Fully Coupled Numerical Methods and a Simple Method for Calculating Settlements of Foundations and Reclamations on Clayey Soils

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ABSTRACT

Many foundations are supported by clayey soil ground. Marine reclamations are mostly constructed on seabed consisting of marine clayey soils or marine deposits. The settlement calculation of such foundations and reclamations is needed for their design and performance assessment. In this paper, the authors will firstly give a brief introduction to issues and mechanisms of large settlements of soft clayey soil grounds caused by creep of clayey soil skeleton. The authors will present both Hypothesis A and Hypothesis B methods for calculating consolidation settlements of clayey soils, and the history and equations of the two methods, explaining the inherent logical mistakes of Hypothesis A method. After this, the authors will briefly present fully coupled numerical methods using different Elastic Visco-Plastic (EVP) models for consolidation analysis of clayey soils. They will then introduce a simplified Hypothesis B method, namely simple method, for one-layer and multi-layers of clayey soils. Steps of how to derive this simple method are presented. Two examples of using this simple method by hand calculations or Excel calculations are explained. Verifications of the simple method by comparing with test data and fully coupled numerical methods are presented. Conclusions and remarks are presented at the end.

1 INTRODUCTION

The time-dependent excessive and differential settlement of clayey soil ground are threats to the long-term performance and serviceability of infrastructures such as foundations, road embankments, airports, artificial islands, *etc*. Built on weak soft clay layers, the Leaning Tower of Pisa had suffered from differential settlement since the completion of construction. The uneven distribution of thickness of the clayey soil layer caused the inclination of the tower, costing great effort for stabilization (Burland et al., 2003). The Kansai International Airport in Osaka, Japan consists of two artificial islands constructed over thick layers of marine clayey and sandy soils. The settlement of both islands has exceeded 10 meters, including post-construction settlements of 3 to 6 meters at different positions (http://www.kansai-airports.co.jp). The long-term settlement of the above-mentioned cases is a consequence of the combined effects of consolidation and creep. Rocchi et al. (2006) investigated the long-term settlement of Kansai International Airport on Osaka clay and found that the consolidation analysis without considering creep considerably underestimated its long-term settlement.

Consolidation of soils is induced by the dissipation of excess porewater pressure and the increase of effective stress under loading increment. Creep refers to the time-dependent (visco-plastic) compression of soils under constant effective stress. Viscous compression is a more general term which refers to the time-dependent (visco-plastic) compression of soils under effective stress variations. Other related time-dependent behaviours include stress relaxation and strain rate effects. There are different explanations of creep and other visco-plastic behaviours of clays, most of which have attributed it to the diffuse electrical double layer (bound water) around the negatively charged clay mineral particles. The bound water has significantly different mechanical properties compared with the free water in the soil skeleton. It is believed that creep is related to the viscous movement of the bound water and re-arrangement of soil particles, including but not limited to inter-particle sliding, molecule movements, and inter-particle contact breakdown (Bolt, 1956; Gupta, 1964;

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Kuhn & Mitchell, 1993; Le et al., 2012; Mitchell et al., 1968; Mitchell & Soga, 2005; Taylor, 1942; Taylor & Merchant, 1940; Yin, 2015). Double porosity theory was also interpreted as the origin of creep, that the drainage of double-layer water in the micropores is much slower than that of free water in macropores in the soil skeleton (Borja & Choo, 2016; Wang & Xu, 2007; Zeevaert, 1986). In general, researchers have realized that visco-plasticity is a key material property of clayey soils, and should be considered in engineering design. Several design guidelines have adopted fully coupled or simple methods to consider creep during primary consolidation (CGS, 2023; NPRA, 2022).

This paper focuses on the computational/calculation methods to consider such creep effect (or viscous effect in a more general sense) in consolidation settlement analysis of clayey soil ground. The key issues related to two hypotheses (A and B) are clarified, followed by explanations of Elastic Visco-Plastic (EVP) models and the fully coupled analysis methods with these models. Further, a new simple method based on the 1D EVP model is presented with examples considering different over-consolidation ratios and drainage conditions. The effects of several key parameters and coefficients are discussed. The proposed method is verified with field data and fully coupled numerical simulations for a real project in Hong Kong to reveal its validity for multiple soil layers under multi-stage loadings.

2 HYPOTHESIS A, HYPOTHESIS B, ELASTIC VISCO-PLASTIC MODELS, AND FULLY COUPLED METHODS FOR CONSOLIDATION-CREEP ANALYSIS

2.1 Hypothesis A and Hypothesis B

Ladd et al. (1977) first questioned whether creep is a "separate" phenomenon during primary consolidation, which corresponded to different predictions of long-term settlement in the field. If creep is a separate phenomenon caused by the structural viscosity of soils, the EOP ("End-of-Primary consolidation") strain will be dependent on the primary consolidation duration, which is "Curve B" in Figure 1. Otherwise, the EOP strain will be unique, as shown in "Curve A". Jamiolkowski et al. (1985) formally proposed the concept of two hypotheses, in which "Hypothesis A" considers creep only after primary consolidation while "Hypothesis B" considers creep during and after primary consolidation. For Hypothesis A, the EOP strain of soils is thickness-independent. The "primary consolidation" settlement of a thick layer in the field can be directly calculated based on the EOP stress-strain relationship of a thin specimen of oedometer test in the laboratory, and the "secondary compression" is only calculated after EOP using $\varepsilon_{\sec} = C_\alpha / (1+e_0) \cdot \log(t/t_p)$, where C_α is the "secondary consolidation coefficient" and t_p is the time at end of primary consolidation (Mesri & Choi, 1985). For Hypothesis B, the EOP strain of soils is influenced by creep and will be thickness-dependent.

