen et al. (2007) introduced a moving boundary to simulate the change of the natural structure of clays, and derived solutions for consolidation of structured clays with vertical drains. Ozelim et al. (2015) modelled the natural structure of clays as a telescopic collapsible structure associated with springs to correctly predict the compressibility of structured clays. Xie et al. (2016) proposed approximate analytical solutions for 1D consolidation of double-layered structured clays under time-dependent loading. Hu et al. (2018) provided numerical solutions for 1D nonlinear consolidation of structured clays using a semi-analytical method. Note that all the above consolidation theories of structured clays incorporate the small strain assumption. However, when the ultimate stress exceeds the yield stress, the strain of soft structured clays with high compressibility may exceed 15%. As noted by Indraratna et al. (2017), 15% can be regarded as a threshold between small and large strains. Once the vertical strain goes beyond 15%, large strain analysis must be incorporated, otherwise a considerable difference between large and small strains occurs. Previous studies also showed that the prediction of consolidation settlement under the small strain assumption could deviate significantly from field observations when the strain of clays is large enough (Weber, 1969; Cargill, 1984). Therefore, large strain consolidation theory should be applied to calculate the consolidation settlement when the vertical strain of structured marine soft clays is over 15%.

Since large strain consolidation theory, initially developed by Gibson et al. (1967) by employing the void ratio as a variable, numerical and analytical aspects of the theory of large strain consolidation have subsequently been improved (Gibson et al., 1981; Cargill, 1984; Tan and Scott, 1988; Townsend and McVay, 1990; Morris, 2002; Xie and Leo, 2004). Li et al. (2017b) presented a numerical solution for 1D large strain consolidation of clays with non-Darcian flow which could exist in fine grained clays under a low hydraulic gradient. Pu et al. (2018) proposed numerical solutions for self-weight consolidation of a saturated slurry layer under the large strain assumption. However, all these studies ignored the effect of the natural structure of clays on consolidation behavior. When the effective stress is less than the yield stress of structured clays, the influence of the natural structure of clays on their compressibility and permeability must be considered in the large strain theory of consolidation.

In conclusion, the small strain assumption has been adopted in the all existing consolidation theories which incorporate the influences of the natural structure on the variation of compressibility and permeability. Furthermore, previous studies on large strain consolidation ignored the effect of the natural structure of clays on consolidation behavior. Consolidation theories which consider the large strain assumption and the natural structure of soft soils have rarely been reported in the literature. The objective of this study was to develop a large strain consolidation theory for normally consolidated structured soft clays, in which the variation of compressibility, permeability, and yield stress of structural soils is considered. The solutions for this model were derived using the finite difference method to investigate the difference in consolidation behavior between large and small strain assumptions. The influence of the natural structure of marine soft clays on consolidation behavior was also investigated.

2 Compressibility and permeability of structured clays

The difference in compressibility between structured and disturbed clays has been investigated (Wang et al., 2004; Zeng et al., 2011). For structured clays (Fig. 1), the compression curves in the e–lgσ' coordinates determined by oedometer tests can be divided into a pre-yield regime and a post-yield regime according to the yield stress σ_y'. In the pre-yield regime, the effective stress σ' varies between the initial effective stress σ_0' and the yield stress σ_y', in which the compressibility of structured clays is small. Once the effective stress exceeds the yield stress, the compressibility of structured clays increases rapidly. For simplicity, the compression curve of structured clays can be simplified in the e–lgσ' coordinates (Fig. 2) (Wang et al., 2004), and the following equations are adopted to describe their compressibility:

$$e = e_y - C_{cn}(\lg\sigma' - \lg\sigma_y'),\ \ \sigma_y' > \sigma' \geq \sigma_0', \tag{1}$$

$$e = e_1 - C_{cr}(\lg\sigma' - \lg\sigma_1'),\ \ \sigma' \geq \sigma_y', \tag{2}$$

where e is the void ratio, e_y is the void ratio at $\sigma'=\sigma_y'$, C_{cn} is the compression index in the pre-yield regime, C_{cr} is the compression index in the post-yield regime, σ_1' is an arbitrary effective stress in the post-yield curve or its extension, and e_1 is the void ratio corresponding to σ_1'. The yield stress σ_y' of structured soil varies with depth in a soil deposit, and depends on the self-weight stress of natural structured clays (Nagaraj et al., 1990). The yield stress is influenced mainly by the secondary compression and chemical cementation. For the secondary compression, the relationship between the yield stress and initial effective stress is near to linear. For chemical cementation, the yield stress can be calculated by the initial effective stress plus chemical cementation. Therefore, the relationship between structured yield stress and initial effective stress suggested by Wang and Li (2007) can be expressed as

$$\sigma_y' = k_1\sigma_0' + k_2, \tag{3}$$

where k_1 and k_2 are the fitting parameters that can be determined from compression tests of several soil samples from different depths. Eq. (3) can be used to determine the structured yield stress of the whole soil deposit at different depths. With Eqs. (1) and (2), the following equations can be derived:

$$\frac{\partial e}{\partial \sigma'} = -\frac{C_{cn}}{\sigma' \ln 10},\ \ \sigma_y' > \sigma' \geq \sigma_0', \tag{4}$$

$$\frac{\partial e}{\partial \sigma'} = -\frac{C_{cr}}{\sigma' \ln 10},\ \ \sigma' \geq \sigma_y'. \tag{5}$$

The void ratio decreases with increasing effective stress, along with a decrease of permeability. The relationship e–lgk_v is usually used to describe the variation of permeability. Some studies showed that the variation of permeability of structured clays reasonably follows the relationship e–lgk_v (Horpibulsuk et al., 2007; Zeng et al., 2011). Therefore, the permeability of natural structured clays during the consolidation process can be expressed as

$$e - e_1 = C_k\left(\lg k_v - \lg k_{v1}\right), \tag{6}$$

where C_k is the permeability index, k_v is the coefficient of permeability of natural structured clays, and k_{v1} is the coefficient of permeability corresponding to the void ratio e_1. With Eqs. (1), (2), and (6), the coefficient of permeability of natural structured clays varies with the effective stress as follows:

