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RESEARCH PAPER

Study of the one-dimensional consolidation and creep of clays with different thicknesses using different hypotheses and three elastic visco-plastic models

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Abstract

The one-dimensional consolidation analysis of clays considering creep compression is a classical issue in soil mechanics and geotechnical design. The major debate lies in how to predict the consolidation settlement for a thick layer in the field using parameters obtained from a thin specimen from the laboratory. Different hypotheses have been advocated, based on which various methods and constitutive models have been developed. However, there are still some questions unaddressed and concepts inconsistently used, which may mislead engineers in the selection of methods/models and may result in settlements underestimated on a risk design side. In this paper, a state-of-the-art review and a thorough comparison study are performed on the existing methods and models for the consolidation analysis of clays exhibiting creep, from theoretical derivations to numerical simulations in comparison with soil test data. An in-depth discussion is carried out on several key issues related to the thickness effects on the time-dependent compression behaviour of clays. The arguments of Hypothesis A and Hypothesis B are revisited based on the current development of constitutive theories. Three existing elastic visco-plastic (EVP) models that consider the creep compression implicitly during the whole consolidation process can perform well in predicting the settlement of clays with different thicknesses, and are in line with Hypothesis B. It is concluded that using existing EVP models based on porous-media continuum mechanics is a rigorous scientific method (also called "rigorous" Hypothesis B method), which is superior to the old Hypothesis A method which has logic errors and may result in unsafe underestimation of settlements.

Keywords Clay · Consolidation · Creep · Elastic visco-plastic models · Hypotheses A and B

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1 Introduction

One of the essential assumptions in Terzaghi's one-dimensional (1D) consolidation theory is a unique relation between void ratio and effective stress. However, laboratory test data demonstrated that the volume compression of clays continues after the dissipation of excess porewater pressure, i.e., under constant effective stress [5, 38]. In oedometer tests or constant load conditions, such observed creep compression is called secondary compression, while the compression during the dissipation of excess porewater pressure is the primary consolidation. The time-dependency of compression of clays has been extensively studied in the past decades.

However, different interpretations and hypotheses have been proposed for modelling the effective stress-strain behaviour of soils, resulting in dramatically different predictions of consolidation and settling behaviour in field conditions. In the old design methods, the settlement of a thick layer of clay in the field is usually considered to contain three parts: the instant settlement due to distortion and horizontal expansion of the soil, the primary consolidation settlement due to excess porewater pressure dissipation, and the secondary compression due to creep. For 1D compression, the instant settlement could be ignored. For the primary consolidation settlement, some researchers considered that the overall strain at the end of primary consolidation (EOP) is independent of time and thickness, and therefore the EOP stress-strain relation obtained in oedometer tests can be directly applied to the EOP settlement prediction in the field. Different from this old method, some researchers consider that clays should be modelled as a visco-plastic material. Several elastic visco-plastic (EVP) models have been developed, and the visco-plastic strain rate is considered as a function of the effective stress-strain states and histories of the soil, with consistent expressions during both primary and secondary compression.

Behind these theoretical models, there is a long-history and active debate between the so-called "Hypothesis A" and "Hypothesis B" [13, 14, 19, 29]. Hypothesis B has been supported by most EVP models. However, contradictory interpretations are still frequently reported, confusing engineers and general researchers. A recently developed EVP model by Yuan and Whittle [50] was considered to cover both hypotheses with different parameters, while other EVP models are usually only associated with Hypothesis B. It would be interesting to compare the similarities and differences between the current models, and to clarify their matches with Hypotheses A or B for typical cases.

In this study, a thorough comparison will be conducted among existing mainstream theories with different key assumptions and interpretations. The existing methods will be implemented to simulate the consolidation behaviour of clayey soils reported by the literature. Discussions will be conducted on the simulation results and the fundamental issues regarding soft soil modelling methods and the two contradicting hypotheses.

2 Existing methods for analysing creep of clays

Creep is a common phenomenon for many engineering materials, especially for clayey soils. There are different explanations for the mechanism of creep in clayey soils, most of which have been centralized on the existence of viscous bound water, i.e., the diffuse double layer between particles and inside the inter-particle microvoids. The continuous creep of the soil skeleton under constant effective stress is mainly attributed to the viscous process of bound water movement, inter-particle contact sliding, and particle rearrangement [21, 33].

Taylor and Merchant [38] presented a differential equation to describe the compression of soils considering the change of void ratio as a function of both effective stress increment and time:

$$\frac{de}{dt} = \frac{\partial e}{\partial t} + \frac{\partial e}{\partial \sigma'}\frac{\partial \sigma'}{\partial t} \tag{1}$$

where e is the void ratio, t is time, σ' is the effective stress (originally denoted as p). They suggested that the term for partial derivative of void ratio on time, $\frac{\partial e}{\partial t}$, should "follow a law resembling that for viscous flow or creep". However, $\frac{\partial e}{\partial t}$ during the primary consolidation cannot be measured directly since the other term $\frac{\partial e}{\partial \sigma'}\frac{\partial \sigma'}{\partial t}$ is non-zero. Therefore, proper mathematical models need to be proposed and verified. To date, there are mainly two categories of methods: the unique EOP theory and the EVP models.

2.1 Unique EOP: the old Hypothesis A method

To solve Eq. (1), the old method is to ignore the effect of creep during primary consolidation. A more sophisticated assumption for this method is that although creep may happen during the primary consolidation, somehow it does not affect the unique stress-strain (void ratio) relationship of the soil at EOP state [27–32]. This method divides the total compression into "primary consolidation" and "secondary compression", as shown in Fig. 1. The void ratio at EOP is unique, independent of the period required. To guarantee this, the two terms $\frac{\partial e}{\partial \sigma'}\frac{\partial \sigma'}{\partial t}$ and $\frac{\partial e}{\partial t}$ before EOP should be highly interdependent and their relative contributions to the total settlement at EOP are fixed. A thicker

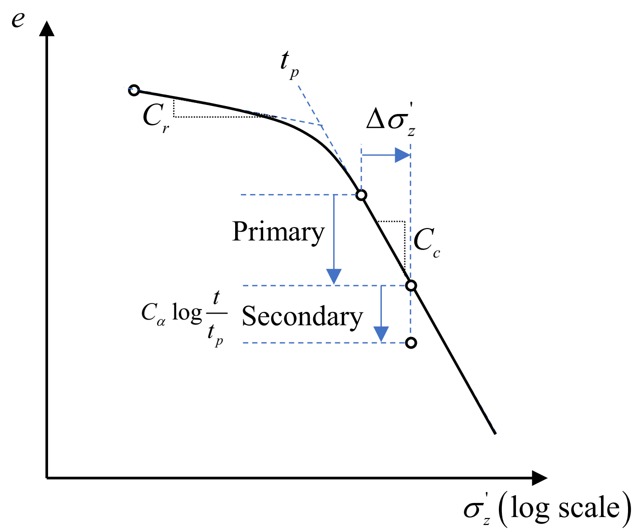


Fig. 1 Conceptual diagram for of the unique EOP theory

soil layer requires a longer period of consolidation and smaller $\frac{\partial e}{\partial t}$ before EOP.

