Development and Verification of a New Simplified Method for Calculating Settlement of a Thick Soil Layer with Nonlinear Compressibility and Creep

by

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ABSTRACT

The variable compressibility is necessary to be considered in settlement calculation of soft soil stratums, especially for thick soil layers. In this study, to calculate the consolidation settlement of a thick soil layer with creep, a new simplified method is presented using Zhu and Yin method (2012) to consider the nonlinear compressibility. The creep settlement during the consolidation stage is calculated, examined, and discussed for the thick soil layer. In the new simplified method, α is an important parameter to approximately consider the couple of consolidation and creep. It is found that the value of α in the new simplified method is a variable and is suggested to take $\alpha = U_z$ as a simplification. A series of cases including different thicknesses of soil stratums, different *OCR* values, different surcharge loadings, and different values of creep parameter are studied to verify the new simplified method by comparing our calculated values with the finite element simulated results. Subsequently, a typical and well-known *Väsby* test fill project with 50 years' measured data is selected as a thick soil layer example to illustrate the feasibility of the new simplified method to be used in practice. Both the new simplified method and finite element modelling are used to obtain the ground settlements. It is found from the comparison that the calculated results from the new simplified method and the FE modelling are in good agreement with the measured data.

***Keywords*:** nonlinear compressibility, consolidation settlement, creep, visco-plastic,

simplified method, thick layer

Introduction

In general, the elastic visco-plastic (EVP) constitutive modelling is the most suitable modelling approach to describe the general stress–strain–time relationship of clayey soils (Bjerrum 1967; Yin and Graham 1996, 1999; Yin *et al.* 2010, 2011; Feng *et al.* 2017). Numerous researchers have attempted to calculate consolidation settlements of soft soil ground by using different EVP models (Qu *et al.* 2010; Yin *et al.* 2008; Yin and Karstunen 2008; Yin *et al.* 2011). However, the numerical methods, such as the finite difference method or the finite element (FE) method, must be adopted to solve a highly nonlinear partial differential consolidation equation system.

The consolidation of clayey soils is directly affected by soil compressibility. In Terzaghi's theory, the stress–strain relationship of soils is linear, resulting that soil compressibility is a constant in the vertical direction. However, the compressibility of soft soils is closely related to the initial and final effective stresses along the depth (Davis and Raymond 1965; Gibson *et al.* 1967; Poskitt 1969; Hawley and Borin 1973; Abbasi *et al.* 2007; Abuel-Naga and Pender 2012; Abuel-Naga *et al.* 2015; Wu *et al.* 2016). Due to the stress history and geological process, the initial effective stress and pre-consolidation pressure normally increase with depth in the field, thus, the soil compressibility varies with depth of the soft soil layer nonlinearly when the ground is subjected to an external loading (Craig 2004). Olson and Ladd (1979) studied the consolidation performance of different nonlinear $e-\log\sigma_z^{'}$ relationships

(e is the void ratio and $\sigma_z^{'}$ is the vertical effective stress). They found that the nonlinear compressibility has an unneglectable influence on the consolidation process of soils. Duncan (1993) pointed out the limitations of the conventional consolidation analysis and stated that the nonuniform strain profile in the field plays an important role in the predicted settlement based on the experiences of San Francisco Bay Island and Kansai International Airport. Subsequently, researchers such as Menéndez *et al.* (2010), Abuel-Naga and Pender (2012), Brandenberg (2016), Wu *et al.* (2016), Pu *et al.* (2018a, 2018b), Li *et al.* (2018), Liu *et al.* (2018) focus on this challenging area and make meaningful achievements. Therefore, the variable compressibility of soft soils is necessary to be considered in settlement calculation of soft soil layers, especially for thick soil layers.

Ladd *et al.* (1977) raised a fundamental question whether creep occurs during the primary consolidation for a thick soil stratum. Many researchers advocated that the creep rate is only related to the current effective stress and strain state (Den Haan 1996; Yin and Graham 1996; Vermeer and Neher 1999; Kim and Leroueil 2001; Nash and Ryde 2001; Grimstad and Degago 2010; Nash 2010; Degago *et al.* 2011). They used Hypothesis B method to consider creep settlement occurred in both "primary" consolidation and "secondary" consolidation. On the contrary, some researchers supported that the creep occurs after the consolidation in both laboratory and field conditions (Mesri and Choi 1985; Mesri and Vardhanabhuti 2006) and used Hypothesis A method for consolidation–creep calculation. Degago *et al.* (2011) reviewed the experimental investigations from previous literatures critically to access the creep during the consolidation phase. He concluded that the measured time-dependent compression of clays has a good agreement with Hypothesis B. In Hypothesis B, a lot of researchers have conducted numerous studies to investigate the coupling of creep and consolidation in one-dimensional (1-D) straining condition. The measured data of soft soils in the field demonstrated that the excess pore water pressure increases up to that higher than the external loading initially, subsequently, it starts to decrease (Becker *et al.* 1985; Kabbaj *et al.* 1988). Yin *et al.* (1994) first incorporated an EVP constitutive model into the consolidation equations and successfully simulated the response of excess pore water pressure, which is the phenomenon observed in the field. Subsequently, Yin and Graham (1996) simulated the 1-D consolidation and excess pore water pressure in clayey soils with the EVP constitutive model by using the finite difference method. Yin and Zhu (1999) found that the increase of excess pore water pressure in clays under 1-D straining condition is caused by creep compression. Stolle *et al.* (1999) presented a nonlinear consolidation model accounting for the "secondary" compression and identified that the normalized excess pressure of the thicker layer is larger than 1. Yuan and Whittle (2013) examined the performance of 1-D consolidation using the soft soil creep (SSC) model and isotache constitutive model proposed by Imai (1995). They explained that increase of excess pore water pressure in the consolidation stage is due to the inconsistency between the total strain rate and visco-plastic strain rate. Yuan and Whittle

(2018) proposed a new formulation for elasto-viscoplastic model by introducing an internal state variable to describe the time-dependent compression of clayey soils. The increase of excess pore water pressure in the consolidation phase is also modelled for the high value of new state variable, (R_a). Therefore, the creep compression is not simply displayed in the total settlement, it is also reserved in the excess pore water pressure and expressed as the total settlement gradually in the coupling of the creep and consolidation.

Recently, a new simplified Hypothesis B method was proposed and validated for the calculation of consolidation settlement of soft soil layers following the EVP constitutive relationship and "equivalent time" concept (Yin and Feng 2017). In this new simplified method, U_z is defined as the *average degree of consolidation*, indicating the dissipation of the excess pore water pressure of the total soil layer. U_z is calculated using the empirical equations of Terzaghi's theory for one-layered soil and Zhu and Yin method (1999) for double-layered soil. The *coefficient of volume compressibility* m_v is taken as a constant (the average value of the sublayers' *volume compressibility*) in the calculation. The creep settlement is calculated by introducing a parameter α with a constant value in the calculation to approximately consider the coupling of the creep and consolidation. However, there is a deficiency in calculating settlement for the thick soil layers with the constant value of α. In this study, the new simplified method is developed by considering the nonlinear compressibility with depth to obtain the U_z for a thick soil layer accurately, and the physical meaning of the parameter α is discussed. Examples of different thicknesses of soil stratums, different surcharge loadings, different stress–strain states (over-consolidation ratio, *OCR*=1, 1.5, and 2), and different values of creep parameter are calculated and verified by the results from FE analysis. Finally, this new simplified method is used in the project of *Väsby* test embankment and the calculated result is compared with the 50 years' measured data in the field for the application.