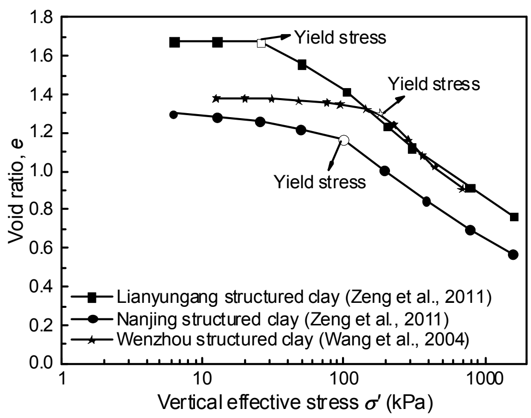

Fig. 1 Compressibility of natural structured clays

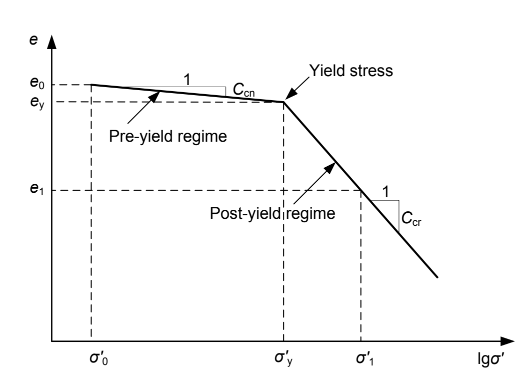

Fig. 2 Simplified compression model of structured clays

e_0 is the initial void ratio of natural structured clays

$$k_{\mathrm{v}}=k_{\mathrm{v1}}\left(\frac{\sigma_1'}{\sigma_{\mathrm{y}}'}\right)^{\frac{C_{\mathrm{cr}}}{C_{\mathrm{k}}}}\left(\frac{\sigma_{\mathrm{y}}'}{\sigma'}\right)^{\frac{C_{\mathrm{cn}}}{C_{\mathrm{k}}}},\quad \sigma_{\mathrm{y}}'>\sigma'\geq\sigma_0', \tag{7}$$

$$k_{\mathrm{v}}=k_{\mathrm{v1}}\left(\frac{\sigma_1'}{\sigma'}\right)^{\frac{C_{\mathrm{cr}}}{C_{\mathrm{k}}}},\quad \sigma'\geq\sigma_{\mathrm{y}}'. \tag{8}$$

3 Model of large strain consolidation

A schematic of large strain consolidation of a structured clay layer is shown in Fig. 3, where the clay deposit is assumed to be saturated. The original thickness of the soft clay deposit is H. The top surface of the clay layer is permeable, and the bottom surface impermeable. The governing equations of the problem are investigated in the Lagrangian and convective coordinates. The top surface of the clay layer is referred to as a=0 in the Lagrangian coordinate system, and the bottom surface is referred to as a=H. A datum plane of ξ=0 is defined in the convective coordinate, which corresponds to the top surface in the Lagrangian coordinate, and the settlement of the top surface at time t is denoted by S_t. The settlement of clay at the Lagrangian coordinate position a and time t is denoted by $S_t(a, t)$, and as such the relationship between the Lagrangian coordinate a and the convective coordinate ξ is expressed by $\xi=a+S_t(a, t)$. Under this condition, the top and bottom surfaces are referred to as $\xi=S_t$ and $\xi=H$ in the convective coordinates, respectively. The deformation of the clay layer can reach a stable state under its self-weight stress σ_0'. A time-dependent surcharge pressure $q(t)$ is applied at the top surface of the clay layer. As shown in Fig. 4, q_0 and q_{u} are the initial and ultimate values of the external loading, respectively, and t_{c} is the construction time. The conditions of instantaneous loading and ramp loading are two special cases of time-dependent loading.

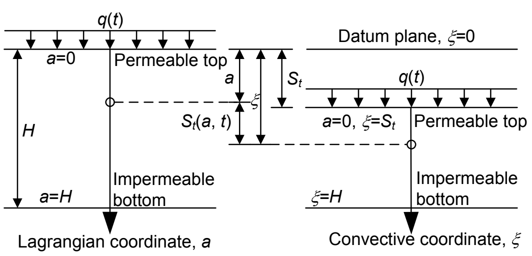

Fig. 3 Lagrangian and convective coordinates

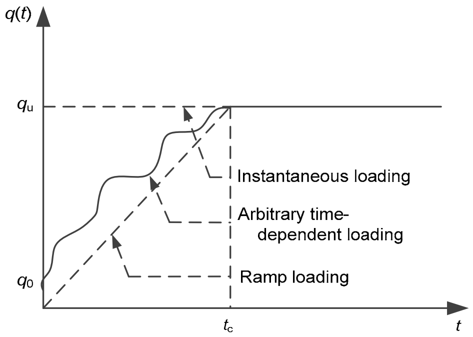

Fig. 4 Time-dependent loading

To develop the governing equations of 1D large strain consolidation of natural structured clays, the following assumptions were made:

(1) The structured clay layer is saturated, and both the clay particles and pore water in the void are incompressible;

(2) Darcy's law is used to describe the water flow in natural structured clays;

(3) The variation of compressibility of natural structured clays follows Eqs. (1) and (2), and the variation of permeability follows Eqs. (7) and (8);

(4) The large strain assumption is adopted to derive the settlement of soft marine clays during the consolidation process.

Based on the relationship between Lagrangian and convective coordinates proposed by Gibson et al. (1967), the hydraulic gradient i in the Lagrangian coordinate can be expressed as

$$i=\frac{1}{\gamma_{\mathrm{w}}}\frac{1+e_0}{1+e}\frac{\partial u}{\partial a}, \tag{9}$$

where γ_{w} is the unit weight of water, e is the void ratio as a function of time t and Lagrangian coordinate a, e_0 is the initial void ratio of natural structured clays, which is determined by the initial effective stress σ_0', and u is the excess pore water pressure. The constitutive relationship of water flow in clays can be described by Darcy's law as follows:

$$v=-\frac{k_{\mathrm{v}}}{\gamma_{\mathrm{w}}}\frac{1+e_0}{1+e}\frac{\partial u}{\partial a}, \tag{10}$$

where v is the average velocity of water flow in clays relative to clay particles in the Lagrangian coordinate.

Based on Xie and Leo (2004), the continuity condition for 1D large strain consolidation in the Lagrangian coordinate is written as

$$\frac{\partial v}{\partial a}=-\frac{1}{1+e_0}\frac{\partial e}{\partial t}. \tag{11}$$

In this study, the creep effect of the clay skeleton on consolidation was ignored. Hence, the void ratio of natural structured clay is dependent only on the change of effective stress, and thus Eq. (11) can be rewritten as

$$\frac{\partial v}{\partial a}=-\frac{1}{1+e_0}\frac{\partial e}{\partial \sigma'}\frac{\partial \sigma'}{\partial t}. \tag{12}$$

The effective stress principle during the process of consolidation can be expressed as

$$\sigma'=\sigma_0'+q(t)-u. \tag{13}$$

Substituting Eqs. (10) and (13) into Eq. (12), the following equation can be obtained:

$$\frac{\partial}{\partial a}\left(\frac{k_{\mathrm{v}}}{\gamma_{\mathrm{w}}}\frac{1+e_0}{1+e}\frac{\partial u}{\partial a}\right)=\frac{1}{1+e_0}\frac{\partial e}{\partial \sigma'}\left(\frac{\mathrm{d}q}{\mathrm{d}t}-\frac{\partial u}{\partial t}\right). \tag{14}$$