Given the unique EOP, the creep deformation before EOP need not be analysed. The secondary compression caused by creep after EOP is calculated by:

$$S_{\text{secondary}} = \frac{C_\alpha}{1 + e_0} L_0 \log \frac{t}{t_p} \tag{2}$$

where S is the settlement of the soil layer, L_0 is the initial thickness, e_0 is the initial void ratio, t is the time elapsed from load application, t_p is the time of primary consolidation, C_α is the secondary compression coefficient. Equation (2) will predict parallel consolidation curves ($e - \log t$) of different thicknesses, with identical EOP strain and parallel secondary lines, corresponding to the "curve A" described by Ladd et al. [19], as shown in Fig. 2. Therefore, the method is also termed "Hypothesis A method".

The Hypothesis A method is convenient for engineers, and it was claimed that this assumption of "unique EOP" is supported by many test data [30, 32]. However, some researchers have challenged such conclusions and provided opposite interpretations [5, 9, 14].

2.2 Elastic visco-plastic constitutive models

The second method for modelling the creep behaviour of clays is using visco-plastic constitutive equations, where the plastic deformation of the soil skeleton is considered time-dependent or rate-sensitive. During consolidation, the total deformation of the clays is coupled with the dissipation of porewater pressure. Therefore, it can also be called a "rigorous" method.

2.2.1 Models based on equivalent time lines and isotache

Bjerrum [3] proposed the concepts of "instant compression" and "delayed compression" to replace the interpretation of primary and secondary consolidation, and described a family of time lines to describe the time-dependent compression, as shown in Fig. 3. Yin and Graham [46–48] further proposed and developed the concepts of instant time line, reference time line, and equivalent time lines into a 1D elastic visco-plastic model, as shown in

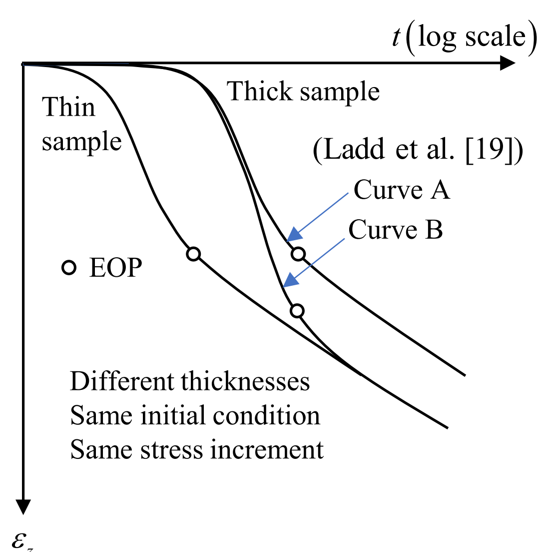


Fig. 2 Conceptual diagram for consolidation settlement curves of clays with different thicknesses

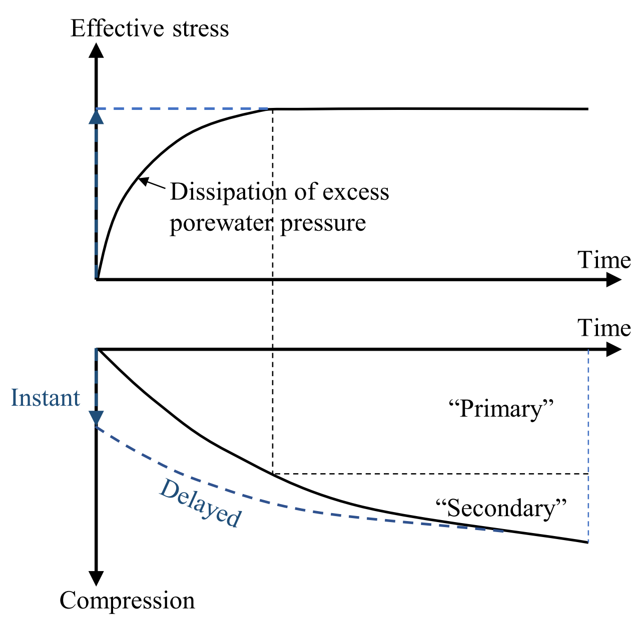


Fig. 3 Illustration of the instant and delayed compression strain coupled with consolidation

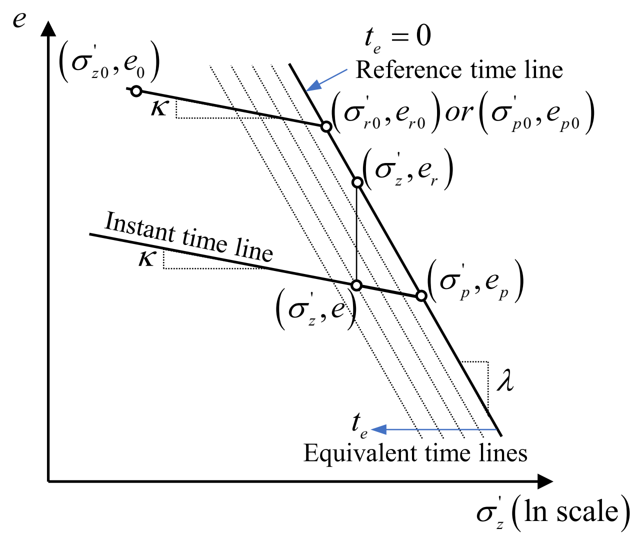


Fig. 4 Conceptual diagram for a 1D EVP model based on equivalent time or isotache

Fig. 4. The vertical strain ε_z of a soil element under 1D compression condition can be described by:

$$\varepsilon_z = \varepsilon_{r0} + \frac{\lambda}{1+e_0}\ln\frac{\sigma'_z}{\sigma'_{r0}} + \frac{\psi}{1+e_0}\ln\frac{t_0+t_e}{t_0} \tag{3}$$

where σ'_z is the effective vertical stress, t_0 is a reference time, t_e is the equivalent time, λ is the slope of the normal compression line (reference time line), $(\sigma'_{r0}, \varepsilon_{r0})$ is a fixed point on the reference time line, ψ is the creep coefficient. The vertical visco-plastic (creep) strain rate $\dot{\varepsilon}_z^{vp}$ can be derived as:

$$\dot{\varepsilon}_z^{vp} = \frac{\partial \varepsilon_z^{vp}}{\partial t_e} = \frac{\psi}{(1+e_0)t_0}\exp\left[\frac{1+e_0}{\psi}\left(\varepsilon_{r0}-\varepsilon_z\right)\right]\left(\frac{\sigma'_z}{\sigma'_{r0}}\right)^{\frac{\lambda}{\psi}} \tag{4}$$

The model depicts a unique relationship between creep rate $\dot{\varepsilon}_z^{vp}$ (or t_e), effective stress, and vertical strain (or void ratio). Meanwhile, the elastic deformation of clays is described by the "instant time line", which can be expressed as:

$$\dot{\varepsilon}_z^{e} = \frac{\partial \varepsilon_z^{e}}{\partial \sigma'_z}\frac{\partial \sigma'_z}{\partial t} = \frac{\kappa}{1+e_0}\frac{\dot{\sigma}'_z}{\sigma'_z} \tag{5}$$

where κ is the slope of instant time line (recompression line in the Cam-Clay model).