Equations of a New Simplified Method for Calculating Consolidation Settlement of a Thick Soil Layer with Nonlinear Compressibility and Creep

In the new simplified method, the total settlement S_{totalB} is the summation of consolidation settlement $S_{consolidation}$ and creep settlement S_{creep}. A general equation of this new simplified method for 1-D consolidation settlement calculation of clayey soils with creep can be expressed as

$$S_{totalB} = S_{consolidation} + S_{creep} \tag{1}$$

This new simplified method is developed from the ideas of the EVP constitutive model based on the "equivalent time" concept, which was proposed by Yin and Graham (1989, 1994). The subscript of "*totalB*" is utilized in Eq. (1) because the creep is considered in the consolidation stage (Hypothesis B). In this equation, consolidation settlement $S_{consolidation}$ is related to the nonlinear stress–strain relationship and the U_z of a soft soil layer. Creep settlement S_{creep} is expressed by the creep time line at a certain time, which will be presented in following parts.

Consolidation Settlement with Respect to Nonlinear Compressibility of Soil Layers

The *coefficient of volume compressibility* m_v is usually adopted to describe the volume change per unit volume with respect to the increase of the effective stress, and it is a parameter used to determine the *coefficient of consolidation*, $c_v = \frac{k_z}{\gamma_w m_v}$ (Craig 2004). Due to the variation of the initial effective vertical stress with depth, the soil layer is divided into sublayer soils, each of which has a suitable thickness (*e.g.* 0.5 m). With the help of Figure 1(a), the initial stress–strain state is assumed at Point 1, and the consolidation settlement is related to soil compression on the instant time line when the final effective stress is at Point 2 (from Point 1 to Point 2) or the reference time line if the final effective stress is at Point 4 (from Point 1 to Point 4). The total consolidation settlement is the sum of the compression of sublayers. The compression of each sublayer is calculated based on the stress–strain state with respect to pre-consolidation pressure at the center of each sublayer in the new simplified method:

(a) From Point 1 to Point 2

$$S_{f,i} = \varepsilon_{f,i} H_i = \frac{\kappa}{V} \ln\left(\frac{\sigma'_{z0,i} + \Delta\sigma'_{z,i}}{\sigma'_{z0,i}}\right) H_i \qquad for\ \Delta\sigma'_{z,i} \le \sigma'_{zp,i} - \sigma'_{z0,i} \quad (2a)$$

(b) From Point 1 to Point 4

$$S_{f,i} = \varepsilon_{f,i} H_i = \left\{\frac{\kappa}{V} \ln\left(\frac{\sigma'_{zp,i}}{\sigma'_{z0,i}}\right) + \frac{\lambda}{V} \ln\left(\frac{\sigma'_{z0,i} + \Delta\sigma'_{z,i}}{\sigma'_{zp,i}}\right)\right\} H_i \qquad for\ \Delta\sigma'_{z,i} > \sigma'_{zp,i} - \sigma'_{z0,i} \quad (2b)$$

$$m_{v,i} = \frac{S_{f,i}}{H_i \Delta\sigma'_{z,i}} = \frac{\varepsilon_{f,i}}{\Delta\sigma'_{z,i}} \tag{3}$$

$$S_f = \sum_{i=1}^{n} S_{f,i} \tag{4}$$

$$S_{consolidation} = U_z S_f \tag{5}$$

where H_i is the thickness of a sublayer; $m_{v,i}$ is the coefficient of volume compressibility of the sublayer; V is the specific volume, $V = 1 + e_o$; κ and λ are defined as the compression parameters in the EVP constitutive model proposed by Yin and Graham (1989. 1994); $\sigma'_{z0,i}$, $\Delta\sigma'_{z,i}$, and $\sigma'_{zp,i}$ are initial effective vertical stress, effective vertical stress increment, and pre-consolidation pressure for each sublayer. Yin (2015) explained the fundamental concepts in the EVP constitutive model and regarded that this model is an extension of Maxwell's linear rheological model with the consideration of the nonlinear behavior of clayey soils, as illustrated in Figure 1(b). It is noted that the value of $m_{v,i}$ varies along the depth because there is an nonlinear relationship between strain compression and initial effective vertical stress $\sigma'_{z0,i}$, effective vertical stress increment $\Delta\sigma'_{z,i}$, pre-consolidation pressure $\sigma'_{zp,i}$ of each sublayer.

In fact, the variable compressibility of each sublayer was noticed by Yin and Feng (2017). They used the values of *coefficient of volume compressibility* of all sublayers to calculate an average as a constant for the whole soil layer to obtain the consolidation settlement. However, influence of variable compressibility on the calculation of the U_z, is not fully considered in Yin and Feng (2017). Zhu and Yin (1999) presented an analytical solution for the consolidation of a soil layer with a constant value of vertical consolidation coefficient under the depth-dependent ramp load. Conversely, this solution can be used to obtain the U_z for a soil layer considering the depth-dependent compressibility under a constant load. Subsequently, Zhu and Yin (2012) analyzed a general variation of compressibility and permeability along depth for the soft marine deposits and obtained a mathematical solution for the U_z of a thick clay layer. In this study, the permeability along the depth is regarded as a constant, the compressibility variation in a thick soil layer is considered by adopting this solution by Zhu and Yin (2012) to calculate the U_z of this soil layer

$$m_v(z) = m_{v0}\left(1+\zeta\frac{z}{H}\right)^q \tag{6}$$

where m_{v0} is the volume compressibility value of top of this thick soil layer; ζ and q are two fitting parameters for the nonlinear depth-dependent compressibility of soil layer. The U_z can be calculated as

(a) For one-drainage boundary condition

$$U(T,\zeta) = 1 - \frac{4(1+q)}{(2+q)\left[(1+\zeta)^{1+q}-1\right]}\sum_{m=1,2..}^{\infty}\frac{4}{\eta_m^2\left\{\left[\pi b\eta_m Z_v^m(-\eta_m q)\right]^2-4\right\}}\exp\left(-\frac{(\pi\eta_m)^2}{4\eta_1^2}T\right) \tag{7}$$

where η_m is *m-th* positive root of $\frac{1}{2+q}Z_v(\eta b)+\eta b Z_v^{'}(\eta b)=0$; b is the parameter considering nonlinear compressibility: $b=(1+\zeta)^{1+\frac{q}{2}}$; T is time factor for consolidation, $T=\frac{c_{v0}(2+q)^2\zeta^2\eta_1^2 t}{(\pi H)^2}$.

(b) For two-drainage boundary condition

$$U(T,\zeta)=\begin{cases}1-\dfrac{4(1+q)}{(2+q)\left[(1+\zeta)^{1+q}-1\right]}\displaystyle\sum_{m=1,2..}^{\infty}\dfrac{\left[2+\pi b^{1-v}\eta_m Z_{v-1}^{m}(\eta_m b)\right]^2}{\eta_m^2\left\{\left[\pi b\eta_m Z_v^m(-\eta_m b)\right]^2-4\right\}}\exp\left(-\dfrac{(\pi\eta_m)^2}{4\eta_1^2}T\right) & v=\dfrac{1}{2+q}\\ 1-\dfrac{4(1+q)}{(2+q)\left[(1+\zeta)^{1+q}-1\right]}\displaystyle\sum_{m=1,2..}^{\infty}\dfrac{\left[2-\pi b^{1+v}\eta_m Z_{v+1}^{m}(\eta_m b)\right]^2}{\eta_m^2\left\{\left[\pi b\eta_m Z_v^m(-\eta_m b)\right]^2-4\right\}}\exp\left(-\dfrac{(\pi\eta_m)^2}{4\eta_1^2}T\right) & v=\dfrac{-1}{2+q}\end{cases} \tag{8}$$

where η_m is *m-th* positive root of $Z_v^m(x)=Y_v(\eta_m)J_v(x)-J_v(\eta_m)Y_v(x)=0$.