Substituting Eqs. (4), (5), (7), and (8) into Eq. (14), the governing equations of 1D large strain consolidation of natural structured clays in the Lagrangian coordinate are:

$$c_{\mathrm{v1}}\frac{C_{\mathrm{cr}}}{C_{\mathrm{cn}}}\frac{1+e_0}{1+e_1}\frac{\sigma'}{\sigma_1'}\frac{\partial}{\partial a}\left[\left(\frac{\sigma_1'}{\sigma_{\mathrm{y}}'}\right)^{\frac{C_{\mathrm{cr}}}{C_{\mathrm{k}}}}\left(\frac{\sigma_{\mathrm{y}}'}{\sigma'}\right)^{\frac{C_{\mathrm{cn}}}{C_{\mathrm{k}}}}\frac{1+e_0}{1+e}\frac{\partial u}{\partial a}\right]=\frac{\partial u}{\partial t}-\frac{\mathrm{d}q}{\mathrm{d}t},\quad \sigma_{\mathrm{y}}'>\sigma'\geq\sigma_0', \tag{15}$$

$$c_{\mathrm{v1}}\frac{1+e_0}{1+e_1}\frac{\sigma'}{\sigma_1'}\frac{\partial}{\partial a}\left[\left(\frac{\sigma_1'}{\sigma'}\right)^{\frac{C_{\mathrm{cr}}}{C_{\mathrm{k}}}}\frac{1+e_0}{1+e}\frac{\partial u}{\partial a}\right]=\frac{\partial u}{\partial t}-\frac{\mathrm{d}q}{\mathrm{d}t},\quad \sigma'\geq\sigma_{\mathrm{y}}', \tag{16}$$

where $c_{\mathrm{v1}}=\frac{k_{\mathrm{v1}}}{\gamma_{\mathrm{w}}}\frac{(1+e_1)\sigma_1'\ln 10}{C_{\mathrm{cr}}}$.

The top surface of the clay layer is permeable, and the bottom surface impermeable, expressed in the Lagrangian coordinate under the large strain assumption as follows:

$$u(0,t)=0, \tag{17}$$

$$\left.\frac{\partial u}{\partial a}\right|_{a=H}=0. \tag{18}$$

The initial condition for the consolidation model is

$$u(z,0)=q_0. \tag{19}$$

The initial effective stress induced by self-weight stress can be determined by two methods in the theory of nonlinear consolidation.

1. The initial effective stress increases linearly with depth as determined by

$$\sigma_0'=(\gamma_{\mathrm{sat}}-\gamma_{\mathrm{w}})a, \tag{20}$$

where γ_{sat} is the saturated unit weigtht of clay.

2. When the influence of sedimentation on the initial effective stress σ_0' is considered (de Simone and Viggiani, 1975), the initial effective stress in the Lagrangian coordinate can be calculated by

$$\left(1+e_1+\frac{C_{\mathrm{cr}}}{\ln 10}\right)\sigma_0'-C_{\mathrm{cr}}\sigma_0'\lg\left(\frac{\sigma_0'}{\sigma_1'}\right)=\gamma_{\mathrm{w}}(G_{\mathrm{s}}-1)a, \tag{21}$$

where G_{s} is the specific gravity of clay particles. Furthermore, the initial void ratio e_0 can be calculated by Eq. (1) when $\sigma'=\sigma_0'$. It can be expressed as

$$e_0=e_{\mathrm{y}}-C_{\mathrm{cn}}(\lg\sigma_0'-\lg\sigma_{\mathrm{y}}'). \tag{22}$$

4 Numerical solutions for the model

4.1 Dimensionless variables

To obtain the numerical solutions for the above governing equations, the following dimensionless variables are defined as

$$Z=\frac{a}{H},\ U=\frac{u}{\sigma_1'},\ Q=\frac{q(t)}{\sigma_1'},$$
$$Q^{\mathrm{u}}=\frac{q_{\mathrm{u}}}{\sigma_1'},\ Q^0=\frac{q_0}{\sigma_1'},\ T_{\mathrm{v}}=\frac{c_{\mathrm{v}1}t}{H^2},\tag{23}$$
$$T_{\mathrm{vc}}=\frac{c_{\mathrm{v}1}t_{\mathrm{c}}}{H^2},\ S=\frac{\sigma_0'}{\sigma_1'},\ Y=\frac{\sigma_{\mathrm{y}}'}{\sigma_1'}.$$

By using the above dimensionless variables, Eqs. (15) and (16), which govern the large strain consolidation of natural structured clays, can be rewritten as

$$\beta\frac{1+e_0}{1+e_1}\frac{\partial}{\partial Z}\left[\alpha\frac{1+e_0}{1+e}\frac{\partial U}{\partial Z}\right]=\frac{\partial U}{\partial T_{\mathrm{v}}}-\frac{\mathrm{d}Q}{\mathrm{d}T_{\mathrm{v}}},\tag{24}$$

where α and β are presented in Eqs. (A1) and (A2) in Appendix A, respectively. In terms of these dimensionless variables, Eqs. (17)–(19) can be expressed as

$$U(Z,T_{\mathrm{v}})\big|_{Z=0}=0,\tag{25}$$

$$\left.\frac{\partial U(Z,T_{\mathrm{v}})}{\partial Z}\right|_{Z=1}=0,\tag{26}$$

$$U(Z,T_{\mathrm{v}})\big|_{T_{\mathrm{v}}=0}=Q^0.\tag{27}$$

4.2 Finite difference solutions for excess pore water pressure

To solve the above equations numerically, a differential grid is placed in the normalized depth versus time factor (Z, T_{v}) plane with a spatial isometric step ΔZ and a non-isometric time step $\Delta T_{\mathrm{v}k}$. As illustrated in Fig. 5, the jth nodal point in the spatial domain is denoted by Z_j (i.e. $Z_j=j\Delta Z$, j=0, 1, 2, …, n). The time domain T_{v} is also divided into a number of small intervals. If the kth time interval is denoted by $\Delta T_{\mathrm{v}k}$, k=1, 2, 3, …, the final time of the kth time interval, $T_{\mathrm{v}k}$, is:

$$T_{\mathrm{v}k}=\sum_{r=1}^{k}\Delta T_{\mathrm{v}r}.\tag{28}$$

Based on the mesh discretization, the difference equation corresponding to Eq. (24) can be derived following the implicit difference scheme of the partial differential equation:

$$U_j^{k+1}-U_j^k-\left(Q^{k+1}-Q^k\right)$$
$$=\beta_j^k\left[\alpha_{j+1/2}^k\left(U_{j+1}^{k+1}-U_j^{k+1}\right)-\alpha_{j-1/2}^k\left(U_j^{k+1}-U_{j-1}^{k+1}\right)\right],\tag{29}$$

where U_j^k is the dimensionless value of excess pore water pressure at the jth spatial node when $T_{\mathrm{v}}=T_{\mathrm{v}k}$ (j=1, 2, …, n; k=0, 1, 2, …); Q^k is the dimensionless value of external load at time $T_{\mathrm{v}}=T_{\mathrm{v}k}$; parameters β_j^k, $\alpha_{j+1/2}^k$, and $\alpha_{j-1/2}^k$ are given in Eqs. (A3)–(A5) in Appendix A. In terms of the differential grid, the boundary condition and the initial condition can be expressed as

$$U_0^k=0,\tag{30}$$

$$\frac{U_{n+1}^k-U_{n-1}^k}{2\Delta Z}=0,\tag{31}$$

$$U_j^0=Q^0,\ j=1,2,...,n.\tag{32}$$

Based on the criterion adopted by Mishra and Patra (2019), i.e. $\Delta T_{\mathrm{v}k}/\Delta Z^2\le 0.5$, $\Delta T_{\mathrm{v}k}/\Delta Z^2\le 0.1$ was incorporated to ensure the stability and convergence of the finite difference method in the following analysis.