Another popularly used model is the Soft Soil Creep (SSC) model developed by Vermeer and Neher [40] based on Den Haan [10]. The model follows Bjerrum's [3] interpretation of creep, and describes the creep strain rate by the current effective stress and updated pre-consolidation pressure, which can be written as:

$$\dot{\varepsilon}_z^{vp} = \frac{\mu^*}{\tau}\left(\frac{\sigma'_z}{\sigma'_p}\right)^{\frac{\lambda^*-\kappa^*}{\mu^*}} \tag{6}$$

where $\lambda^*, \kappa^*, \mu^*, \tau$ are constant parameters and equivalent to $\frac{\lambda}{1+e}, \frac{2\kappa}{1+e}, \frac{\psi}{1+e}, t_o$ in Yin and Graham's EVP model [46]. σ'_p is the pre-consolidation pressure which is changing as soil compresses, as shown in Fig. 4. The strain ε used by Vermeer and Neher [40] and Den Haan [10] is a logarithmic strain instead of an engineering strain. However, when ignoring this difference and considering small-strain conditions, $\lambda^*, \kappa^*, \mu^*$ can be approximated as $\frac{\lambda}{1+e_0}, \frac{2\kappa}{1+e_0}, \frac{\psi}{1+e_0}$ respectively in Yin and Graham's 1D EVP model.

Both EVP models by Yin&Graham and Vermeer&Neher accord with the "isotache" concept developed since Šuklje [37], which refers to separated stress-strain curves under different rates of strain, and therefore the models are sometimes called "isotache-type" models. There are also other EVP models [1, 4, 11, 12, 16, 18, 26, 49] developed based on such an idea. Although expressed in various mathematical formulas, these models all depict that the visco-plastic strain rate is dependent on the current effective stress-strain state of the soils, but independent of the loading paths or degree of consolidation.

2.2.2 Model beyond isotache–MIT-SR

Beyond the widely used "isotache-type" models, Yuan and Whittle [50] established a 1D EVP model considering strain rate effects and temporal effects with different methods. The complete formulations considering 3D stress-strain condition have been developed based on this 1D model, namely MIT-SR model [51]. It was considered that the visco-plastic strain rate of clays is not only dependent on the current effective stress and void ratio state, but also influenced by the memory of the soil skeleton. Under 1D compression, the visco-plastic strain rate can be expressed as:

$$\dot{\varepsilon}^{vp} = R_a \cdot \left(\frac{\sigma'_z}{\overline{\sigma}'_p}\right) \tag{7}$$

where R_a is an independent variable called "internal strain rate", $\overline{\sigma}'_p$ (originally denoted as σ'_p by Yuan and Whittle) is the yielding stress dependent on void ratio. It should be noted that $\overline{\sigma}'_p$ in Eq. (7) corresponds to the stress on the reference compression line (termed "limiting compression curve", LCC) at the current void ratio, as shown in Fig. 5, which is slightly different from σ'_p in Eq. (6). R_a represents the history effects of visco-plastic straining, which evolves with time following a first-order differential equation:

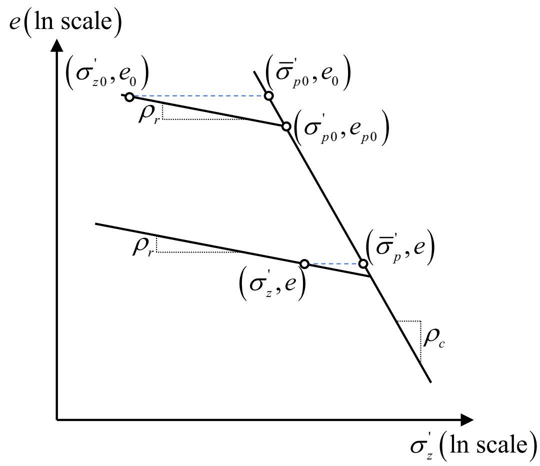


Fig. 5 Conceptual diagram for the 1D MIT-SR model by Yuan and Whittle [50]

$$\dot{R}_a = [f(\dot{\varepsilon}) - R_a]m_t \tag{8}$$

where $f(\dot{\varepsilon})$ is an activation function for applied total strain rate $\dot{\varepsilon}$, m_t is a transient coefficient. Equation (8) allows R_a to transit smoothly from R_{a0} to $f(\dot{\varepsilon})$ with time. $f(\dot{\varepsilon})$ represents a steady state of the soil, which is formulated as:

$$f(\dot{\varepsilon}) = \frac{\rho_c - \rho_r}{\rho_c}\dot{\varepsilon}\left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_{ref}}\right)^{-\beta} \tag{9}$$

And the mathematical expression of m_t is given by Yuan and Whittle [50] as:

$$m_t = \left(\frac{\rho_c}{\rho_\alpha} - 1\right)\frac{\dot{\varepsilon}^{vp}}{\rho_r n} + \dot{\varepsilon} \tag{10}$$

where $\rho_r = \kappa/e$ and $\rho_c = \lambda/e$ are the slopes of the swelling line and normal compression line respectively in $\ln e - \ln \sigma'_z$ coordinate, $\dot{\varepsilon}_{\text{ref}}$ is the reference strain rate adopted in constant-rate-of-strain (CRS) tests, β is the parameter describing the rate-sensitivity of compression lines from CRS tests.

Figure 6 presents the parametric studies on the compression curves of San Francisco Bay Mud based on CRS test data [17, 50]. Parametric studies are carried out to reveal the effects of different β and ρ_α. It can be found that β mainly controls the rate sensitivity in the steady state, while ρ_α controls the transient behaviour beyond the steady state, including the significant overshooting behaviour. When $\beta = 0$, the compression curves at steady-state converge regardless of strain rates. For the isotache-type model discussed earlier, the strain rate is directly controlled by the stress-strain state, and there is no transient effect, which corresponds to the case of $\beta = 0.065$ and $\rho_\alpha = 0.017$ (i.e., $\beta = \frac{\rho_\alpha}{\rho_c}$) and the steady state of all cases.

Table 1 summarizes the similarities and differences between the formulations of Yuan and Whittle's MIT-SR model versus Yin and Graham's EVP model as a representative of isotache-type model. The MIT-SR model contains more parameters related to creep, including β for rate-sensitivity, ρ_α for temporal transient behaviour of R_a, and $\dot{\varepsilon}_{ref}$ as the reference strain rate. R_{a0} is an input initial value of R_a. While the isotache-type models have fewer parameters and are easier to use, the MIT-SR model is able to describe the nonlinear transient and overshooting behaviour of clays during the change of strain rates in CRS tests, as shown in the last row of Table 1.