To date, while there are still debates between the two hypotheses (Degago et al., 2011; Mesri & Vardhanabhuti, 2005; Szavits-Nossan, 2015), most researchers now support Hypothesis B that creep should be considered during primary consolidation and the EOP strain is different for different thicknesses (Degago et al., 2011; Grimstad et al., 2022; Kabbaj et al., 1988; Kavazanjian, 1988; Leroueil, 2006; Szavits-Nossan, 2015; Taylor, 1942; Yin & Feng, 2017). Kavazanjian (1988) and Szavits-Nossan (2015) pointed out that the conventional Hypothesis A method that ignores the creep during primary consolidation violated the principles of continuum mechanics. Degago et al. (2011) re-analyzed the previous test data that were considered to support Hypothesis A, and concluded that most of them support Hypothesis B if the initial states of the soils are carefully considered. However, the old Hypothesis A method was widely adopted due to its convenience for settlement calculation in engineering. To use Hypothesis B, the creep effects during the porewater pressure dissipation should be properly analyzed. In the following sections, the fully coupled numerical methods and a simple method that consider creep in consolidation settlement calculations based on Hypothesis B will be presented.

2.2 Elastic Visco-Plastic (EVP) model for clayey soils

A constitutive model describes the relations between different state variables, including strain, effective stress, and others such as strain rates. Reliable constitutive models are essential to the analysis of soil behaviour in the field. There are a series of constitutive models that can consider the creep effects of clayey soils (Adachi

& Oka, 1982; Den Haan, 1994; Vermeer & Neher, 1999; Yin et al., 2002; Yin & Graham, 1989; Yuan & Whittle, 2018) based on the isochrone or isotache concepts (Bjerrum, 1967; Šuklje, 1957).

Figure 2 shows a conceptual diagram of Yin and Graham's 1D EVP model (Yin & Graham, 1989, 1994, 1996). The model proposes an instant time line, reference time line, and equivalent time lines. The visco-plastic strain rate and equivalent time of a soil element are uniquely dependent on the current states of effective stress and vertical strain, but independent of the stress paths. The reference time line describes the soil state when the equivalent time is zero. For a given soil state, the total strain rate $\dot{\varepsilon}_z$ is contributed by elastic strain rate $\dot{\varepsilon}_z^e$ and the visco-plastic strain rate $\dot{\varepsilon}_z^{vp}$, and is calculated as:

$$\dot{\varepsilon}_z = \dot{\varepsilon}_z^e + \dot{\varepsilon}_z^{vp} = \frac{\kappa}{1+e_0}\frac{\dot{\sigma}_z^{'}}{\sigma_z^{'}} + \frac{\psi}{(1+e_0)t_0}\exp\left[\left(\varepsilon_{r0}-\varepsilon_z\right)\frac{1+e_0}{\psi}\right]\left(\frac{\sigma_z^{'}}{\sigma_{r0}^{'}}\right)^{\frac{\lambda}{\psi}} \tag{1}$$

where e_0 is the initial void ratio of the soil when strain is zero, κ is the slope of instant time line in $e-\ln\sigma_z^{'}$, λ is the slope of reference time line in $e-\ln\sigma_z^{'}$ coordinate, $\left(\sigma_{r0}^{'},\varepsilon_{r0}\right)$ is a fixed point at reference time line, usually taken as the pre-consolidation pressure point ($\sigma_p^{'},\varepsilon_p$) from oedometer tests, ψ is the creep coefficient (the slope of secondary compression section of $e-\ln t$ curves in oedometer tests), t_0 is the reference time, normally 24 hours in conventional oedometer tests. $\sigma_z^{'},\varepsilon_z$ represent the vertical effective stress and vertical strain in 1D condition. For convenience, the subscript z may be dropped in this paper.

The EVP model has been extended to a three-dimensional (3D) model in general stress-strain space based on the critical state models and overstress theory (Yin et al., 2002; Yin & Graham, 1999).

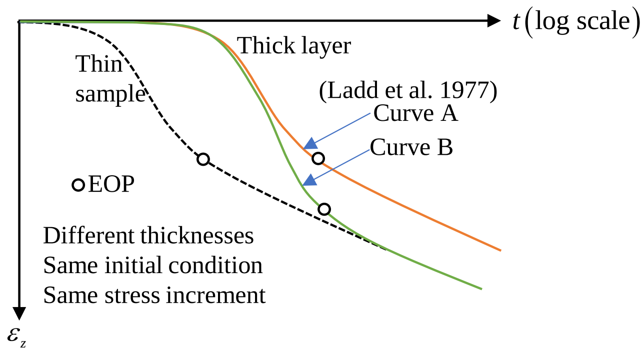

Figure 1: Illustration of Hypothesis A and Hypothesis B in 1D consolidation condition