Calculation of Creep Settlement

The creep settlement is calculated by using the following equation:

$$S_{creep}=\alpha S_{creep,f}+(1-\alpha)S_{creep,d} \tag{9}$$

where $S_{creep,f}$ is the total creep settlement with respect to final effective stress ignoring the coupling of the excess pore water pressure; $S_{creep,d}$ is the delayed creep settlement due to the coupling of the excess pore water pressure; The distinguished difference of $S_{creep,f}$ and $S_{creep,d}$ is the beginning of accounting the creep: the time for $S_{creep,f}$ is the default in this study, $t_o=1day$, while that for $S_{creep,d}$ is after the consolidation of soil layer, $t_o=t_{EOP}$; α is a parameter for calculating the creep settlement with the value in the range of 0~1 to account for the couple of consolidation and creep. As shown in Figure 1(a), the creep is vividly expressed by equivalent time lines, which is directly related to creep coefficients ψ/V and t_o. The creep strain rate on equivalent time line is independent on the stress path, which combines the over-consolidation state and normal consolidation state, expressed as

$$S_{creep,f}=\sum_{i=1}^{n}S_{creep,f,i}=\sum_{i=1}^{n}\varepsilon_{creep,f,i}H_i \tag{10}$$

$$\varepsilon_{creep,f,i}=\psi/V\ln\left(\frac{t_o+t_{e,i}}{t_o+\Delta t_{e,i}}\right)\quad for\ \Delta\sigma'_{z,i}\le\sigma'_{zp,i}-\sigma'_{z0,i} \tag{11a}$$

$$\varepsilon_{creep,f,i} = \psi / V \ln\left(\frac{t_o + t_{e,i}}{t_o}\right) \qquad for\ \Delta\sigma'_{z,i} > \sigma'_{zp,i} - \sigma'_{z0,i} \tag{11b}$$

where $\Delta t_{e,i}$ and $t_{e,i}$ are determined from the following equations

$$\Delta t_{e,i} = t_o \times \exp\left((\varepsilon_{f,i} - \varepsilon_{zp,i})\frac{V}{\psi}\right)\left(\frac{\sigma'_{z0,i} + \Delta\sigma'_{z,i}}{\sigma'_{zp,i}}\right)^{-\frac{\lambda}{\psi}} - t_o$$

$$t_{e,i} = t - t_o + \Delta t_{e,i}$$

The equivalent time $t_{e,i}$ illustrates that the creep strain rate is dependent on the effective stress-strain state and the preconsolidation pressure of the soil skeleton. For the calculation of delayed creep settlement $S_{creep,d}$ the end of consolidation time t_{EOP} is utilized to replace t_o in Eq. (11). expressed as:

expressed as

$$S_{creep,d} = \sum_{i=1}^{n} \varepsilon_{creep,d,i} H_i \tag{12}$$

$$\varepsilon_{creep,d,i} = \psi / V \ln\left(\frac{t_{EOP} + t_{e,i}}{t_{EOP} + \Delta t_{e,i}}\right) \quad for\ \Delta\sigma'_{z,i} \le \sigma'_{zp,i} - \sigma'_{z0,i} \tag{13a}$$

$$\varepsilon_{creep,f,i} = \psi / V \ln\left(\frac{t_{EOP} + t_{e,i}}{t_{EOP}}\right) \qquad for\ \Delta\sigma'_{z,i} > \sigma'_{zp,i} - \sigma'_{z0,i} \tag{13b}$$

It is usually taken the time when $U_z = 98\%$ as t_{EOP} .

In Eq. (9), α is an important parameter to describe the creep settlement expressed in the consolidation stage in the new simplified method. The creep effect can be directly investigated by comparing the FE simulated results with different values of creep parameter (adopting the SSC model in the Plaxis software). In the FE modelling, two values of creep parameter μ^* listed in Table 1 are considered to interpret the creep effect in the consolidation stage. As illustrated in Figure 2(a), the top and middle points are pre-set in the

FE simulation for monitoring the ground settlement and excess pore water pressure of soil layers (one-drainage condition). Details of the FE modelling can be referred in Section 3.1. The responses of excess pore water pressure and ground settlement of the soil layer with 8m are plotted in Figure 2(b), and those of the soil layer with 16m are shown in Figure 2(c). The shadow area is the different performances of the FE simulated results from two different creep parameter values (other parameter values are remained the same), therefore, the shadow area is only influenced by the creep. It is seen that the creep is reserved in the excess pore water pressure initially, subsequently, it is displayed as the surface settlement gradually. Therefore, the parameter α is a variable value related to the U_z rather than a constant. In this study, we take $\alpha = U_z$ as a simplification.

Calculation Procedures for New Simplified Method

For the calculation of thick soil layer, it is necessary to divide the soil layer into sublayers with thickness less than 0.5 m because the initial effective vertical stress increases with the depth (Yin and Feng 2017). Afterwards, the final effective vertical stress can be obtained $\sigma'_{zf,i} = \sigma'_{zo,i} + \Delta\sigma'_{z,i}$ and it will be compared with the preconsolidation pressure of each sublayer to determine the soil stress state, as shown in Figure 3(a). Then, the total settlement of new simplified method can be calculated based on the flow diagram, as presented in Figure 3(b). It can be noted that it is very important to correctly calculate the value of U_z for both consolidation settlement and creep settlement.

Hypothesis A Method for Consolidation Settlement of Soil Layers

A simple method of Hypothesis A is utilized in this study for the calculation of the total consolidation settlement S_{totalA} in the field:

$$S_{totalA} = S_{"primary"} + S_{"secondary"} = \begin{cases} U_z S_f & for\, t \le t_{EOP} \\ U_z S_f + \dfrac{C_{\alpha e}}{V}\log\left(\dfrac{t}{t_{EOP}}\right) & for\, t > t_{EOP} \end{cases} \tag{14}$$

where $S_{"primary"}$ is the "primary" consolidation settlement at time *t* and it equals to $S_{consolidation}$ in Eq. (5); t_{EOP} represents the end of "primary" consolidation in the field, and it is usually taken the time for $U_z = 98\%$; $C_{\alpha e}/V$ is the coefficient of "secondary" consolidation. For the over-consolidated soils, a smaller value of $C_{\alpha e}/V$ will be taken in practice (Feng and Yin 2018). In Hypothesis A, "secondary" consolidation occurs after the "primary" consolidation, therefore, there is no need to consider the coupling of the consolidation and creep.

Verification of the New Simplified Method by Comparing Calculated Values with FEM

In this part, we select Hong Kong Marine Deposits (HKMD) as a typical example of a soft soil layer to analyze the consolidation settlement of soils with creep. The typical parameter values for the new simplified method and FE modelling are listed in Table 2. Cases including different thickness values (8m, 16m, and 32m), different applied loadings (20 kPa, 50 kPa, and 100 kPa), different OCR values (*OCR*=1, 1.5, and 2), and different creep parameters (ψ / V =0.0076, 0.0038, and 0.00076) are considered in this study to illustrate the influence of variable compressibility and creep of thick soil layers under different stress–strain states. The

FE software (Plaxis 2015 version) was utilized to analyze the coupling of the consolidation and creep by adopting the SSC constitutive model. The coupling performance of consolidation and creep in Plaxis (2015 version) has been verified and reported by Neher *et al.* 2001, Yin and Feng (2017). The simulated result from FE software is regarded as the rigorous coupled results of consolidation and creep, and it is utilized to evaluate the performance of the new simplified method and Hypothesis A method.

Two parameters in terms of *relative difference of settlement* $\Delta S_{totalB,t}$ and *relative error* $\xi_{totalB,t}$ are defined to assess the performance of the new simplified method:

$$\Delta S_{totalB,t} = S_{totalB,t} - S_{FE,t} \tag{15}$$

$$\xi_{totalB,t} = \frac{\left|S_{totalB,t} - S_{FE,t}\right|}{S_{FE,t}} \times 100\% \tag{16}$$

where $S_{FE,t}$ is the simulated settlement from FE program at a certain time, $S_{totalB,t}$ is the calculated settlement from the new simplified method. Similar parameters of $\Delta S_{totalA,t}$ and $\xi_{totalA,t}$ are defined to examine the accuracy of Hypothesis A method.