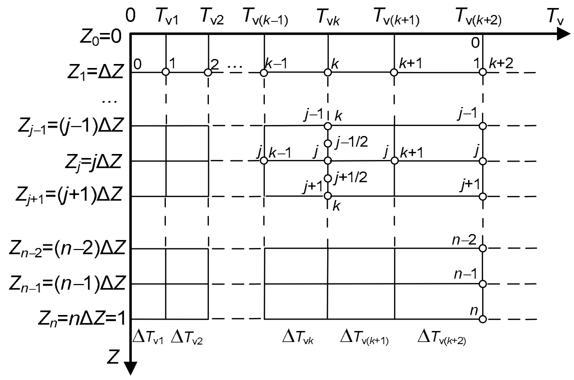

Fig. 5 Discretization of grids in the Z-T_{v} plane

4.3 Solutions for settlement and average degree of consolidation

The initial void ratio of natural structured clays at the jth spatial node can be obtained by

$$e_{0j}=e_{\mathrm{y}}+C_{\mathrm{cn}}\lg\left(S_j/Y_j\right),\tag{33}$$

$$e_{\mathrm{y}j}=e_1+C_{\mathrm{cr}}\lg\left(Y_j\right),\tag{34}$$

where e_{0j} is the initial void ratio at the jth spatial node, $e_{\mathrm{y}j}$ is the void ratio corresponding to the yield stress at the jth spatial node, S_j is the dimensionless value of the initial effective stress at the jth spatial node, and Y_j is the dimensionless value of the yield stress of structured clays at the jth spatial node. The void ratio at the jth spatial node when $T_{\mathrm{v}}=T_{\mathrm{v}k}$ can be calculated as follows:

$$e_j^k=\begin{cases}e_{\mathrm{y}}+C_{\mathrm{cn}}\lg\left(\dfrac{Y_j}{S_j+Q^k-U_j^k}\right), & Y_j>S_j+Q^k-U_j^k\geq S_j,\\ e_1+C_{\mathrm{cr}}\lg\left(\dfrac{1}{S_j+Q^k-U_j^k}\right), & S_j+Q^k-U_j^k\geq Y_j.\end{cases}\tag{35}$$

where e_j^k is the void ratio of natural structured clays at the jth spatial node when $T_{\mathrm{v}}=T_{\mathrm{v}k}$ (j=0, 1, 2, …, n; k=1, 2, 3, …).

The final void ratio at the jth spatial node is determined by

$$e_j^{\infty}=\begin{cases}e_{\mathrm{y}}+C_{\mathrm{cn}}\lg\left(\dfrac{Y_j}{S_j+Q^{\mathrm{u}}}\right), & Y_j>S_j+Q^{\mathrm{u}}\geq S_j,\\ e_1+C_{\mathrm{cr}}\lg\left(\dfrac{1}{S_j+Q^{\mathrm{u}}}\right), & S_j+Q^{\mathrm{u}}\geq Y_j.\end{cases}\tag{36}$$

Using the result of void ratio at time $T_{\mathrm{v}k}$, the corresponding settlement, S_t, of the clay layer surface can be obtained by

$$S_t=\frac{H}{n}\sum_{j=1}^{n}\frac{e_{0j}-e_j^k}{1+e_{0j}}.\tag{37}$$

According to the final void ratio e_j^{∞}, the final settlement of the structured clay layer, S_{∞}, can be determined by

$$S_{\infty}=\frac{H}{n}\sum_{j=1}^{n}\frac{e_{0j}-e_j^{\infty}}{1+e_{0j}}.\tag{38}$$

The average degree of consolidation in terms of strain, U_{st}, is calculated as the settlement ratio at time $T_{\mathrm{v}k}$ and at the end of consolidation:

$$U_{\mathrm{st}}=S_t/S_{\infty}.\tag{39}$$

The average degree of consolidation defined by stress, U_{pt}, can also be evaluated as the average effective stress ratio at time $T_{\mathrm{v}k}$ and in the final stage of consolidation:

$$U_{\mathrm{pt}}=\frac{1}{Q^{\mathrm{u}}}\left(Q^k-\frac{1}{n}\sum_{j=1}^{n}\frac{U_{j-1}^k+U_j^k}{2}\right).\tag{40}$$

5 Evaluation of numerical methods

To evaluate the effectiveness of the above finite difference solution, two types of analysis were performed:

1. When the characteristic of structured clays is ignored (i.e. $C_{\mathrm{cr}}=C_{\mathrm{cn}}$), the large-strain consolidation model of structured clays developed in this investigation is degenerated into the solution of Li et al. (2000). The parameters adopted in the calculation are listed in Table 1, and the comparison results are shown in Fig. 6.

2. When the small strain assumption is adopted, the governing equations for 1D consolidation of natural structured clays are written as follows:

$$c_{\mathrm{v}1}\frac{C_{\mathrm{cr}}}{C_{\mathrm{cn}}}\frac{1+e_0}{1+e_1}\frac{\sigma'}{\sigma_1'}\frac{\partial}{\partial z}\left[\left(\frac{\sigma_1'}{\sigma_{\mathrm{y}}'}\right)^{\frac{C_{\mathrm{cr}}}{C_{\mathrm{k}}}}\left(\frac{\sigma_{\mathrm{y}}'}{\sigma'}\right)^{\frac{C_{\mathrm{cn}}}{C_{\mathrm{k}}}}\frac{\partial u}{\partial z}\right]=\frac{\partial u}{\partial t}-\frac{\mathrm{d}q}{\mathrm{d}t},\quad \sigma_{\mathrm{y}}>\sigma'\geq\sigma_0',\tag{41}$$

$$c_{\mathrm{v}1}\frac{1+e_0}{1+e_1}\frac{\sigma'}{\sigma_1'}\frac{\partial}{\partial z}\left[\left(\frac{\sigma_1'}{\sigma'}\right)^{\frac{C_{\mathrm{cr}}}{C_{\mathrm{k}}}}\frac{\partial u}{\partial z}\right]=\frac{\partial u}{\partial t}-\frac{\mathrm{d}q}{\mathrm{d}t},\ \sigma'\geq\sigma_{\mathrm{y}}',\tag{42}$$

where z is the vertical coordinate. The solution conditions for Eqs. (41) and (42) are the same as those under the large strain assumption. A solution for 1D consolidation of structured clays under the small strain assumption was derived by Li et al. (2017a). Therefore, a comparison between our solution and that of Li et al. (2017a) under the small strain assumption can also be made. The parameters adopted in the calculation are tabulated in Table 2, and the results of the comparison are shown in Fig. 7.