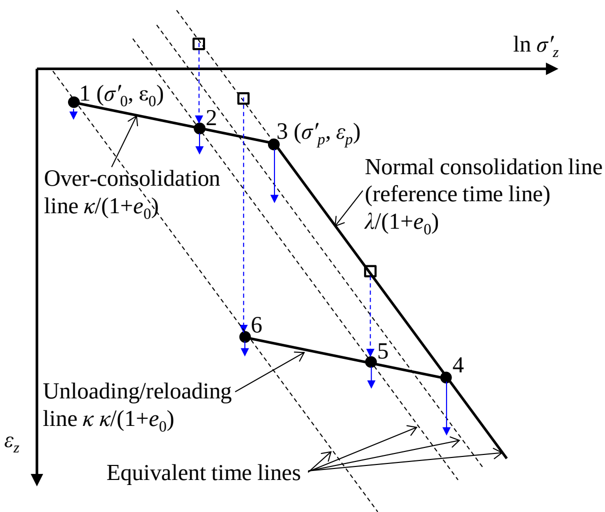

Figure 2: Conceptual diagram of Yin and Graham's 1D EVP model

2.3 Fully coupled numerical simulations based on EVP models

During the consolidation process of soils, the porewater pressure dissipates, and the effective stress is not constant. As shown in Equation (1), the strain rate of the soil is dependent on the stress-strain state. Therefore, creep is coupled with consolidation. A rigorous method is the fully hydro-mechanically coupled numerical simulation, such as finite element method (FEM) or finite difference method (FDM). In the fully coupled method, two sets of partial differential equations are to be solved: (i) a consolidation equation based on mass continuity and Darcy's law; and (ii) a constitutive equation such as Equation (1).

There have been successful applications of Yin and Graham's 1D and 3D EVP models in FDM and FEM, with verifications by measured data (Chen et al., 2021a, 2021b; Yin & Graham, 1996; Yin & Zhu, 1999). Chen et al. (2021a, b) successfully implemented the 3D EVP model in FEM software PLAXIS for analysis of long-term settlement of embankments on multiple layers of soft soils in Canada and Sweden. Another popularly used EVP model in commercial finite element analysis software PLAXIS is the Soft Soil Creep (SSC) model (Vermeer & Neher, 1999). The SSC model which was also called "isotache model" (Degago et al., 2011), shares a high similarity with Yin and Graham's EVP model in both theoretical frameworks and computation results. Besides, in recent years, more complicated EVP models have been developed to further consider the effects of anisotropy (Leoni et al., 2008; Zhou et al., 2005), creep limit (Yin et al., 2002), and temperature (Chen et al., 2023; Laloui et al., 2008).

3 A SIMPLE HYPOTHESIS B METHOD

Although the fully coupled methods for implementing EVP models have been successfully applied, they require a proper numerical programme and related skills, which may be unviable for designers. Conventional design methods by hand calculations or spreadsheets are still widely recommended in many design codes and widely used in engineering. However, these methods are usually developed based on Hypothesis A. As discussed, Hypothesis A underestimates the creep effects during primary consolidation. Therefore, it becomes ungently necessary to develop a proper simple design method based on Hypothesis B and the EVP model.

3.1 Formulations of the simple Hypothesis B method

Yin et al. (2022) proposed a simplified Hypothesis B method for calculating the consolidation settlement of clayey soil ground under different conditions. In this method, the total settlement S_{total} of the soil are analyzed by:

$$S_{total} = US_f + S_{creep} \tag{2}$$

where U is the average degree of consolidation, S_f is the final settlement estimated without considering creep effect, S_{creep} is the creep settlement separated from the "primary consolidation" settlement US_f. In this way, Equation (2) is a "de-coupled" method for creep analysis. S_f is calculated by the compressibility of soils measured from laboratory tests. If the volume compressibility m_v is known, S_f for a soil layer can be calculated by:

$$S_f = m_v \Delta\sigma' H \tag{3}$$

where $\Delta\sigma'$ is the load increment, H is the thickness of the layer. S_f can also be determined from the $e - \log\sigma'$ curves from oedometer tests to consider the nonlinear effective stress-strain behaviour. The $e - \log\sigma'$ curves are obtained at a fixed time t_0 (normally 24 hours). A single layer can be divided into sub-layers. Yin et al. (2022) suggested that a maximal thickness of 0.5m for each sub-layer can achieve enough preciseness. For each sub-layer, $S_{f,j}$ is calculated as:

$$S_{f,j}=\begin{cases} H_j\dfrac{C_r}{1+e_0}\log\dfrac{\sigma_0^{'}+\Delta\sigma_f^{'}}{\sigma_0^{'}} & for\ \sigma_0^{'}<\sigma_0^{'}+\Delta\sigma_f^{'}\le\sigma_p^{'} \\ H_j\dfrac{C_r}{1+e_0}\log\dfrac{\sigma_p^{'}}{\sigma_0^{'}}+H\dfrac{C_c}{1+e_0}\log\dfrac{\sigma_0^{'}+\Delta\sigma_f^{'}}{\sigma_p^{'}} & for\ \sigma_0^{'}\le\sigma_p^{'}<\sigma_0^{'}+\Delta\sigma_f^{'} \\ H_j\dfrac{C_c}{1+e_0}\log\dfrac{\sigma_0^{'}+\Delta\sigma_f^{'}}{\sigma_0^{'}} & for\ \sigma_p^{'}\le\sigma_0^{'}<\sigma_0^{'}+\Delta\sigma_f^{'} \end{cases} \tag{4}$$