Case Description and Finite Element Simulation

Different cases of thick soil layers of HKMD were simulated by the FE software, as mentioned above. In the FE modelling, the *plane strain* model type was pre-set, *15*-Noded element and *fine mesh* were selected to reduce the discretization errors and obtain accurate results (Yin and Feng 2017). The top of soft soil layer is seabed and normally filled by sand materials, therefore, it was set as a drained condition in FE modelling. The bottom was regarded as an impermeable condition (*half-closed* layer condition, Craig 2004). The displacement boundary of the bottom was fixed in both horizontal and vertical directions, and two side boundaries were only fixed in the horizontal direction. The default K_0-condition, which has been proved to be suitable for the normally consolidated soil (Neher *et al.* 2001; Fatahi, *et al.* 2013), was utilized to generate the initial stress state in this study. In the FE modelling, the SSC model was adopted, and parameter values of the thick soil layer are listed in Table 2 (Feng and Yin 2017). The groundwater table was above the top of the simulated soil layer. In the FE modelling, the surcharge loading (20 kPa, 50 kPa, or 100 kPa) was applied instantly and remained 10^6 days in the consolidation stage to make sure the dissipation of excess pore water pressure was totally completed for all cases of the thick soil layers.

Calculation Procedures of New Simplified Method and Hypothesis A Method

In the new simplified method and Hypothesis A method, the calculation procedures are very similar to that presented by Yin and Feng (2017) and Feng and Yin (2017). Firstly, the soil layer is divided into sublayers with 0.5 m and initial/final effective stress of each sublayer is determined based on the soil unit weight and the applied loading. It should be noted that the compressibility variation in different cases is described by using Eq. (6) rather than the average value of the m_v. The Solver in Microsoft Excel was used to determine the values of q and ζ. All the fitted values of m_{v0}, q and ζ for all the cases are listed in Table 3.

Taking the example of 8 m soil layer, the curve is fitted with calculated compressibility values of sublayer with 0.5 m, as shown in Figure 4.

To obtain the *average degree of consolidation* U_z, Eq. (7) will be utilized and calculated. It should be noted that the time factor is normalized including the eigenvalue η_1 and parameters (q and ζ) related to the depth-dependent compressibility. The eigenvalue η_1 can be referred in the table by Zhu and Yin (2012). In this study, Eq. (7) is implemented into a MATLAB program to calculate the U_z accurately.

Verification and Discussion of the New Simplified Method from FE Modelling

The calculation results using the new simplified method, Hypothesis A method are compared with the FE simulated results for different cases mentioned above. The certain time points for *t*=10000-*th* day, 100000-*th* day, and 1000000-*th* day are set as the reference time to examine to illustrate the accuracy of the new simplified method and Hypothesis A method.

(a)Thickness Effect of Soil Stratum

Three different soil layers (8m, 16m, and 32m) are calculated in this part. The calculated results from the new simplified method and Hypothesis A method are compared with the results of FE modelling in Figure 5. The results of the new simplified method agree well with the FE simulations for three different thicknesses of soil stratums, which indicates that new simplified method using Zhu and Yin method (2012) could properly consider the nonlinear compressibility effect for the thick soil layers. For the Hypothesis A method, the consolidation settlement is gradually underestimated with the consolidation time when comparing with the FE simulation results. As listed in Table 4, it is found that the *relative errors* of the Hypothesis A method are in the range of 22.9% ~ 57.5% with under-estimation. The *relative difference of settlement* is as large as 2.9 m for 32 m soil layer. Comparatively, the *relative errors* of the new simplified method range from 0.3% to 12.2% and *the relative difference of settlement* is within 0.2 m, which are acceptable in the geotechnical designs.

(b) Effect of Compressibility Variation

The stress–strain relationship of the soft soil is nonlinear, which is closely related to the initial stress state and the surcharge loading, the compressibility of soft soils along depth is variable when a thick soil layer is subjected to a surcharge loading (Craig 2004). The compressibility variation may be also related to the nonlinear relationship between permeability and void ratio due to the surcharge loading (Tavenas *et al.* 1983; Deng *et al.* 2011; Zagorščak *et al.* 2017). It is recommended that the variation of soil permeability needs also be considered when the soil layer is subjected to a large external load. In this part, three values of surcharge loading (20 kPa, 50 kPa, and 100 kPa) are considered and calculated for the settlement of 16m soil layer (*OCR*=1). It is found that the surcharge loading mainly influences the value of m_{v0} in Zhu and Yin method (2012), as listed in Table 3. As expected, the curve of the new simplified method agrees well with the FE simulations, as shown in Figure 6, the *relative error* is within 12% for this new simplified method. The large underestimation of the Hypothesis A method can not be ignored for the prediction of the long-term period: the *relative errors* of Hypothesis A method are in the range of 6.0% ~ 41.9%, and the largest underestimation is 1.4 m for the soil layer (16m with *OCR*=1) subjected to 50 kPa.

(c) OCR Effect

Figure 7 shows the comparison of the settlement and time (log–scale) from the new simplified method, Hypothesis A method, and the FE simulations for 16m soil layer with *OCR*=1, 1.5, and 2, respectively. The surcharge loading is 20 kPa. The sublayers below 8m (for *OCR*=1.5) or below 4m (for *OCR*=2) for 16m soil layer are at over-consolidated state. Curves of the new simplified method using Zhu and Yin method (2012) nearly overlap the results of the FE simulation, which indicates that the creep settlement in the new simplified method is also correctly calculated based on the "equivalent time" concept (Bjerrum 1967; Yin and Graham 1989, 1994, 1996). Comparing with the FE results, the Hypothesis A method obviously underestimates the total settlement for all three *OCR* values, which is consistent with the previous findings (Yin and Graham 1996; Yin and Feng 2017). It is found that the *relative errors* of Hypothesis A method are in the range of 37.9% ~ 59.5% with underestimation of total settlements, whereas those of the new simplified method overestimate the total settlement from 0.03 m to 0.27 m, resulting in the *relative errors* of 0.1% ~25.9%.

(d) Creep Effect

The relationships of settlement *versus* time (log–scale) from the new simplified method, Hypothesis A method are compared with results from the FE simulations for 16m thick soil layer (*OCR*=1) with three values of creep parameter, as shown in Figure 8. It can be observed that there is very small difference before 100 days in FE simulations with different values of creep parameter because the creep effect is reserved into the increase of excess pore water pressure (as analyzed in Section of 2.2). Afterwards, the settlement difference increases in the FE simulations gradually for three different values of creep parameter. In other words, the creep effects on the settlement of the soft soil layers express after 1000-*th* day in this simulated soft soil layer (16m and *OCR*=1). In the new simplified method, the expression of creep settlement is related to the U_z. It is found from Figure 7 that the new simplified method can capture the creep settlement during the consolidation stage properly. Conversely, it is supposed that the creep settlement does not occur in the Hypothesis A method, therefore, there is no difference of "primary consolidation" when adopting three different values of creep parameter, which is not reasonable.

Application and Verification of the New Simplified Method using Measured Data from *Väsby* Test Fill Site

Site Description of Väsby Test Fill and FE Modelling

As reported by Larsson and Mattsson (2003), the *Väsby* test fill site located nearby the village Lilla Mellösa of Upplands Väsby, 30 km north of Stockholm on the east coast of Sweden, it was constructed in October 1947. The dimensions of this site are 30 m × 30 m, and the fill height is 2.5 m with a slope of 1/1.5 (vertical direction /horizontal direction). Underneath the test fill, there are post glacial clays and lower glacial clays with a total thickness of 14 m. A very thin layer of grey sand covers on the bedrock surface (Chang 1969), which can be regarded as the drained boundary. Detailed information of the *Väsby* test fill can be found in Larsson and Mattsson (2003). Chang (1981) conducted a series of laboratory experiments to determine the soil basic properties, such as Atterberg limit, organic content, sensitivity, soil compressibility, and water content. The main laboratory test results are shown in Figure 9. It is seen that the soil unit weight increases with depth from 12.75 kN/m^3 to 17.66 kN/m^3, the water content ranges from 130% to 70% along the depth of soil layer. The value of the plastic index in the top layer is around 85%, while that in the bottom layer is 34%. It should be noted that there is a crust layer with a thickness of 1.5 m~2 m, and the pre-consolidation pressure is 51 kPa (corresponding OCR=6), as shown in Figure 9(e). According to Larsson and Mattsson (2003), the density of fill materials is 17 kN/m^3, leading to the applied stress of 40.6 kPa based on the height of fill materials. However, the fill materials, which are initially above the groundwater, sank into the groundwater in the consolidation stage due to the large amount of settlement. This would reduce the actual load on the ground. The load reduced from 40.6 kPa in 1947 to 27.5 kPa in 1968, and the load decreased continuously to 23.5 kPa in 2003 (as shown in Figure 10).