Table 1 Soil parameters used in the comparison against the solution of Li et al. (2000)

Parameter	Value	Parameter	Value
σ_1' (kPa)	50	C_{cr}=C_{cn}	0.6378
e_1	1.571	C_k	0.8
k_{v1} ($\times10^{-8}$ m/s)	1	t_c	0
G_s	2.75	H (m)	20

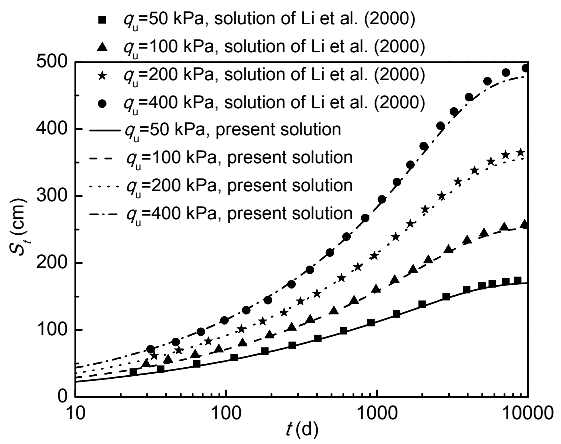

Fig. 6 Settlement comparison calculated using the present solution and the solution of Li et al. (2000)

As shown in Fig. 6 and Fig. 7, the difference in the results calculated using different methods is negligible. For example, the maximum difference in settlement computed using our solution and the solution of Li et al. (2000) occurs at a consolidation period of 25 d for structured clays subjected to a surcharge pressure of q_u=50 kPa. The settlement calculated by semi-analytical method is 32.9 cm, and that calculated by finite differential method is 37.1 cm. The difference in the settlement between the two methods lies within 11%. The average degree of consolidation evaluated by our solution differs from the solution of Li et al. (2017a) by 1% at a time factor of T_v=0.0853. The stability and convergence of the semi-analytical method at the early stage of consolidation is so rigorous that the calculation error induced at the early stage of consolidation is evident. But the stability and convergence of the finite difference method proposed in this study has no bearing on the stage of consolidation. Therefore, there is a difference of settlement between the semi-analytical and finite difference methods at a consolidation period of 25 d for structured clays subjected to a surcharge pressure of q_u=50 kPa. Hence, it can be inferred that the aforementioned numerical method for 1D large strain consolidation of natural structured clays can reproduce the solutions of other methods under special conditions.

Table 2 Soil parameters used in the comparison against the solution of Li et al. (2017a)

Parameter	Value	Parameter	Value
σ_1' (kPa)	50	C_{cr}	0.5
e_1	1.57	C_{cn}	0.09
k_{v1} ($\times10^{-9}$ m/s)	0.815	C_k	0.75
γ_{sat} (kN/m^3)	18	T_{vc}	0.02
k_1	1	q_u (kPa)	120
k_2 (kPa)	75	H (m)	20

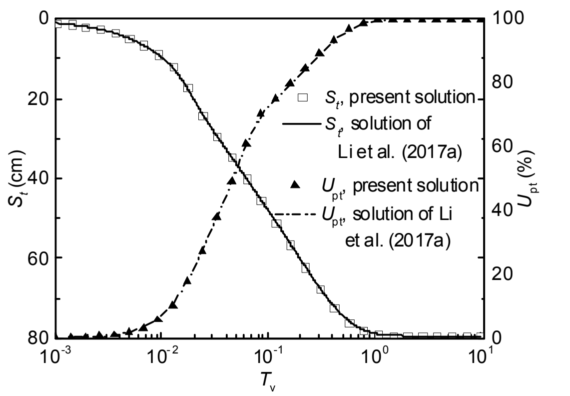

Fig. 7 Comparisons between the present solution and the solution of Li et al. (2017a) under the small strain assumption

6 Analysis of consolidation behavior

To investigate the influence of the natural structure of marine soft clays and different geometric assumptions on consolidation behavior, analyses were conducted. The adopted parameters are given as: σ_1'=50 kPa, e_1=1.57, k_{v1}=8.15×10^{-9} m/s, G_s=2.75, C_{cn}=0.07, C_{cr}=0.85, k_1=1.03, and H=20 m. The other parameters were defined in Table 3. A ramp loading pattern was adopted to simulate the influence of time-dependent loading on consolidation behavior. In the following calculations, the initial effective stress, σ_0', is obtained by Eq. (20) or Eq. (21). The yield stress of structural soils is determined by $\sigma_y'=k_1\sigma_0'+k_2$, and the ultimate applied stress can be obtained by $\sigma_0'+q_u$.

Table 3 Soil parameters adopted in the parametric study

Figure	σ'_0 (kPa)	γ_{sat} (kN/m^3)	C_{cr}/C_k	k_2 (kPa)	t_c (d)	q_u (kPa)
Fig. 8	Eq.(21)	–	1.0	50	300	200, 400
Fig. 9	Eq.(21)	–	1.0	50	300	200, 400
Fig. 10	Eq.(21)	–	1.0	50	300	200, 400
Fig. 11	Eq.(21)	–	1.0	50	0, 600	400
Fig. 12	Eq.(21)	–	1.0	50	0, 600	400
Fig. 13	Eq.(21)	–	0.5, 1.0, 1.5	50	300	400
Fig. 14	Eq.(21)	–	0.5, 1.0, 1.5	50	300	400
Fig. 15	Eq.(20)	18.75	1.0	50	300	400
Fig. 16	Eq.(20)	18.75	1.0	50	300	400
Fig. 17	Eq.(21)	–	1.0	0, 100, 300	300	400
Fig. 18	Eq.(21)	–	1.0	0, 100, 300	300	400
Fig. 19	Eq.(21)	–	1.0	0, 100, 300	300	400

6.1 Influences of q_u on consolidation behavior

Fig. 8 shows the variation of normalized excess pore water pressure with normalized depth under different external loads when T_v=0.5. The residual excess pore water pressure under the large strain assumption is smaller than that under the small strain assumption when the soil is subjected to the same level of external load. Moreover, the difference in residual excess pore water pressure calculated using solutions under large strain and small strain assumptions increases with the external load. That is, the dissipation rate of excess pore water pressure under the large strain assumption is faster than that under the small strain assumption. The difference in consolidation behavior induced by the assumption of consolidation theory can be found in Fig. 9. At the same time factor T_v, the average degree of consolidation, U_{pt}, under the large strain assumption is greater than that under the small strain assumption, and the difference in U_{pt} for solutions between large strain and small strain assumptions becomes more significant for a higher surcharge pressure. The drainage distance under the large strain assumption decreases with increasing consolidation time during the process of consolidation, whereas it remains constant under the small strain assumption. Therefore, the dissipation rate of excess pore water pressure under the large strain assumption should be faster than that under the small strain assumption. In other words, the strain in the structured clay layer in the case of q_u=400 kPa is greater than that in the case of q_u=200 kPa. The difference in consolidation rate between large strain and small strain assumptions increases with increasing strain level in the structured clay layer. When the strain in the clay layer is equal to or less than 15% (the final strain obtained under q_u=200 kPa is less than 15% in this example), it can be seen from Figs. 8, 9, and 10 that the difference in consolidation behavior between large strain and small strain assumptions can be neglected (i.e. the difference is less than 5%). This is generally consistent with the conventional point of view that an excellent approximation to the large strain can be achieved using the small strain analysis when the strain is less than about 15%. When the strain in the clay layer is greater than 15% (the final strain obtained under q_u=400 kPa is about 25% in this example), the difference in consolidation behavior induced by consolidation theory must be considered during the process of consolidation (i.e. the difference is more than 10%). Note that the final settlement of the structured clay layer under the small strain and large strain assumptions is the same for given conditions of external load and soil parameters (Fig. 10).