where $\sigma_0^{'},\sigma_p^{'}$ is the initial effective stress and the pre-consolidation pressure respectively. $C_c=\lambda\ln 10$ is the compression index for the normal consolidation state, and $C_r=\kappa\ln 10$ is the recompression index for the over-consolidation state. The S_f of the whole layer is obtained by superposition of all sublayers, and m_v of the whole layer can then be calculated by

$$m_v=\frac{S_f}{H\Delta\sigma_f^{'}}=\frac{\sum_{j=1}^{n}S_{f,j}}{H\Delta\sigma_f^{'}} \tag{5}$$

It shall be pointed out that m_v above is calculated for the whole layer. The m_v value will be used to calculate the consolidation coefficient $c=k/(\gamma_w m_v)$ where k is hydraulic conductivity of this soil. The c value is then used to calculate the average degree of consolidation U . U can be calculated by classical consolidation theories, such as Terzaghi's 1D consolidation theory, unit cell theory for vertical drains (Hansbo, 1981), and others considering multi-layer profiles (Walker & Indraratna, 2009; Zhu & Yin, 2005), with known consolidation coefficient and thickness.

The other part of the total settlement contributed by creep S_{creep} is calculated using the following equation:

$$S_{creep}=\begin{cases}\alpha U^{\beta}S_{creep,f} & for\ t\ge t_{EOP,lab} \\ \alpha U^{\beta}S_{creep,f}+\left(1-\alpha U^{\beta}\right)S_{creep,d} & for\ t\ge t_{EOP,field}\end{cases} \tag{6}$$

In Equation (6), α,β are empirical coefficients to consider the delayed effective stress during porewater pressure dissipation. $S_{creep,f}$ is a creep settlement calculated under the final effective stress state without considering the delay by porewater pressure. $S_{creep,d}$ is a creep settlement under the final effective stress delayed by the consolidation process, assumed to start after the "End of Primary consolidation" (t_{EOP}). t_{EOP} usually corresponds to the time of $U=90\sim 98\%$ based on the engineering practice. α varies from 0 to 1. If $\alpha=0$, the method corresponds to the old Hypothesis A method. According to Yin and Graham's 1D EVP model, $S_{creep,f}$ and $S_{creep,f}$ should be calculated as:

$$\begin{cases} S_{creep,f}=H\dfrac{C_{\alpha}}{1+e_0}\log\dfrac{t_{ef}+t}{t_{ef}+t_0} \\ S_{creep,d}=H\dfrac{C_{\alpha}}{1+e_0}\log\dfrac{t_{ef}+t}{t_{ef}+t_{EOP,field}}\end{cases} \tag{7}$$

where $C_{\alpha}=\psi\ln 10$ is the creep coefficient for log scale, e_0 is the initial void ratio of the soil, t_{ef} is the equivalent time under the final effective stress and final settlement. According to Yin and Graham's EVP model, t_{ef} is calculated by:

$$t_{ef}=\exp\left[-\left(\varepsilon_{r0}-\varepsilon_f\right)\frac{1+e_0}{\psi}\right]\left(\frac{\sigma_f^{'}}{\sigma_{r0}^{'}}\right)^{-\frac{\lambda}{\psi}}-t_0 \tag{8}$$

where $\sigma_f^{'},\varepsilon_f$ is the final effective stress and the final vertical strain based on the $e-\log\sigma^{'}$ curves obtained at a fixed time t_0 (normally 24 hours). If the final state of soil is located at the normal consolidation line (reference time line), such as points 3 and 4 in Figure 2, the equivalent time is zero. If the final state is at an over-consolidation state (such as 2, 5, and 6 in Figure 2), the equivalent time will be larger than zero.

3.2 Verifications for a single layer

The proposed method in 3.1 is easy to implement in a spreadsheet. In this section, verifications on a single layer of clayey soil will be presented. The soil layer is the Hong Kong marine deposit (HKMD), which is a typical clayey soil in Hong Kong (Koutsoftas et al., 1987; Zhu et al. 2001). The parameters are listed in the Table 1. All mechanical parameters of soils can be conveniently obtained from conventional oedometer tests. For example, C_r is fitted as the slope of $e-\log\sigma_v^{'}$ curves at unloading-reloading state, C_c is fitted as the slope of $e-\log\sigma_v^{'}$ curves at normal consolidation state, C_α is fitted as the slope of $e-\log t$ curve of secondary consolidation at normal consolidation state. k_v, k_h are the hydraulic conductivity of undisturbed soils in vertical direction and horizontal direction. k_s represents the hydraulic conductivity of disturbed soils in the smear zone due to drain installation. r_d, r_s, r_e are the radii of vertical drains, smear zone, and the equivalent soil column surrounding each vertical drain respectively. The soil layer is subjected to a uniform vertical load of 20kPa. All of the calculation results are compared with simulations by PLAXIS 2D with the SSC model (i.e., fully coupled method) using the same set of parameters.