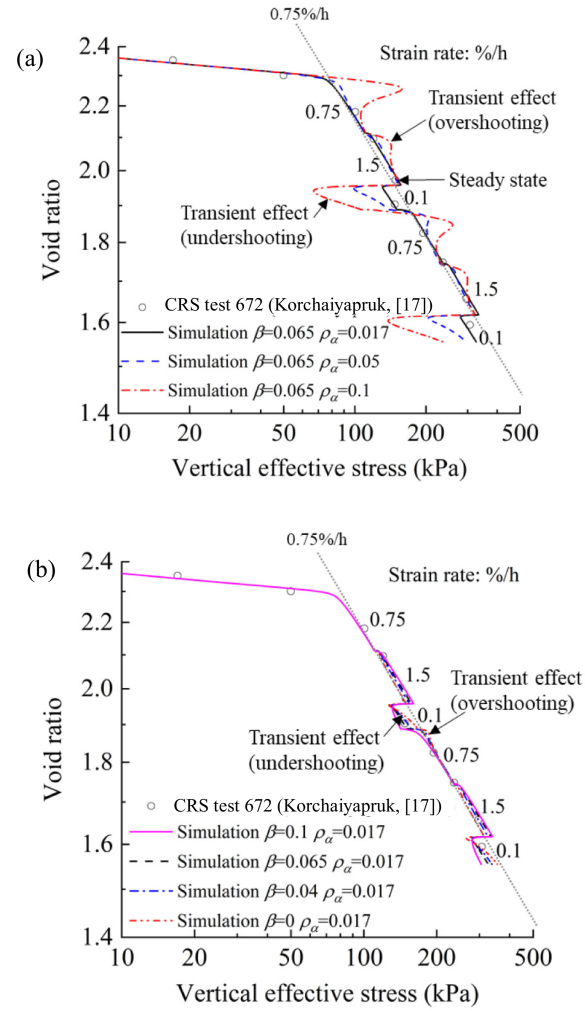
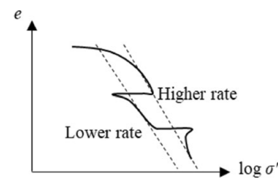
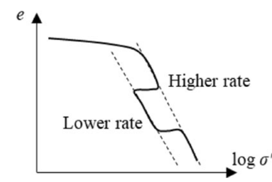


Fig. 6 Parameter studies of a CRS test simulated by the 1D MIT-SR model with **a** different ρ_α and same β; **b** different β and same ρ_α

3 Comparisons of settlement calculations for clays with different thicknesses

The consolidation of clays with different thicknesses is a core issue in studying the time-dependent behaviour of clays. In this section, the interpretations of different

Table 1 Summary of theoretical differences between the two EVP models

Developers and model names	Yuan and Whittle 1D MIT-SR	Yin and Graham 1D EVP
Linearization coordinate	$\log e - \log \sigma'$ coordinate	$e - \log \sigma'$ coordinate
Total strain rate composition	$\dot{\varepsilon} = \dot{\varepsilon}^e + \dot{\varepsilon}^{vp}$	
Elastic strain rate	$\dot{\varepsilon}^e = \rho_r n \cdot \frac{\dot{\sigma}'}{\sigma'}$	$\dot{\varepsilon}^e = \frac{\kappa}{1+e_0}\frac{\dot{\sigma}'}{\sigma'}$
Visco-plastic strain rate	$\dot{\varepsilon}^{vp} = R_a \cdot \left(\frac{\sigma'_z}{\overline{\sigma}'_p}\right)$	$\dot{\varepsilon}_z^{vp} = \frac{\psi}{(1+e_o)t_o} \exp\left[\frac{1+e_o}{\psi}(\varepsilon_{ro} - \varepsilon_z)\right] \left(\frac{\sigma'_z}{\sigma'_{ro}}\right)^{\frac{\lambda}{\psi}}$
Parameters related to creep	$\rho_\alpha, \beta, \dot{\varepsilon}_{ref}$	ψ, t_0
Variables related to initial creep rate	$\sigma'_0, \overline{\sigma}'_p, R_{a0}$	σ'_0, σ'_{p0}
Typical CRS behaviour		

existing models will be compared. The four different models/theories presented in the previous section are adopted for comparison considering their representativeness: (1) the unique EOP method, which is also referred to as "Hypothesis A method", was widely used in conventional design; (2) Yin and Graham's 1D EVP model, an isotache-type model, which is one of the earliest works and convenient to use in both simple methods and numerical simulations; (3) Vermeer and Neher's SSC model, another isotache-type model, which is popular in PLAXIS software; (4) Yuan and Whittle's 1D MIT-SR model, a new non-isotache EVP model.

3.1 Implementation of the theoretical models

3.1.1 Numerical simulation

In our previous simulation of CRS tests, the soil was treated as an element without boundary effects and excess porewater pressure dissipation. To study the consolidation settlement of soils with different thicknesses, hydro-mechanical coupled finite element analyses with different EVP models are implemented. The consolidation governing equations describing creep by incorporating the EVP models developed by Yin and Graham [46, 47] and Yuan and Whittle [50] are composed of a set of partial differential equations (PDEs). To obtain the numerical solution, the coupled governing equations are implemented in the PDE modules of the commercial Multiphysics programme, COMSOL. This software offers a user-friendly interface for solving user-defined PDEs utilizing the finite element method. Within the PDE module of COMSOL, a range of PDE types are available and the coefficient-type PDE is selected to tackle the governing equations. Firstly, through adjustment of the PDE coefficient in COMSOL, the coupled governing equations can be effectively implemented. The initial and boundary conditions are established based on the initial stress state and drained conditions ensuring consistency with the simulated consolidation test. The consolidation governing equations are then discretized using the Galerkin method and solved by applying the differential algebraic equation solver in COMSOL. The SSC model by Vermeer and Neher is available in another commercial finite element programme PLAXIS.

3.1.2 Spreadsheet calculation

The conventional approach based on Hypothesis A and the unique EOP concept is implemented by Excel spreadsheet with Terzaghi's 1D consolidation theory and Eq. (2). For Yin and Graham's 1D EVP model, a spreadsheet calculation method has been developed as well [44, 45]. The method is called a "simplified Hypothesis B method", in which the total settlement of a soil layer under a load increment is calculated as:

$$S_{\text{total}} = US_f + S_{\text{creep}} \tag{11}$$

where U is the average degree of consolidation calculated with Terzaghi's consolidation theory, S_f is the settlement under final effective stress without considering any creep effects of the $e - \log \sigma'_z$ curves, S_{creep} is the creep settlement, which is calculated as:

Table 2 Soil parameters of Ma11 for the three EVP models

Yuan & Whittle, MIT-SR		ρ_c	ρ_r	ρ_α	β	$\dot{\varepsilon}_{ref}$	$\overline{\sigma}'_{p0}$
	(a)	0.3	0.022	0.009	0.03	0.036%/h	700 kPa
	(b)	0.3	0.022	0.015	0.015	0.0072%/h	
Yin & Graham, EVP	λ	κ	ψ	t_0	σ'_{r0}		
	0.37	0.027	0.011	1 day	720 kPa		
Vermeer & Neher, SSC	λ^*	κ^*	μ^*	τ	σ'_{p0}		
	0.154	0.023	0.046	1 day	720 kPa		
Other parameters	e_0	σ'_{z0}	C_k	k_{v0}			
	1.23	489 kPa	1.15	2.55E−10 m/s			

$$S_{\text{creep}} = \begin{cases} \alpha U \cdot \dfrac{\psi}{V} \ln \dfrac{t + t_{ef}}{t_0 + t_{ef}}, \text{ for } t_0 \leq t < t_{\text{EOP}} \\ \alpha U \cdot \dfrac{\psi}{V} \ln \dfrac{t + t_{ef}}{t_{\text{EOP}} + t_{ef}} + (1 - \alpha U) \cdot \dfrac{\psi}{V} \ln \dfrac{t + t_{ef}}{t_{\text{EOP}} + t_{ef}}, \text{ for } t \geq t_{\text{EOP}} \end{cases} \tag{12}$$

where α is an empirical parameter (usually 0.8), t_{ef} is the value of equivalent time at the stress-strain state to S_f under final effective stress, t_{EOP} is the EOP duration in the field, which is the same as t_p.