As illustrated in Figure 9(a), FE geometry by using the Plaxis software (2015 version) was modelled based on the information mentioned above. The *plane strain* model type and fine mesh were set. The fill layer, crust, and soft soil layers were simulated in the FE model by using Mohr-Coulomb (MC) model and SSC model, respectively. 15-Noded elements were selected and the fill material was activated in the construction period to accurate simulate the total settlement. The FE modelling could simulate the applied stress change due to fill materials sinking into ground water. The parameters of each soil layer are listed in Table 5, which are consistent with the data reported by Le (2015). The top and bottom boundary conditions of soft soil layer were set as drained according to the geological condition. Thus, drainage type of fill material was chosen as "drained" in the FE modelling. The construction period was also 25 days to simulate the real construction process. The total duration lasts for 20745 days to make a comparison with measured data in the field.

Calculation and Comparison of Results from the New Simplified Method, Hypothesis A method, FE simulations and Measured Settlement

For the calculation of the new simplified method, the procedures are not repeated here. It is noted that the loading is taken as 32.05 kPa (the average value of 40.6 kPa and 23.5 kPa) in the calculation of new simplified method. The nonlinear compressibility of soil layer is fitted by Eq. (6), as shown in Figure 11. In *Väsby* test fill, the boundary condition is double drainage, thus, Eq. (8) is utilized to calculate the U_z. Results of the new simplified method and Hypothesis A method for the thick soil layers are examined by the actual measured settlement for 50 years in this section.

As shown in Figure 12, the curve of the new simplified method using Zhu and Yin method is very close to the actual measured data. The values of *relative error* and *relative difference of settlement* are listed in Table 6. It is observed that *relative error* is 16.4% on 470-*th* day for the new simplified method, which is mainly reduced by the small value of measured data (0.278m). In fact, the *relative difference of settlement* is only 0.0456 m compared with the measured data. On 20744-*th* day, there is 0.127 m overestimation of the predicted settlement by the new simplified method. Comparing with the measured settlement, the *relative error* is 6.2% for the new simplified method, whereas, the *relative error* is 59.1% for the Hypothesis A method. The main reasons may consist of the average value of the applied loadings taken in the new simplified method, without considering the permeability decrease of the soft soil layer due to the large compression in the field (Larsson and Mattsson 2003). In the FE simulations, the overestimation of settlement can be reduced by properly considering the decrease of applied loading. Again, Hypothesis A method obviously underestimates the total settlement, especially for the long-term performance.

Conclusions

In this study, A new simplified method considering both the nonlinear compressibility and creep has been proposed and verified for the consolidation settlement of a thick soil layer. In this method, Zhu and Yin method (2012) is utilized to calculate the U_z considering the nonlinear compressibility of a thick soil layer. Fully coupled FE consolidation analysis of thick soil layers and a test fill with measured data was carried out and used to verify this new simplified method. The Hypothesis A method is also used to calculate settlements which are used for comparison. Based on the results and discussions, main conclusions are drawn as follows:

(a) Zhu and Yin method (2012) is suitable to calculate the *average degree of consolidation* U_z of a thick soil layer with the nonlinear compressibility along the depth.

(b) The coupling of the creep and consolidation is necessary to be considered for a thick soil layer. In this study, the value of α in the new simplified method is a variable, taken as $\alpha = U_z$ (in the range of 0~1) rather than a constant.

(c) Cases with different thicknesses of soil layers, different *OCR* values, different surcharge loadings, and different values of creep parameter are analyzed using both a fully coupled FE model and the new simplified method using Zhu and Yin method (2012) for U_z. It is found that the calculated curves using the new simplified method with Zhu and Yin method for U_z are in a good agreement with curves from FE simulations for all cases.

(d) Lastly, the new simplified method and the FE modelling are used to determine the ground settlements in *Väsby* test fill with 50 years' measured data. The comparison shows that the calculated settlements using the new simplified method are in good agreement and also are consistent with the measured data from this test fill site.

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Table 1. Parameter values used in finite element (FE) analysis

γ_{soil} (kN/m^3)	OCR	κ^*	λ^*	μ^*	k_z (m/day)	c' (kPa)	ϕ' (°)	
15	1	0.0217	0.174	0.0076 or 0.000076	1.9×10^{-4}	0.1	30	

Note: details of parameters can be referred in *Plaxis* manual (2015)

Table 2. Parameter values used in the new simplified method and FE modelling

(a) Values of parameters in the new simplified method

The new simplified method							
γ_{soil} (kN/m^3)	OCR	κ/V	λ/V	ψ/V	k_z (m/day)	t_0 (day)	
15	1, 1.5, or 2	0.0108	0.174	0.0076	1.9×10^{-4}	1	
FE modelling							
γ_{soil} (kN/m^3)	OCR	κ^*	λ^*	μ^*	k_y (m/day)	c' (kPa)	ϕ' (°)
15	1, 1.5, or 2	0.0217	0.174	0.0076	1.9×10^{-4}	0.1	30

Table 3. Summary of fitted values of nonlinear compressibility using Eq.(6) for thick soil layers

Case	m_{v0}	ζ	q
8 m	0.024344	0.5631	-5
16 m		0.6631	
32 m		0.8631	

<table>
<tr><td>20 kPa</td><td>0.024344</td><td rowspan="3">0.6631</td><td rowspan="6"></td></tr>
<tr><td>50 kPa</td><td>0.012797</td></tr>
<tr><td>100 kPa</td><td>0.007582</td></tr>
<tr><td>OCR= 1</td><td rowspan="3">0.024344</td><td>0.6631</td></tr>
<tr><td>OCR= 1.5</td><td>1.2631</td></tr>
<tr><td>OCR= 2</td><td>2.1631</td></tr>
</table>

Table 4. Relative error *and* relative difference of settlement *values of the new simplified method using Zhu and Yin method and Hypothesis A method*

Case	Time (day)	FE simulation (m)	New simplified method			Hypothesis A method		
			$S_{totalB,t}$ (m)	$\Delta S_{totalB,t}$ (m)	$\xi_{totalB,t}$ (%)	$S_{totalA,t}$ (m)	$\Delta S_{totalA,t}$ (m)	$\xi_{totalA,t}$ (%)
8m	10000	1.3557	1.5048	0.1492	11.0	1.0446	-0.3110	22.9
	100000	1.8547	1.9711	0.1164	6.3	1.3316	-0.5231	28.2
	1000000	2.0152	2.1111	0.0959	4.8	1.4716	-0.5436	27.0
16m* (OCR=1; 20 kPa)	10000	1.4593	1.4716	0.0123	0.8	0.8809	-0.5784	39.6
	100000	2.8544	3.0397	0.1854	6.5	1.6584	-1.1960	41.9
	1000000	3.2486	3.3503	0.1017	3.1	1.9551	-1.2936	39.8
32m	10000	1.6933	1.4868	-0.2065	12.2	0.7193	-0.9740	57.5
	100000	4.1530	4.1639	0.0109	0.3	1.7841	-2.3689	57.0
	1000000	5.3287	5.4589	0.1302	2.4	2.4451	-2.8836	54.1
50 kPa	10000	2.4697	2.7657	0.2960	12.0	1.9995	0.4702	19.0
	100000	4.1234	4.3218	0.1984	4.8	2.9920	1.1314	27.4
	1000000	4.4885	4.6027	0.1142	2.5	3.1327	1.3559	30.2
100 kPa	10000	3.7161	4.4254	0.7093	19.1	3.4933	0.2228	6.0
	100000	5.4268	5.5972	0.1703	3.1	4.4549	0.9719	17.9