Table 1: Parameters of HKMD for the calculation

General parameters	e_0	k_v (m/day)	k_h (m/day)	k_s (m/day)	r_d (m)	r_s (m)	r_e (m)
	2.65	1.90×10^{-4}	1.90×10^{-4}	1.90×10^{-4}	0.027	0.134	0.788
Fully coupled method (PLAXIS)	κ^* $[=2\kappa/(1+e_0)]$	λ^* $[=\lambda/(1+e_0)]$	μ^* $[=\psi/(1+e_0)]$	τ (day)	$c^{'}$ (kPa)	$\varphi^{'}$ (deg)	
	0.02172	0.174	0.0076	1	0.1	30	
Simple method	C_r $[=\kappa\ln(10)]$	C_c $[=\lambda\ln(10)]$	C_α $[=\psi\ln(10)]$	t_0 (day)			
	0.0913	1.4624	0.0639	1			

(a) Calculations for a soil layer with different OCRs with/without vertical drains

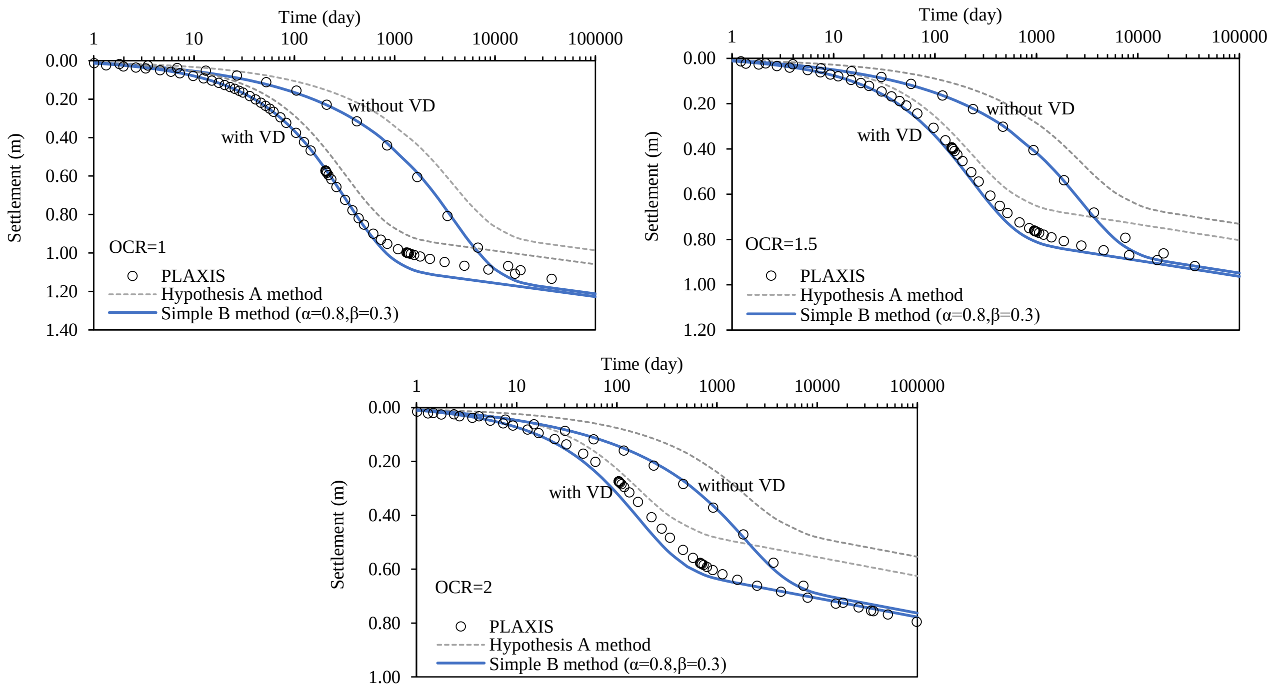

Figure 3: Calculation results by different methods for a clay layer with different OCRs with and without VD

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Different initial over-consolidation ratio (OCR) from 1, 1.5, to 2, and drainage conditions with and without vertical drains (VD) are considered in the calculations. OCR is defined as the ratio between $\sigma_p^{'}$ and $\sigma_0^{'}$ for each sub-layer. For soils without VDs, U is calculated using Terzaghi's equations. For soils with VDs, U is analyzed using Hansbo (1981)'s consolidation theory. The radii of drains, smear zone, and equivalent soil are adopted following Zhu et al. (2001). $\alpha = 0.8, \beta = 0.3$ are suggested values in the previous studies. The time achieving $U = 95\%$ is selected as $t_{EOP,\,field}$.

Figure 3 shows the computation results for these different cases. It can be found that for all cases of an HKMD layer, the calculation results by the simple Hypothesis B method are highly consistent with the fully coupled method by PLAXIS. The old Hypothesis A method always obtains smaller settlements compared with the other two methods. The underestimation of Hypothesis A is more significant in the case without VD than the cases with VD. The reason is that for soils with VDs, the consolidation period is much shorter, and the creep developed during the "primary consolidation" will be smaller, which slightly reduces the error caused by Hypothesis A.