3.2 Settlement prediction

3.2.1 Thickness effects on consolidation: a numerical illustration

The consolidation problem of Osaka marine clay has been investigated and reported in the literature. With a liquid limit of around 80–100% and plastic limit of 30–40%, a clay fraction up to 40% constituted by smectite, chlorite, kaolin, illite, and mixed layered minerals, the Osaka marine clay exhibited high plasticity and significant time-dependency [39, 41–43, 50]. To investigate the effects of thickness on the consolidation of clays, Watabe et al. [42] conducted inter-connected consolidometer tests on intact Osaka marine clay samples (Ma11) with four different thicknesses 20 mm, 50 mm, 200 mm, and 400 mm. Nevertheless, the original test showed contradictory results, without a clear trend. One of the reasons might be the impurities and inhomogeneity that were inevitable for intact samples. Yuan and Whittle [50] calibrated the parameters for Ma11 based on the results of Watabe et al. [42]. Two sets of parameters were obtained based on two different specimens. Here only one set of parameters based on the 20-mm-thickness specimen is used for comparison, as summarized in Table 2 (case a). The parameters can be translated to relevant parameters in other models. For Yin and Graham's EVP model, the soil parameters can be calculated as:

$$\frac{\kappa}{V} = \rho_r \cdot n, \ \frac{\lambda}{V} = \rho_r \cdot n, \ \frac{\psi}{V} = \rho_r \cdot n, \ \sigma'_{r0} = \overline{\sigma}'_{p0} \left(\frac{\overline{\sigma}'_{p0}}{\sigma'_{z0}} \right)^{\frac{\rho_r}{\rho_c - \rho_r}} \tag{13}$$

where $n = \frac{e}{1+e}$ is the porosity of the soil and can be estimated as the constant initial void ratio $e = 1.23$ for the 20-mm-specimen in Watabe et al. [42]. For Vermeer and

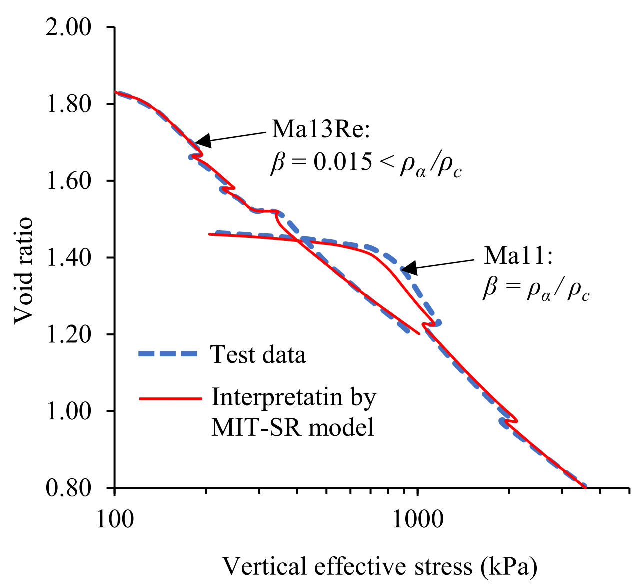


Fig. 7 Calibration of β in MIT-SR model for Ma11 and Ma13Re from CRS tests (after Yuan and Whittle [50])

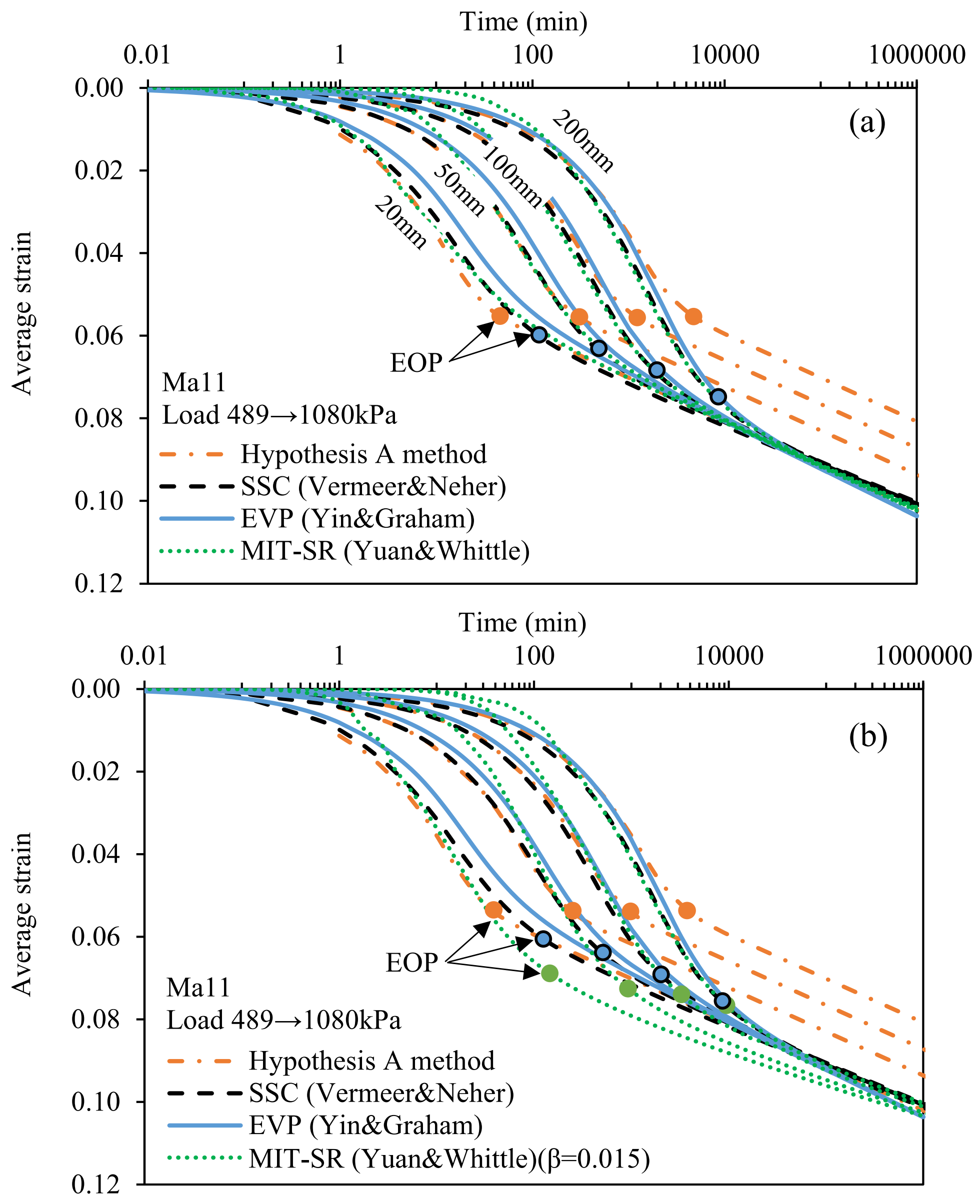


Fig. 8 Simulation of consolidation settlement for intact Osaka clay with different thicknesses by different methods: **a** original case $\beta = \rho_\alpha/\rho_c$; **b** parametric case $\beta = 0.015 < \rho_\alpha/\rho_c$