	1000000	5.7670	5.8772	0.1102	1.9	4.7349	1.0321	17.9
OCR=1.5	10000	1.0716	1.3491	0.2776	25.9	0.5974	0.4741	44.2
	100000	1.8954	2.0120	0.1166	6.2	0.8138	1.0816	57.1
	1000000	2.2596	2.2920	0.0323	1.4	0.9451	1.3146	58.2
OCR=2	10000	0.7955	0.9816	0.18611	23.4	0.4938	0.3016	37.9
	100000	1.2247	1.2666	0.0418	3.4	0.5546	0.6701	54.7
	1000000	1.5431	1.5416	-0.0015	0.1	0.6246	0.9185	59.5

*This is the reference case

Table 5. Parameter values used in new simplified method and FE modelling for *Väsby* test fill

The new simplified method									
γ_{soil} (kN/m^3)	OCR	κ/V	λ/V	ψ/V	k_z (m/day)	t_0 (day)	-	-	-
15	1.1	0.01	0.2006	0.01002	6.0×10^{-5}	1	-	-	-
FE modelling									
Layer (model)	γ_{soil} (kN/m^3)	OCR	κ^*	λ^*	μ^*	k_y (m/day)	E (MN/m^2)	c' (kPa)	ϕ' (°)
Fill (MC model)	17	-	-	-	-	0.1	35	1	38
Crust (SSC model)	15	6	0.02	0.2006	0.01002	6.0×10^{-5}	-	0.1	28
Soft soil (SSC model)	15	1.1	0.02	0.2006	0.01002	6.0×10^{-5}	-	0.1	28

Table 6. Relative error *and* relative difference of settlement *values of the new simplified method using Zhu and Yin method and Hypothesis A method for* Väsby *test fill*

Time (day)	Measured settlement data (m)	New simplified method			Hypothesis A method		
		$S_{totalB,t}$ (m)	$\Delta S_{totalB,t}$ (m)	$\xi_{totalB,t}$ (%)	$S_{totalA,t}$ (m)	$\Delta S_{totalA,t}$ (m)	$\xi_{totalA,t}$ (%)
470	0.2784	0.3240	0.0456	16.4	0.1629	-0.1155	41.5
2014	0.7906	0.7455	-0.0451	5.7	0.3354	-0.4552	57.6
20744	2.0379	2.1649	0.127	6.2	0.8334	-1.2045	59.1

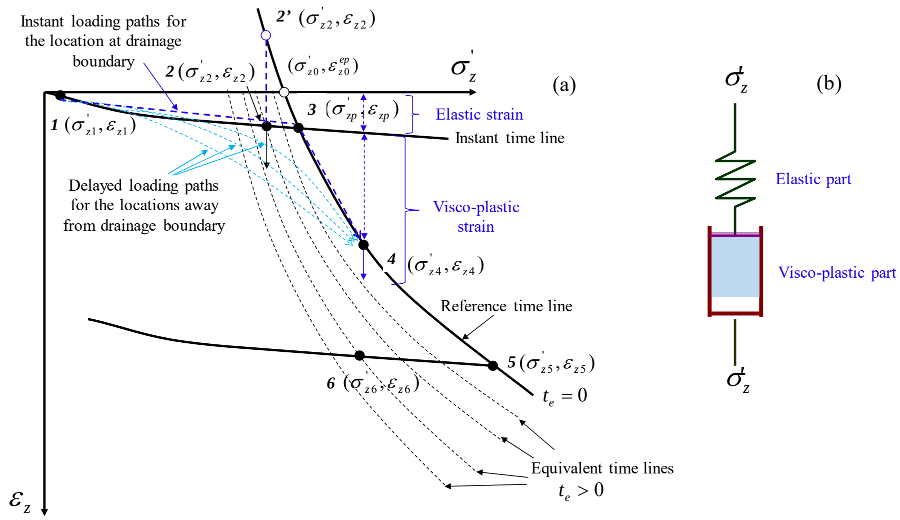

Instant loading paths for the location at drainage boundary
2' $(\sigma'_{z2}, \varepsilon_{z2})$
2 $(\sigma'_{z2}, \varepsilon_{z2})$
$(\sigma'_{z0}, \varepsilon^{ep}_{z0})$
σ'_z
(a)
3 $(\sigma'_{zp}, \varepsilon_{zp})$
Elastic strain
Instant time line
1 $(\sigma'_{z1}, \varepsilon_{z1})$
Delayed loading paths for the locations away from drainage boundary
Visco-plastic strain
4 $(\sigma'_{z4}, \varepsilon_{z4})$
Reference time line
5 $(\sigma'_{z5}, \varepsilon_{z5})$
6 $(\sigma'_{z6}, \varepsilon_{z6})$
$t_e = 0$
Equivalent time lines
$t_e > 0$
ε_z
σ'_z
(b)
Elastic part
Visco-plastic part
σ'_z

Figure 1

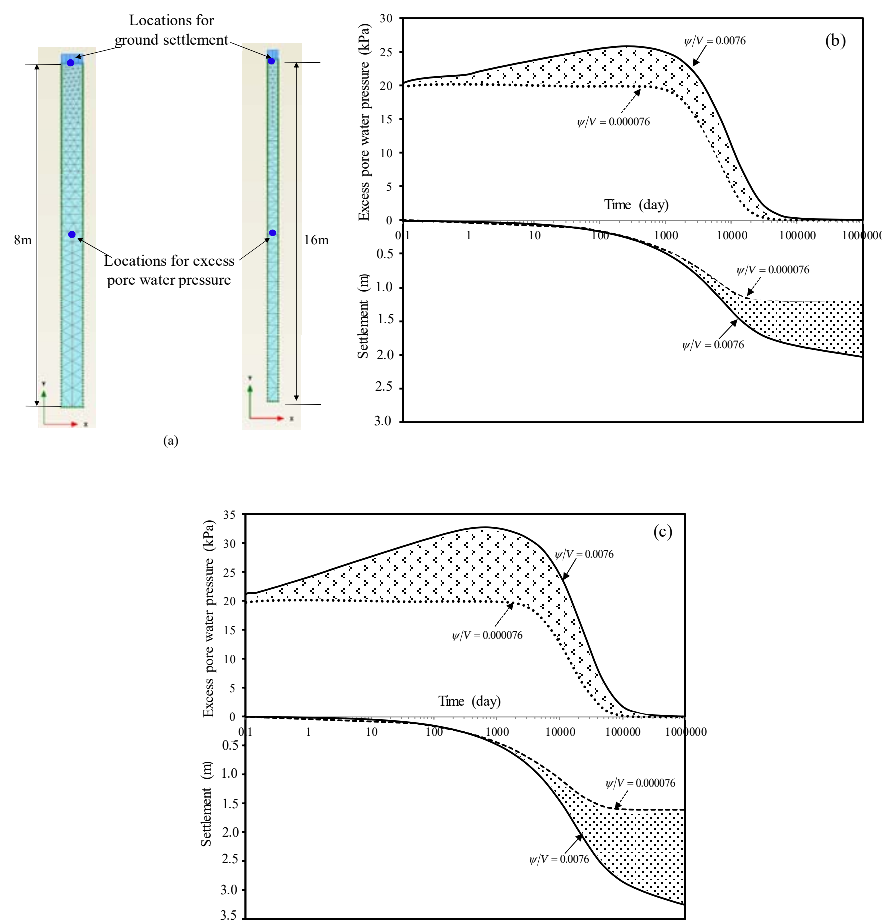

Locations for ground settlement
8m
Locations for excess pore water pressure
16m
(a)
(b)
Excess pore water pressure (kPa)
$\psi/V = 0.0076$
$\psi/V = 0.000076$
Time (day)
Settlement (m)
(c)