(b) Investigations on the effects of α, β

As discussed in the previous sections, α, β are two constant coefficients, which approximately describe the coupling of consolidation on creep settlement. It is important to verify the robustness and versatility of their values for typical cases. Figure 4 shows the calculation results of the simple method with different values of α (0.6, 0.7, and 0.8). Although there is a small difference at the final stages for different values of α, the computations are all reasonable. For the soil layer with OCR from 1, 1.5, to 2, the best-fitted α may slightly increases from 0.6 to 0.8. Similar results are retained in both VD and non-VD cases.

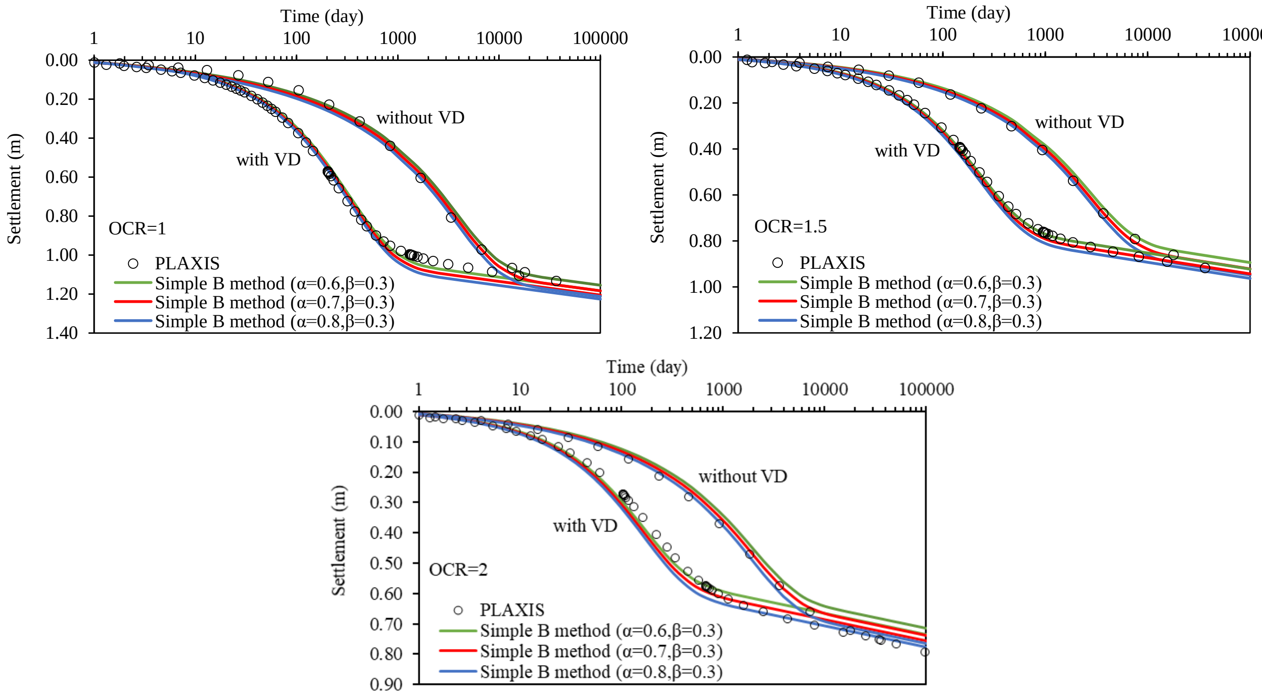

Figure 4: Calculation results by simple methods with different α

Figure 5 shows the calculation results of the simple method with different values of β (0.1, 0.3, and 0.5) under the same α. It can be found that β mainly affects the earlier stages of consolidation, but cannot affect the final stages of consolidation. For the soil layer with OCR from 1, 1.5, to 2, the best-fitted α may slightly increases from 0.6 to 0.8. The influence of β is more significant in the cases without VD than those with VD. Besides, it shows that $\beta = 0.3$ fits the best with the fully coupled analysis in PLAXIS. With increase of β, the computed settlement tends to decrease. However, as consolidation goes on, the effects of β gradually diminish. Although the influence of OCR exists, $\alpha = 0.8, \beta = 0.3$ can still be used for convenience with OCR from 1 to 2, because the errors are minor. The minor effects of OCR on α, β can be attributed to the smaller compressibility of soils in OC state. With smaller compressibility, the consolidation during OC state is faster, and the differences of creep rates of OC soils during primary consolidation will contribute less to the total creep settlement.