Neher [40]'s SSC model, the soil parameters can be calculated as:

$$\kappa^* = 2\rho_r \cdot n,\ \lambda^* = \rho_c \cdot n,\ \mu^* = \rho_r \cdot n,\ \sigma'_{p0} = \overline{\sigma}'_{p0}\left(\frac{\overline{\sigma}'_{p0}}{\sigma'_{z0}}\right)^{\frac{\rho_r}{\rho_c-\rho_r}} \tag{14}$$

For the old Hypothesis A method or the unique EOP theory by Mesri et al., the settlement curves were computed using Terzaghi's 1D consolidation equation with a constant compressibility m_v for different thicknesses. The value of m_v is calculated from the 20-mm-specimen in Watabe et al [42]. The strain increment under loading increment from 489 to 1080 kPa is around 0.06, which yields $m_v = \frac{\Delta\varepsilon_z}{\Delta\sigma'_z}$=0.000102 kPa^{-1}. The secondary compression coefficient C_α is determined as $C_\alpha = \rho_\alpha \cdot e \cdot \ln 10$ (Yuan and Whittle [50]). According to the CRS test, Ma11 exhibits isotache-type behaviour and $\beta = \frac{\rho_\alpha}{\rho_c}$, as shown in Fig. 7. Other parameters are the same as Yuan and Whittle [50], as listed in Table 2. The 1D MIT-SR and 1D EVP models are both implemented by COMSOL, while the SSC model is implemented in PLAXIS. The Hypothesis A method is used with Excel spreadsheets.

Figure 8a shows the simulation results of clay consolidation with different models, including the case (a) for MIT-SR model. It can be found that all three EVP models produce highly consistent results with the parameters in

Table 3 Soil parameters of Ma13Re for the three EVP models

Yuan & Whittle, MIT-SR	ρ_c 0.203	ρ_r 0.052	ρ_α 0.008	β 0.015	$\dot{\varepsilon}_{ref}$ 0.012%/h	R_{a0} 0	$\overline{\sigma}'_{p0}$ 115 kPa
Yin & Graham, EVP	λ 0.32	κ 0.104	ψ 0.0085	t_0 1 day	σ'_{r0} 116 kPa		
Vermeer & Neher, SSC	λ^* 0.106	κ^* 0.02	μ^* 0.0028	τ 1 day	σ'_{p0} 116 kPa		
Other parameters	e_0 1.905	σ'_{z0} 88	C_k 1.15	k_{v0} 8.57E−10 m/s			

Table 3. The EOP strains increase with thickness, and exhibit similarity for three EVP models, as marked in the figure. The simulation results validate the theoretical statement in Yuan and Whittle [50] that MIT-SR model is consistent with isotache-type models with $\beta = \frac{\rho_\alpha}{\rho_c}$. In contrast, the Hypothesis A method predicts EOP strain independent of thickness and the strain-time curves parallel after EOP.

Yuan and Whittle [50] indicated that the value of β which can be smaller than $\frac{\rho_\alpha}{\rho_c}$, and showed an extreme case with $\beta = 0$ for Osaka Clay exhibiting identical EOP strain. However, according to Yuan [52], the values of β for several clays worldwide are all larger than 0.01, and it is $\beta = 0.015$ for reconstituted Osaka Bay clay (Ma13Re), as shown in Fig. 7. In this study, $\beta = 0.01\,5$ is used as another typical case to compute the consolidation settlement of the Osaka clay. According to the MIT-SR model, the creep of soil is influenced by $\beta, \rho_\alpha, \dot{\varepsilon}_{ref}$. If $\beta \neq \frac{\rho_\alpha}{\rho_c}$, the other parameters should be re-estimated by matching the test data. In this study, it was found that adopting $\rho_\alpha = 0.015$ and $\dot{\varepsilon}_{ref} = 0.0072$ %/h can generally match the consolidation curves. The simulation results based on this set of parameters are shown in Fig. 8b. It is found that the consolidation curves by MIT-SR model deviate from the other two EVP models, seeing a slower convergence trend. However, the EOP strains are still influenced by the thickness of the soils, and the settlement curves of different thicknesses gradually join into one line with increasing time. The simulations on the two cases show that for typical cases, the non-unique EOP behaviour is obvious in EVP modelling.

3.2.2 Consolidation analysis for a thick reconstituted clay specimen

The data on Ma11 are not used to verify any of the methods because they did not show a consistent trend that existing methods can explain with a consistent parameter set. The reason is unclear but can be related to the quality of natural samples. Considering the size of the specimens, the existence of possible impurities cannot be neglected. Fortunately, Watabe et al. [42] reported a consolidation test on reconstituted Osaka marine clay (Ma13Re) specimens, which should be much more uniform and properly controlled compared to the natural ones. In this study, the data of Ma13Re will be used to verify the existing model predictions. The parameters of the Ma13Re can be determined by a multistage oedometer test [43] and a CRS test [39] on the same soil.

The values of β and ρ_α in MIT-SR model have been calibrated by Yuan [52] from the CRS test on MA13Re by Tsutsumi and Tanaka (2012), as listed in Table 3. It can be found that different from the intact Ma11, the fitted values of β of Ma13Re is not equal to $\frac{\rho_\alpha}{\rho_c}$. Other parameters including those for the EVP and SSC models are calibrated from an oedometer test reported by Watabe et al. [43]. The results of this oedometer test and the $e - \log \sigma'_z$ curve at both EOP and 24 h are replotted in Fig. 9. The pre-consolidation pressure σ'_{p0} was 134 kPa for the oedometer specimen and 116 kPa for the 100 mm specimen [42, 43]. Based on the 1-day compression curve, the normal compression line λ and the slope of over-consolidation line κ from 88 to σ'_{p0} can be fitted. The creep coefficient ψ is fitted using the $e - \ln t$ relations at the secondary compression stage of the specimen under 275 kPa. The EOP void ratio-stress curve was used to determine the strain increment and compressibility for the Hypothesis A method. The permeability k_v is assumed to follow a correlation with void ratio: $k_v = k_{vi} 10^{\frac{e-e_i}{C_k}}$. Since the permeability of Ma13Re was not measured by Watabe et al., it is assumed here based on the measured data of Ma11, with $k_{vi} = 2.55 \times 10^{-8}$cm/s, $C_k = 1.15$, $e_i = 1.3$ based on Yuan and Whittle [50].