Figure 2

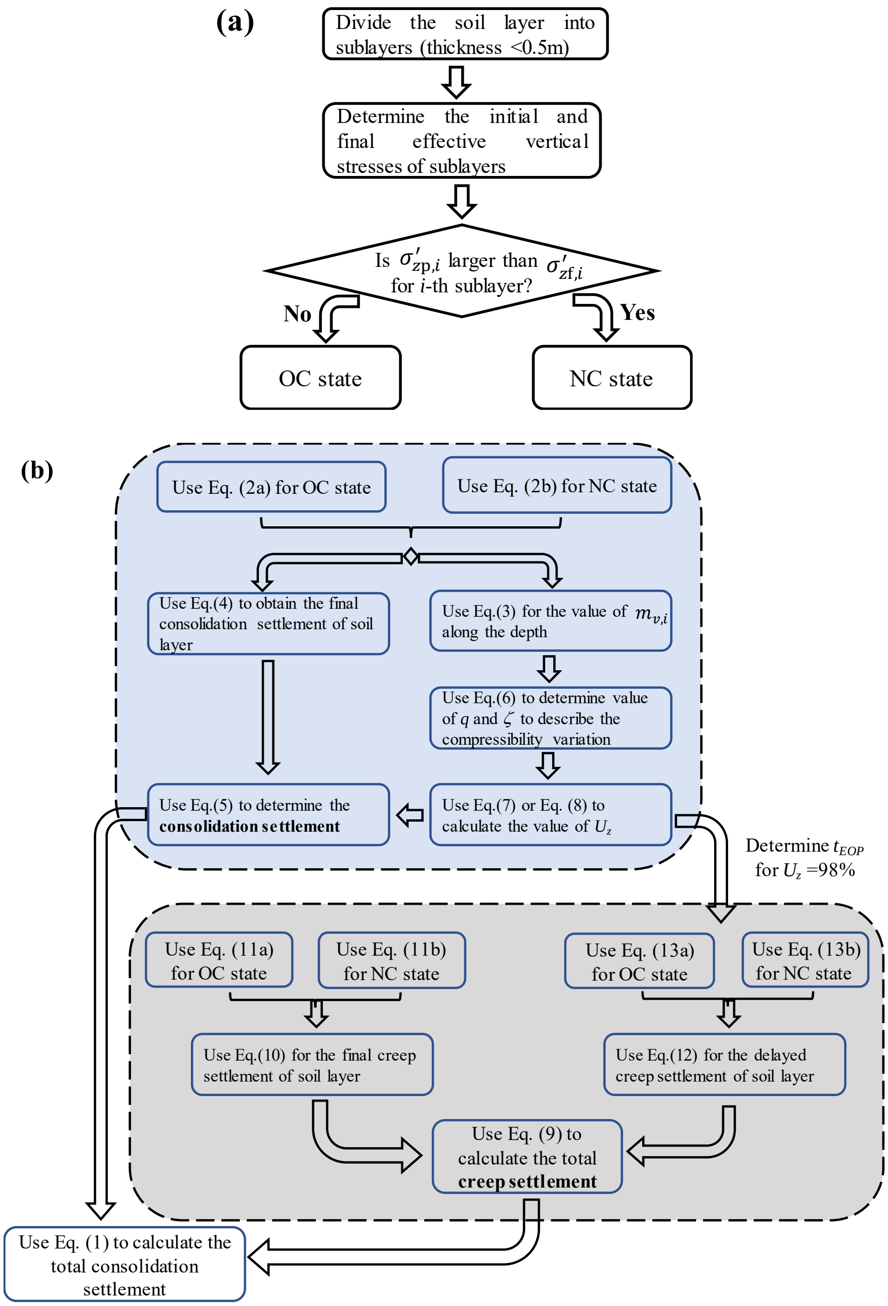


Figure 3

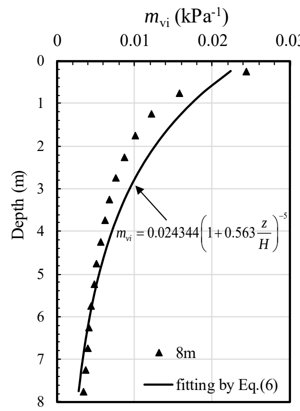


Figure 4

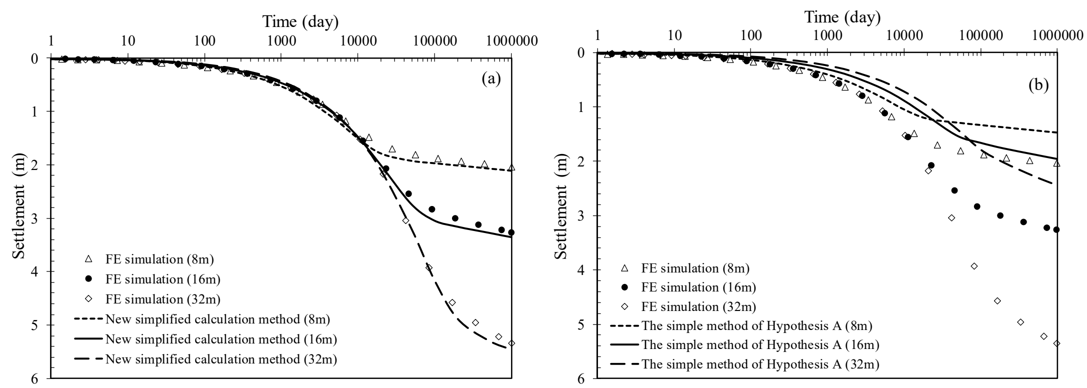


Figure 5

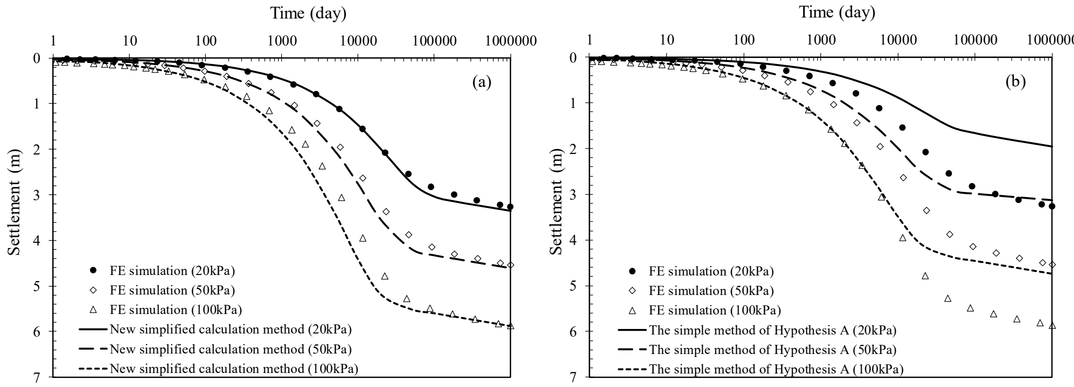


Figure 6

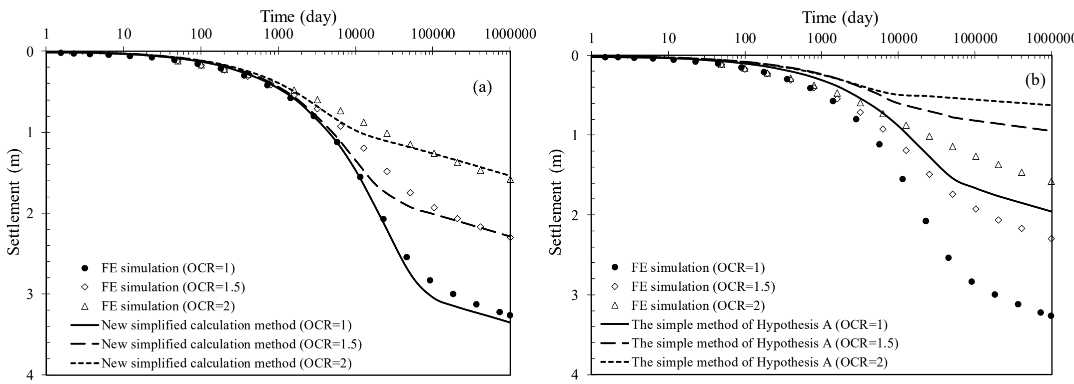


Figure 7

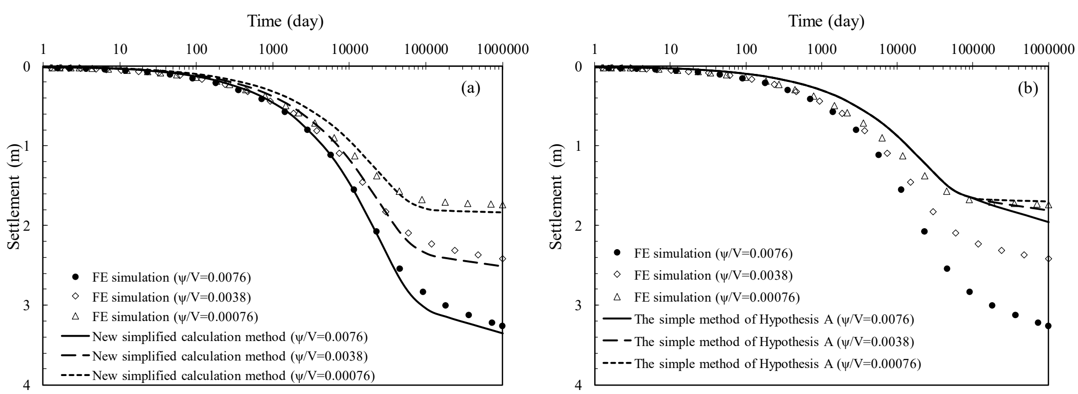
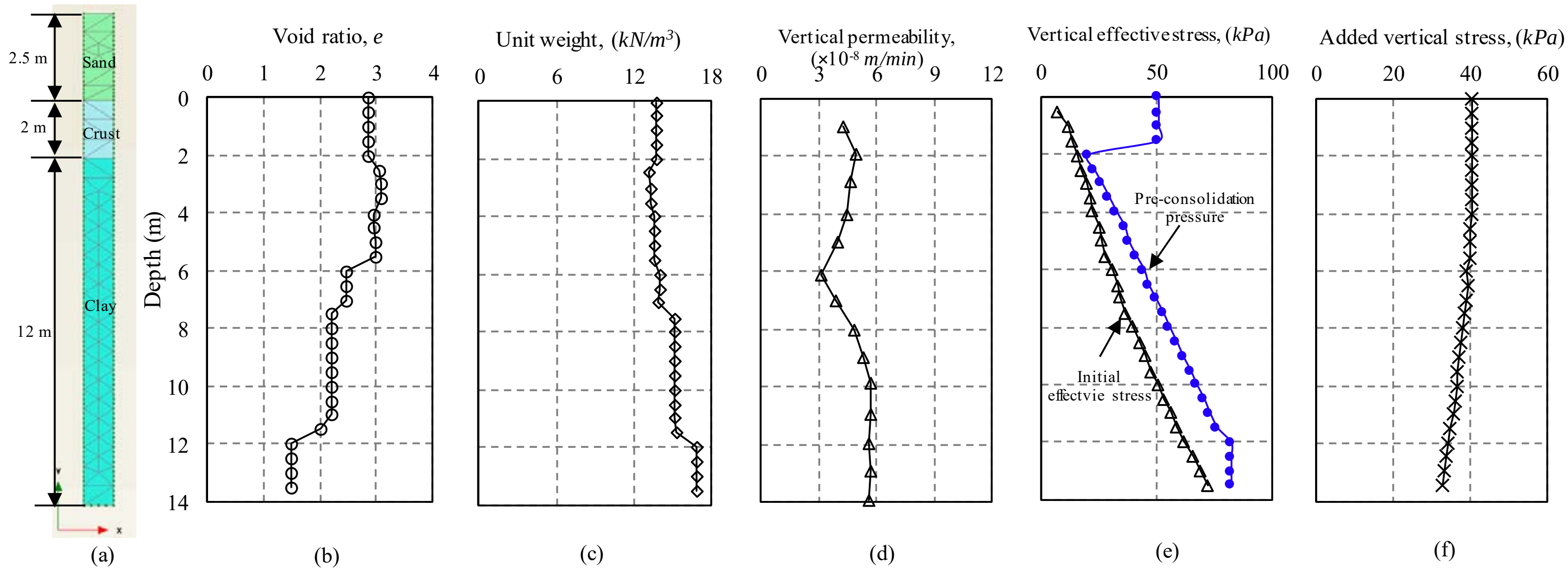


Figure 8



2.5 m
Sand
2 m
Crust
12 m