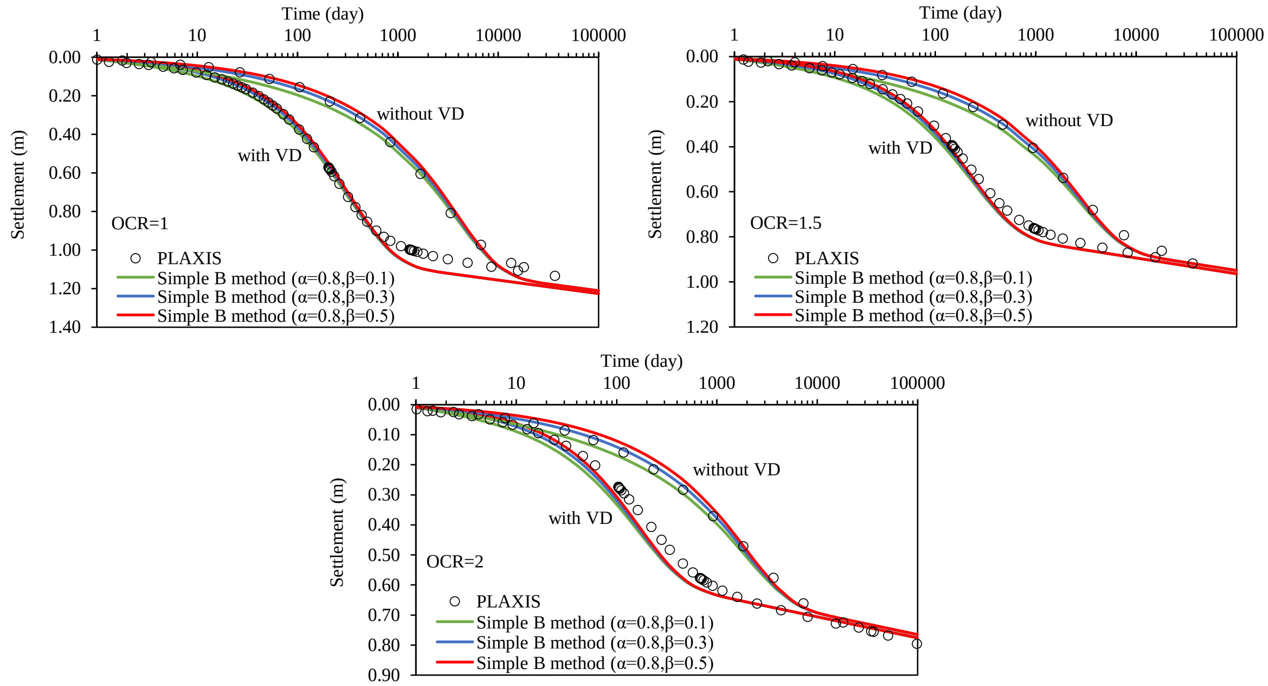

Figure 5: Calculation results by simple methods with different β

(c) Investigations on the criteria degree of consolidation for EOP

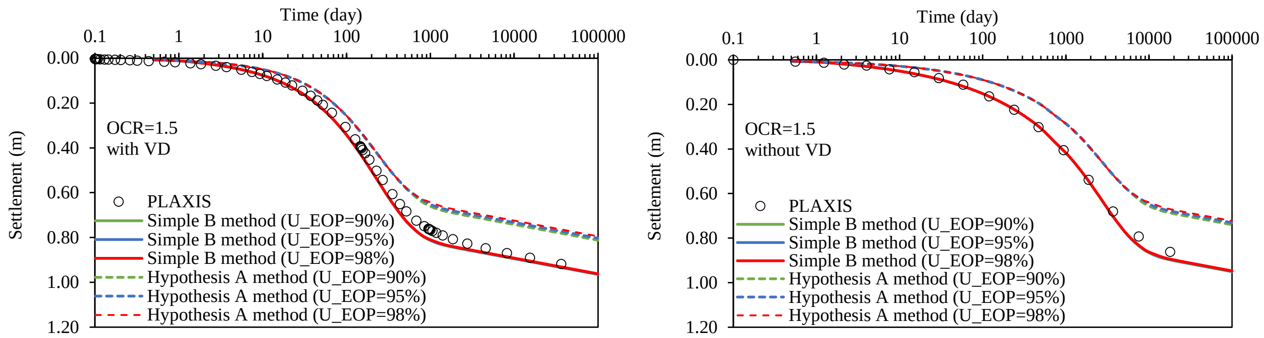

Figure 6: Calculation results by simple methods with different U_{EOP}

According to Equation (6), the determination of t_{EOP} is important. In engineering practice, different criteria could be adopted to determine t_{EOP} by U_{EOP}, which can be 90~98%. In this section, the settlement curves using different U_{EOP} (*i.e.*, 90%, 95%, and 98%) are calculated and compared in Figure 6. It is interesting that within normally adopted ranges, U_{EOP} has little influence on the calculation results. The influence of U_{EOP} is more profound in the Hypothesis A method.

3.3 Verification for multiple clay layers with vertical drains and staged loadings

In real projects, embankments or reclamations can be constructed over multi-layered soils with multi-staged loadings. The proposed method can be applied in analysis of such cases. In this section, the case study on a test embankment for the Hong Kong Chek Lap Kok International Airport is presented.