Figure 10 shows the calculated settlement curves of Ma13Re using different methods and models. The old A method using unique EOP strain underestimates the settlement. Using three EVP models and the simple B method with parameters calibrated from the oedometer or CRS tests, the prediction settlement curves all fit well with the

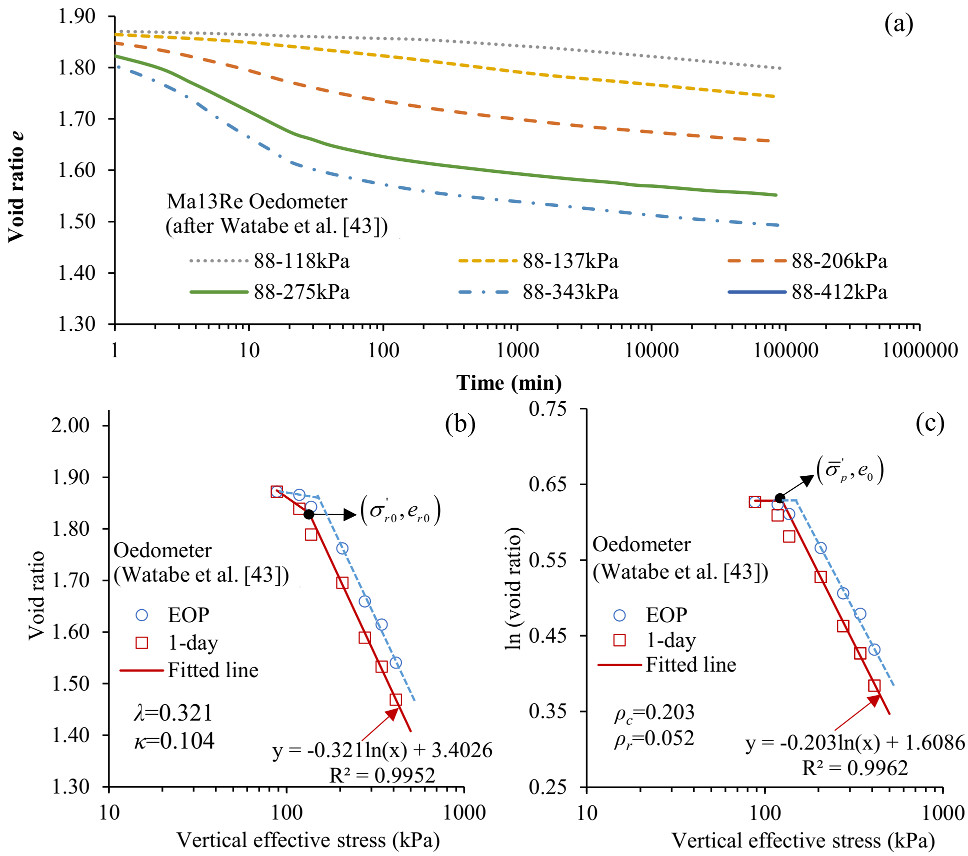


Fig. 9 1D compression curves of Ma13Re (20 mm oedometer specimen) for model calibration: **a** in $e - \ln t$ plane; **b** in $e - \ln \sigma'_z$ plane; **c** in $\ln e - \ln \sigma'_z$ plane (after Watabe et al. [43])

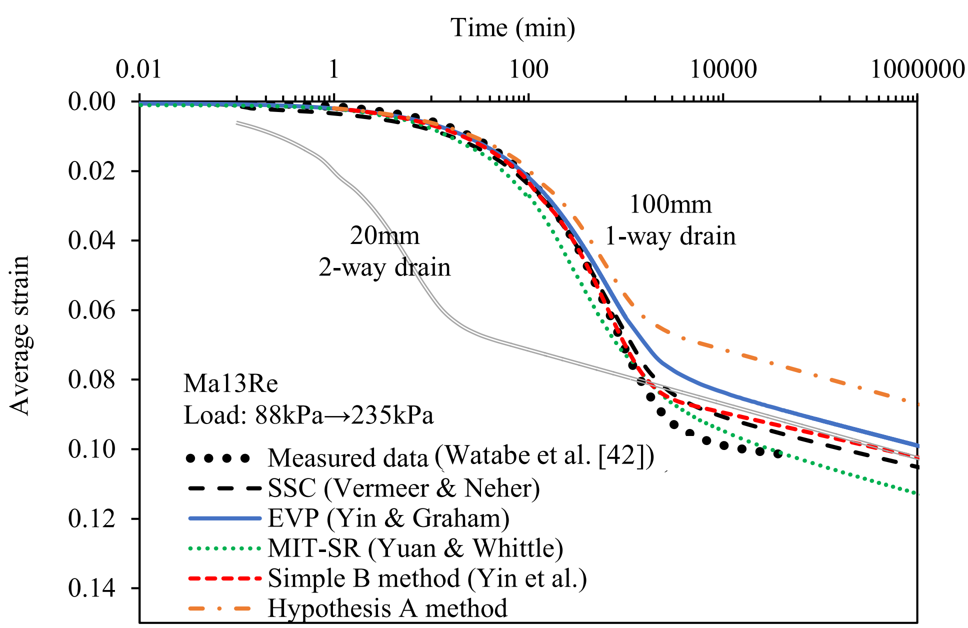


Fig. 10 Simulation of consolidation settlement for Ma13Re with different methods

measured data. Some differences exist in the prediction results, which are probably due to the different formulations and numerical tools among the models. For example, the MIT-SR uses logarithmic void ratio, the SSC model uses logarithmic strain, while Yin & Graham's EVP model uses engineering strain. The Yin & Graham's EVP model and MIT-SR model are implemented by COMSOL, while the SSC is implemented by PLAXIS.

Although the loading scheme of the oedometer test was different from the 100mm consolidation test, the settlement curves of Ma13Re under oedometer set-up (20 mm, 2-way drainage) can still be estimated using Terzaghi's equation, and plotted in Fig. 10. For the standard oedometer specimen, Hypotheses A and B should yield similar results. Comparing the settlements of two thicknesses, the EOP strain of the 100 mm specimen is significantly larger than the EOP strain for the oedometer specimen. The MIT-SR model slightly overestimates during primary consolidation but offers the best matching with measured data at the secondary compression stage. The test results on Ma13Re and all EVP models' predictions are in line with Hypothesis B.

4 Discussions on the key questions about consolidation and creep

With a number of methods and models developed in the literature, they have provided different answers to some well-known and long-history questions, including the famous debate between "Hypothesis A" and "Hypothesis B". The debate is concerned with a series of questions, which were frequently inconsistently interpreted.

4.1 Whether creep happens in primary consolidation

To model the time-dependent compression of clays, the first fundamental problem is to answer whether creep can happen during the "primary" consolidation. Primary consolidation refers to the process in which excess porewater pressure of a soil layer dissipates with time until it gets zero. In many old design methods, such as the FHWA-NHI-06-088 in USA [35], the settlement was divided into primary consolidation and secondary settlement. The primary consolidation settlement was analysed directly using the compressibility measured on thin specimens in the laboratory, as indicated in Fig. 1. The secondary compression due to creep after EOP is calculated using the same method as Eq. (2), while creep during the primary consolidation is not calculated. Under the influences of this method, it was frequently interpreted by many engineers and researchers that creep does not exist before EOP.