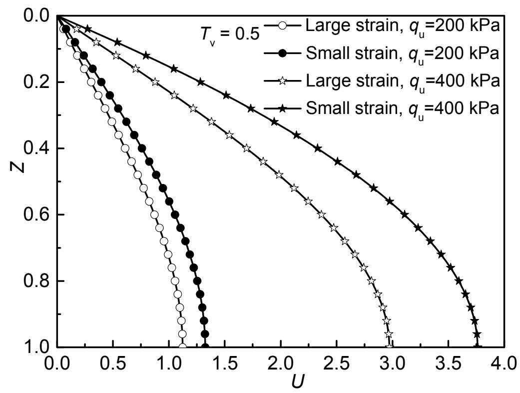

Fig. 8 Influence of q_u on the dissipation of excess pore water pressure

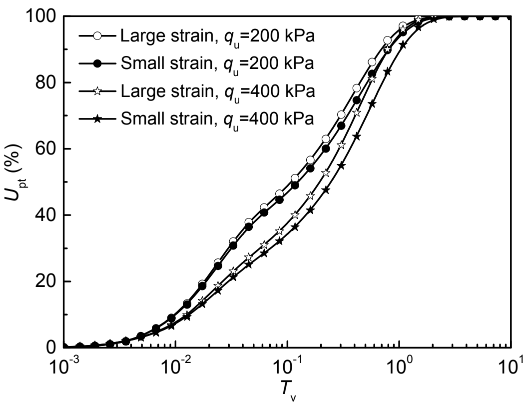

Fig. 9 Influence of q_u on the average degree of consolidation U_{pt}

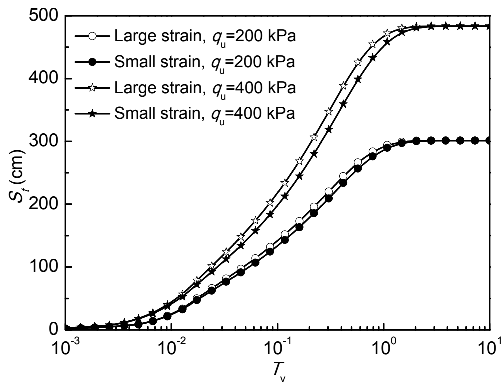

Fig. 10 Influence of q_u on the settlement of the clay layer

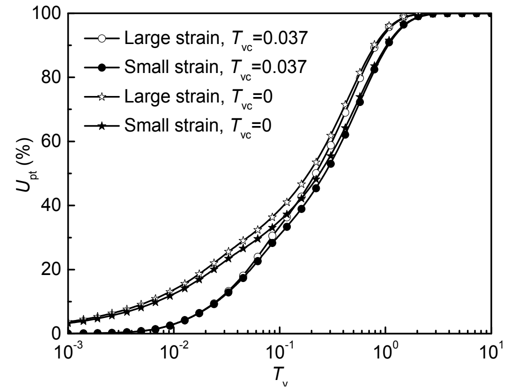

Fig. 11 Influence of t_c on the average degree of consolidation

6.2 Influences of loading rate on consolidation behavior

The construction time t_c is negatively correlated with the loading rate for a ramp loading pattern (Fig. 4). The case of t_c=0 means that the external load is applied at the ground surface of the clay layer instantaneously. The consolidation rate under this case is the fastest. The consolidation rate of natural structured clays decreases with t_c (Figs. 11 and 12). The loading rate does not influence the final strain of natural structured clays, such that the difference in consolidation rate between the large strain and small strain assumptions does not change under different t_c. Furthermore, the final settlement in the natural structured clay layer under different t_c is the same.

6.3 Influences of C_{cr}/C_k on consolidation behavior

When the compression index C_{cr} and the yield stress remain constant, the final settlement of natural structured clays with different C_{cr}/C_k is the same (Figs. 13 and 14). Under the large strain and small strain assumptions, the ratio C_{cr}/C_k can change only the consolidation rate, whereas a higher consolidation rate is obtained under the large strain assumption when C_{cr}/C_k is fixed. Furthermore, when the geometrical assumption is the same, both the average degree of consolidation and the settlement in the clay layer increase with decreasing C_{cr}/C_k.

6.4 Influences of initial effective stress on consolidation behavior

The initial effective stress in the clay layer can be determined by Eq. (20) or (21). Under the same

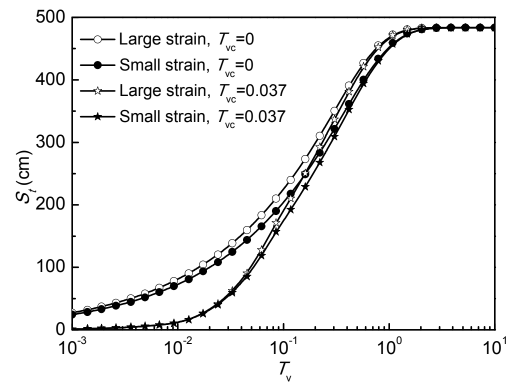

Fig. 12 Influence of t_c on the settlement of the soil layer

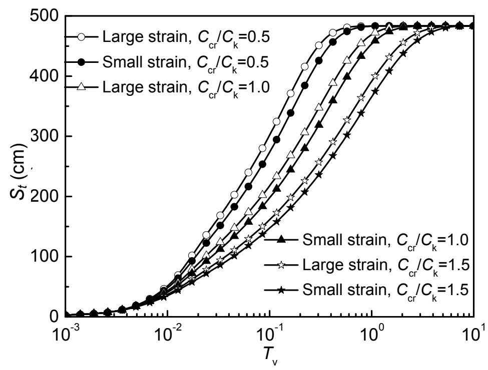

Fig. 13 Influence of C_{cr}/C_k on the settlement of structured clay layers

geometric assumption, the method for determining the self-weight stress has a negligible influence on the dissipation rate of excess pore water pressure (Fig. 15). However, the settlement in the natural structured clay layer based on the initial effective stress calculated by Eq. (21) is greater than that assessed by Eq. (20) (Fig. 16). The initial effective stress induced by self-weight stress greatly influences the settlement. If the calculation method for initial effective stress is different, the settlement calculated using Eq. (20) is different from that using Eq. (21). When the same method for determining the self-weight stress is used, the dissipation rate of excess pore water pressure under the large strain assumption is faster than that under the small strain assumption. However, the final settlement of natural structured clays is the same under different geometric assumptions.