Clay
Void ratio, e
Unit weight, (kN/m3)
Vertical permeability, (×10-8 m/min)
Vertical effective stress, (kPa)
Added vertical stress, (kPa)
Depth (m)
Pre-consolidation pressure
Initial effectvie stress
(a)
(b)
(c)
(d)
(e)
(f)

Figure 9

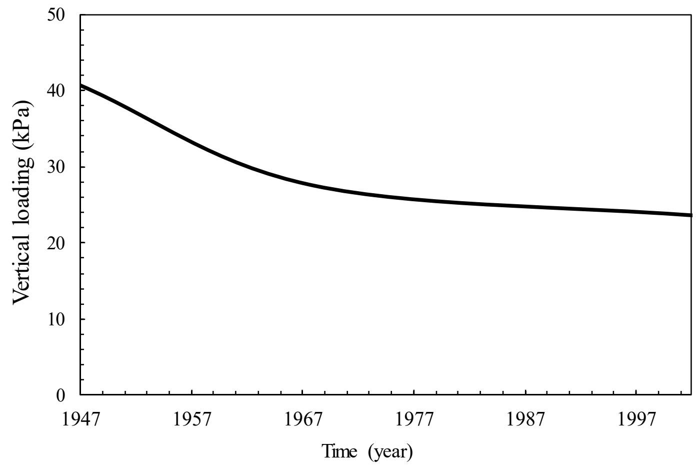


Figure 10

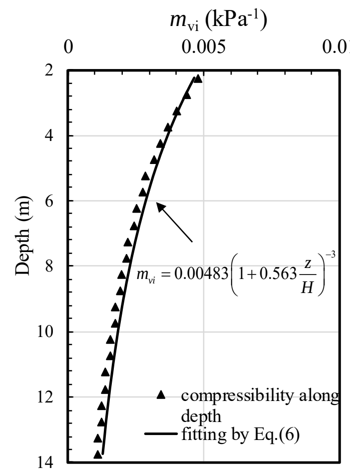


Figure 11

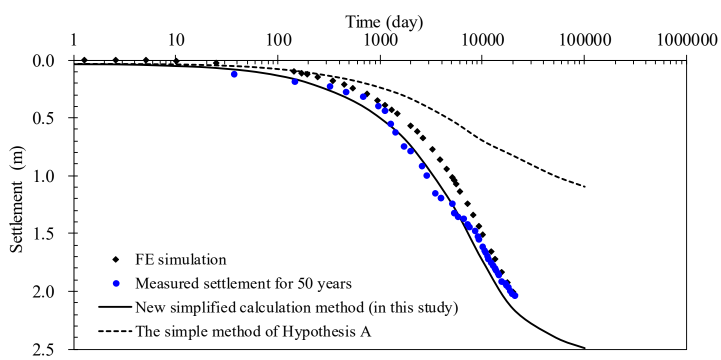


Figure 12