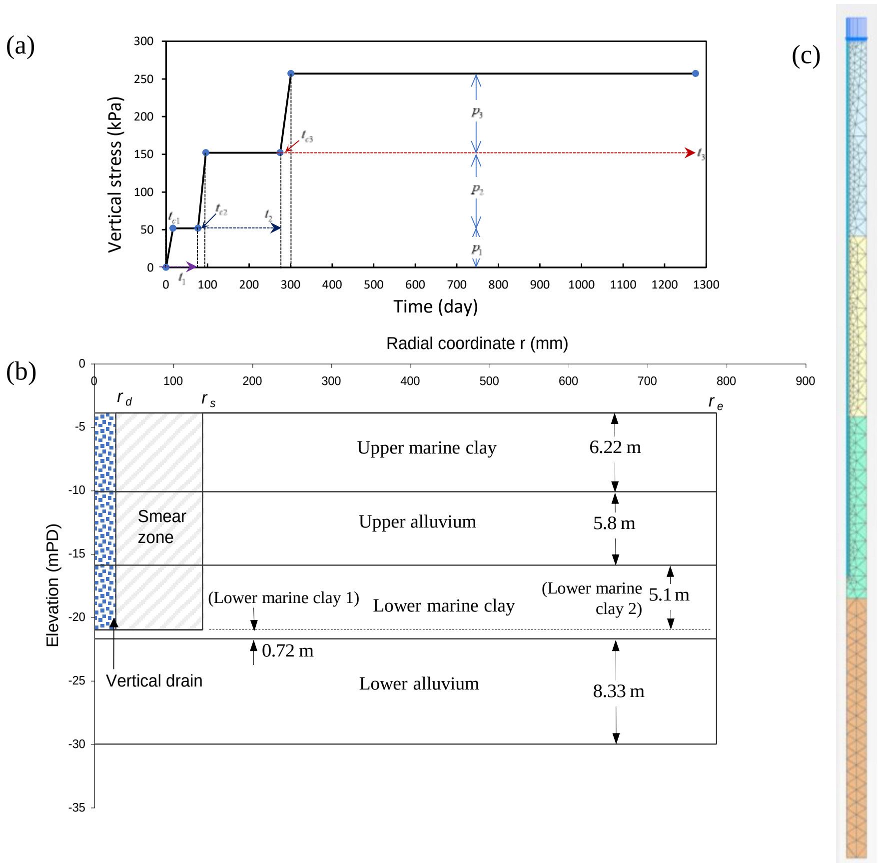

Figure 7: Loading sequence and soil profile of the test embankment (after Zhu et al. 2001): (a) loading history, (b) soil profile and vertical drain dimensions, and (c) axisymmetric finite element model in PLAXIS 2D

The embankment was an artificial island over four layers of marine soils: upper marine clay, upper alluvium, lower marine clay, and lower alluvium. Vertical drains were installed until the lower marine clay layer to accelerate the consolidation. Three stages of loading were applied vertically on the top of the marine clay by gravel filling. Settlement was continuously monitored at four different depths (initially 0, 3, 6, 14.5 meters at depth). The soil profile and loading history are shown in Figure 7. The parameters of each layer of soil are reported in Yin et al. (2022).

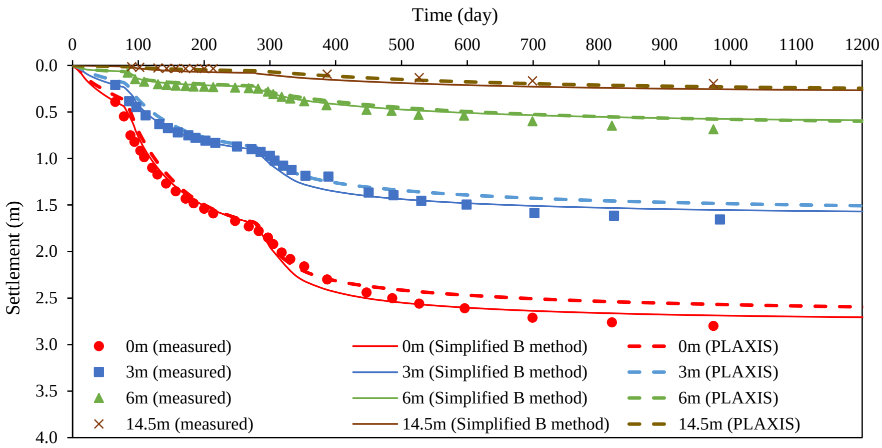

Figure 8: Calculation results of simple method and finite element method compared to measurement for the test embankment of HKIA

The calculation of the multi-layered soil system with vertical drains under each ramp loading is conducted by the spectrum method developed by Walker and Indraratna (2009). The total settlement is calculated by a simple superposition of all different layers and all loading stages. The computed results are compared with the measured data as well as the fully coupled finite element simulations conducted by PLAXIS 2D with SSC model, as shown in Figure 8. The results indicate that the simple Hypothesis B method can provide accurate predictions on the settlement of multi-layered clayey soil ground under multi-staged loadings.

4 CONCLUSIONS

To properly consider the effect of creep, the fully coupled methods and a simple method are presented and verified in this study for both single-layer and multi-layer clayey soils with and without vertical drains. Several conclusions and remarks can be listed as follows.

(i) Creep plays an important role in the time-dependent consolidation settlement of clayey soil grounds. The old Hypothesis A method without properly calculating creep during primary consolidation will underestimate the settlement.

(ii) The proposed simple Hypothesis B method is convenient and efficient for settlement predictions, with high accuracy compared to the fully coupled simulations and the measured data.

(iii) The values of $\alpha = 0.8, \beta = 0.3$ for the simple method are suitable for the typical cases with OCR from 1.0 to 2. Using different degrees of consolidation as criteria of EOP has little influence on the computation results.

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