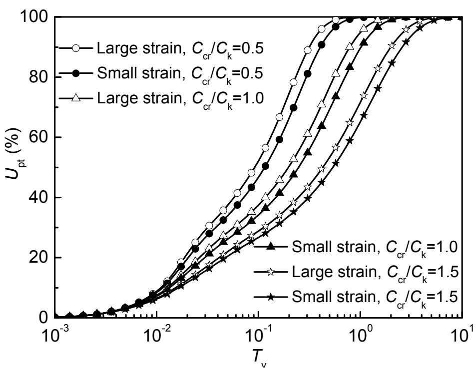

Fig. 14 Influence of C_{cr}/C_k on the average degree of consolidation

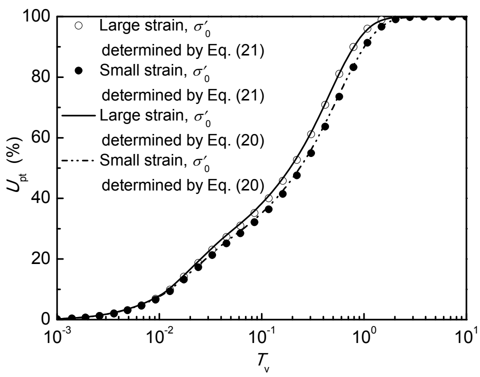

Fig. 15 Influence of initial effective stress on the average degree of consolidation

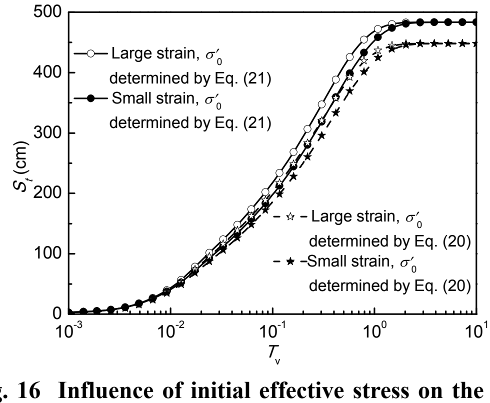

Fig. 16 Influence of initial effective stress on the settlement in the soil layer

6.5 Influences of yield stress on consolidation behavior

For structured clays, when the effective stress is smaller than the yield stress, the compressibility is small. Under this case, the strain of structural soil should be small. As shown in Figs. 17 and 18, when k_2=300 kPa (i.e. the yield stress is large relative to the ultimate stress), the maximal strain of the clay layer is less than 10%, and the differences in the dissipation rate of excess pore water pressure and the settlement of the clay layer between the large strain and small strain assumptions are expected to be negligible. Differences in consolidation behavior due to geometrical assumptions become more significant with the decrease of k_2. The case of k_1=1.03 and k_2=0 means that the yield stress is nearly equal to the initial effective stress, and the structural characteristic of natural clay is not considered in the analysis. Under this condition, the maximal strain of the clay layer exceeds 30%, and the difference in consolidation behavior between the large strain and small strain assumptions is significant, such that the large strain assumption must be considered in the calculation of consolidation.

Fig. 19 shows the difference of settlement distribution at different depths among different yield stresses. When the yield stresses are the same, the difference of settlement at the ground between large strain and small strain is greater than that at other depths. At the same depth, the difference of settlement among different yield stresses increases with a decrease in the yield stress. When the consolidation is fully implemented (e.g. T_v=10), the settlements at

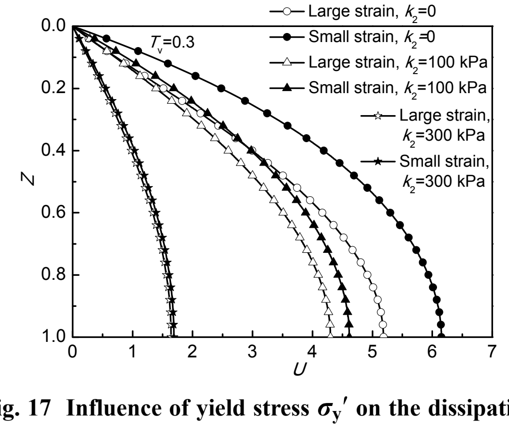

Fig. 17 Influence of yield stress σ_y' on the dissipation of excess pore water pressure

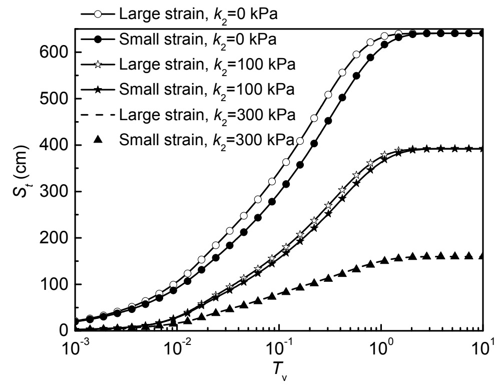

Fig. 18 Influence of yield stress σ_y' on the settlement of the soil layer

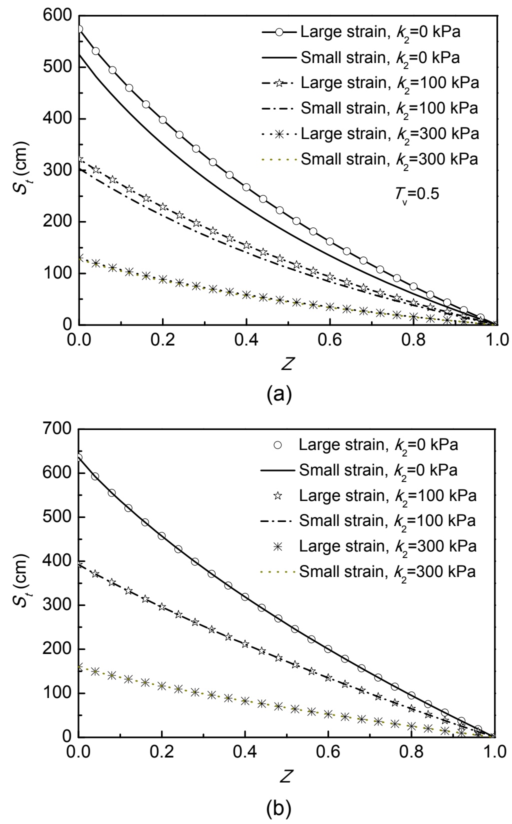

Fig. 19 Influences of yield stress on the settlement at different depths of soil layer

(a) T_v=0.5; (b) T_v=10

different depths between large strain and small strain are the same, and the final settlement of natural structured clays is not influenced by the use of large strain and small strain assumptions when other parameters remain constant.

7 Conclusions

This investigation proposes a novel 1D consolidation model which can take into account the correlation between the mobilized effective stress and the yield stress for structured marine soft clays under the large strain assumption. The dissipation rate of excess pore water pressure of structured clays under the large strain assumption is expected to be faster than that under the small strain assumption, and the difference in consolidation behavior between the two assumptions increases with the strain level of natural structured clays. If the strain level in the clay layer is more than 15%, the difference in consolidation behavior between the large and small strain assumptions must be considered. However, the final settlement of natural structured clays under the large strain assumption is the same as that under the small strain assumption when the calculation method for the initial effective stress is the same. The yield stress of natural structured clays has a great influence on the dissipation rate of excess pore water pressure. The greater the yield stress, the smaller the strain in the clay layer, and the smaller the difference in the dissipation rate of excess pore water pressure between the large and small strain assumptions. Under the same geometric assumption, the method for determining the initial effective stress has no influence on the dissipation rate of excess pore water pressure, whereas the settlement of the natural structured clay layer is different. If the method for determining the initial effective stress is the same, the dissipation rate of excess pore water pressure under the large strain assumption is faster than that under the small strain assumption, whereas the final settlement of natural structured clays is the same.

Contributors

Wen-bing WU designed the research. Chuan-xun LI and Jin-yang XIAO processed the corresponding theory and calculation, and wrote the first draft of the manuscript. Guo-xiong MEI, Peng-peng NI, and Chin Jian LEO helped to organize and revise the manuscript. Chuan-xun LI and Wen-bing WU revised and edited the final version.

Conflict of interest

Chuan-xun LI, Jin-yang XIAO, Wen-bing WU, Guo-xiong MEI, Peng-peng NI, and Chin Jian LEO declare that they have no conflict of interest.

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Appendix A

The expressions of α and β in the dimensionless governing equation of Eq. (24) are:

$$\alpha=\begin{cases}\left(\dfrac{1}{Y}\right)^{\frac{C_{\mathrm{cr}}}{C_{\mathrm{k}}}}\left(\dfrac{Y}{S+Q-U}\right)^{\frac{C_{\mathrm{cn}}}{C_{\mathrm{k}}}}, & Y>S+Q-U\geq S,\\ \left(\dfrac{1}{S+Q-U}\right)^{\frac{C_{\mathrm{cr}}}{C_{\mathrm{k}}}}, & S+Q-U\geq Y,\end{cases} \tag{A1}$$

$$\beta=\begin{cases}(S+Q-U)(C_{\mathrm{cr}}/C_{\mathrm{cn}}), & Y>S+Q-U\geq S,\\ S+Q-U, & S+Q-U\geq Y.\end{cases} \tag{A2}$$

The expressions of β_j^k, $\alpha_{j+1/2}^k$, and $\alpha_{j-1/2}^k$ in the difference equation of Eq. (29) are:

$$\beta_j^k=\begin{cases}\lambda_k\dfrac{C_{\mathrm{cr}}}{C_{\mathrm{cn}}}\dfrac{1+e_{0j}}{1+e_1}\left(S_j+Q^k-U_j^k\right), & Y_j>S_j+Q^k-U_j^k\geq S_j,\\ \lambda_k\dfrac{1+e_{0j}}{1+e_1}\left(S_j+Q^k-U_j^k\right), & S_j+Q^k-U_j^k\geq Y_j,\end{cases} \tag{A3}$$

$$\alpha_{j+1/2}^k=\begin{cases}\left(\dfrac{1}{\frac{Y_{j+1}+Y_j}{2}}\right)^{\frac{C_{\mathrm{cr}}}{C_{\mathrm{k}}}}\left(\dfrac{\frac{Y_{j+1}+Y_j}{2}}{\frac{S_{j+1}+S_j}{2}+Q^k-\frac{U_{j+1}^k+U_j^k}{2}}\right)^{\frac{C_{\mathrm{cn}}}{C_{\mathrm{k}}}}\\ \quad\times\left(\dfrac{1+\frac{e_{0(j+1)}+e_{0j}}{2}}{1+\frac{e_{j+1}^k+e_j^k}{2}}\right), & Y_j>S_j+Q^k-U_j^k\geq S_j,\\ \left(\dfrac{1}{\frac{S_{j+1}+S_j}{2}+Q^k-\frac{U_{j+1}^k+U_j^k}{2}}\right)^{\frac{C_{\mathrm{cr}}}{C_{\mathrm{k}}}}\\ \quad\times\left(\dfrac{1+\frac{e_{0(j+1)}+e_{0j}}{2}}{1+\frac{e_{j+1}^k+e_j^k}{2}}\right), & S_j+Q^k-U_j^k\geq Y_j,\end{cases} \tag{A4}$$

$$\alpha_{j-1/2}^k=\begin{cases}\left(\dfrac{1}{\frac{Y_j+Y_{j-1}}{2}}\right)^{\frac{C_{\mathrm{cr}}}{C_{\mathrm{k}}}}\left(\dfrac{\frac{Y_j+Y_{j-1}}{2}}{\frac{S_j+S_{j-1}}{2}+Q^k-\frac{U_j^k+U_{j-1}^k}{2}}\right)^{\frac{C_{\mathrm{cn}}}{C_{\mathrm{k}}}}\\ \quad\times\left(\dfrac{1+\frac{e_{0j}+e_{0(j-1)}}{2}}{1+\frac{e_j^k+e_{j-1}^k}{2}}\right), & Y_j>S_j+Q^k-U_j^k\geq S_j,\\ \left(\dfrac{1}{\frac{S_j+S_{j-1}}{2}+Q^k-\frac{U_j^k+U_{j-1}^k}{2}}\right)^{\frac{C_{\mathrm{cr}}}{C_{\mathrm{k}}}}\left(\dfrac{1+\frac{e_{0j}+e_{0(j-1)}}{2}}{1+\frac{e_j^k+e_{j-1}^k}{2}}\right), & S_j+Q^k-U_j^k\geq Y_j.\end{cases} \tag{A5}$$

中文概要

题　目：海相沉积结构性软土一维大应变固结分析

目　的：天然沉积的结构性软土分布广泛，但能考虑结构性影响的大应变固结理论鲜有报道。本文考虑结构屈服压力随初始有效应力的变化及土体结构性对压缩与渗透特性的影响，建立结构性软土的大应变固结模型。研究结构性及初始有效应力对大应变固结性状的影响，并探讨结构性软土大、

小应变固结性状的差异，以提高软土固结计算的精准度。

创新点： 1. 建立考虑天然沉积软土结构性影响的一维大应变固结模型，且该模型能考虑结构屈服压力随初始有效应力的变化；2. 分析天然结构性软土大、小应变固结性状的差异，为实际工程中的软土固结计算提供理论依据。

方　法： 1. 总结结构性软土对压缩和渗透特性的影响及结构屈服压力与初始有效应力间的关系；2. 通过理论推导，构建考虑结构性影响的软体一维大应变固结模型（公式（15）和（16））；3. 通过对模型进行数值求解，分析软土结构性对大应变固结性状的影响，以及考虑结构性影响的大、小应变固结性状的差异。

结　论： 1. 大应变假定下结构性软土中超静孔压的消散速率要比小应变假定下快，且这种差异随着土层应变增大而增大；当应变值超过 15%时必须采用大应变假定。2. 如果土层的初始有效应力计算方法相同，则大、小应变不同假定下土层的最终沉降值是相同的。3. 相同几何假定下，初始有效应力计算方法对超静孔压消散速率几乎无影响，但对沉降变形影响明显。4. 大、小应变假定下固结性状间的差异随结构屈服压力的增大而减小。

关键词： 结构性软土；大应变固结；结构屈服压力；有限差分法