Although the old Hypothesis A method was still popular with engineers due to the convenience of usage, nowadays more researchers agree or imply that the effects of creep, or visco-plastic deformation, should be considered during and after primary consolidation [3, 9, 14, 15, 22, 29, 38, 46, 50]. It was also pointed out that assuming no creep during primary consolidation violates the continuum mechanics [15, 36]. In general, creep can be deemed as a result of structure viscosity of the soil skeleton or other mechanisms in the microstructures [21, 33], which is coherent during the whole consolidation process, instead of just happening after EOP, as indicated in Eq. (1). As shown in Fig. 6 and Table 1, both isotache- and non-isotache-type models consider the rate-dependent behaviour during CRS consolidation process.

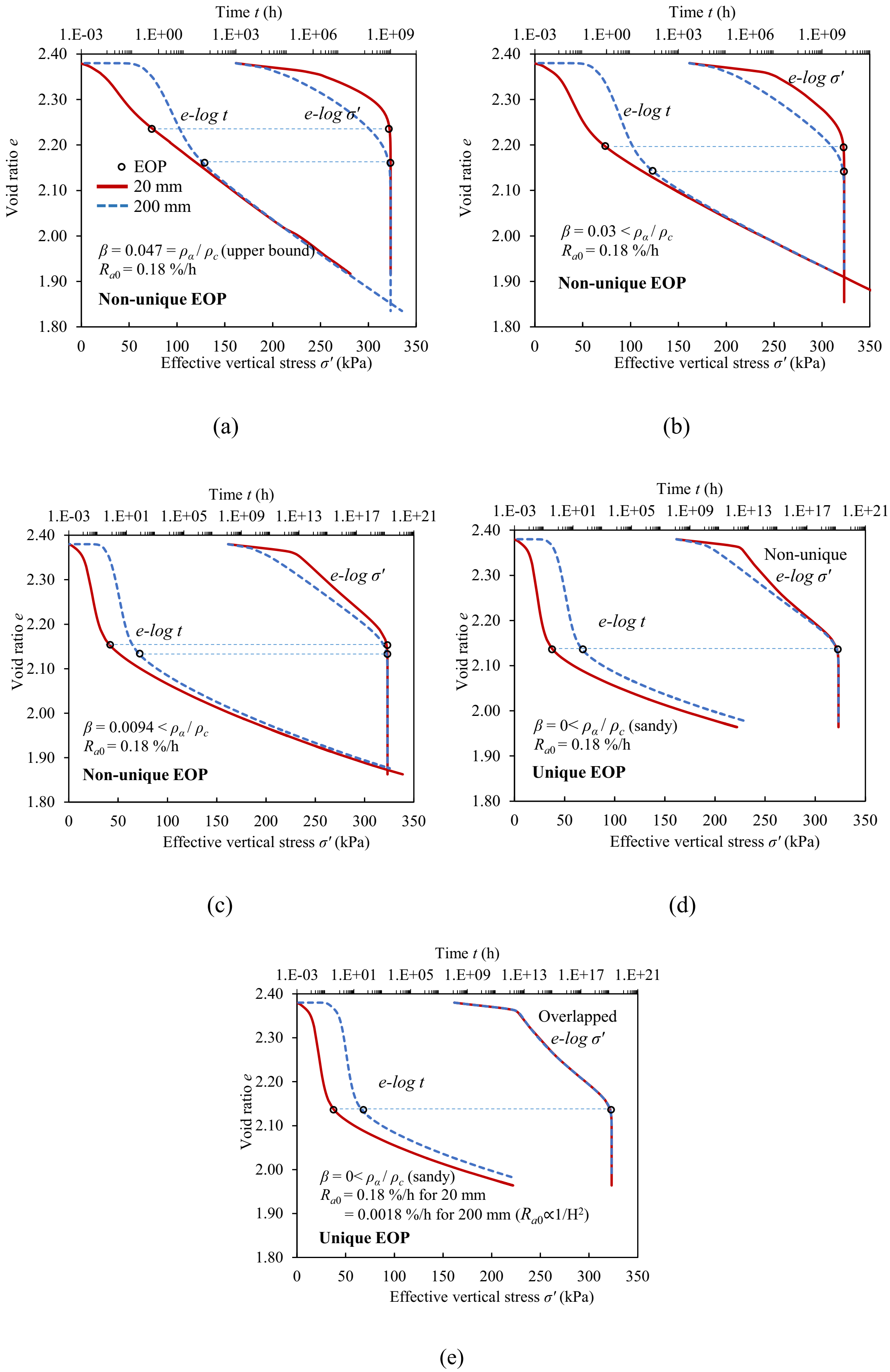
4.2 Whether creep is a separate phenomenon during primary consolidation

Ladd et al. [19] first questioned "whether creep acts as a separate phenomenon while excess pore pressures dissipate during primary consolidation", and it was interpreted that the "rheology models" considered creep as a separate phenomenon due to the structural viscosity of soils. In the representative EVP (rheology) models discussed in previous sections, creep is a spontaneous behaviour caused by the visco-plastic property of clays, but is also dependent on the effective stress, void ratio [40, 46], and the history of strain rates [50], which is not totally isolated during the primary consolidation. In the unique EOP theory, the creep rate must be associated with the degree of consolidation to achieve a unique EOP strain [31]. Both the rheology models and unique EOP theories considered creep dependent on the consolidation process. A more precise statement may be "rheology models consider creep as a spontaneous behaviour, which is not controlled by EOP but dependent on the consolidation process".

4.3 Whether EOP strain relationship is thickness-dependent

4.3.1 Interpretation of unique EOP theory and isotache-type models

Some researchers insist that despite creep may exist during primary consolidation, it will not affect the unique EOP stress-strain relations of soils [27, 29, 31]. If this hypothesis stands as a general rule, there will be no need to analyse the creep compression during the primary consolidation, and the EOP strain of clays with different thicknesses is identical under the same initial state and load increment, as shown in Fig. 8. Therefore, it predicts the same results with the simple hypothesis that creep does not exist during


Time t (h)
Void ratio e
Effective vertical stress σ′ (kPa)
e-log t
e-log σ′
EOP
20 mm
200 mm
β = 0.047 = ρα / ρc (upper bound)
Ra0 = 0.18 %/h
Non-unique EOP
(a)
β = 0.03 < ρα / ρc
Ra0 = 0.18 %/h
Non-unique EOP
(b)
β = 0.0094 < ρα / ρc
Ra0 = 0.18 %/h
Non-unique EOP
(c)
Non-unique
e-log σ′
β = 0< ρα / ρc (sandy)
Ra0 = 0.18 %/h
Unique EOP
(d)
Overlapped
e-log σ′
β = 0< ρα / ρc (sandy)
Ra0 = 0.18 %/h for 20 mm
= 0.0018 %/h for 200 mm (Ra0∝1/H2)
Unique EOP
(e)

◀**Fig. 11** Simulation of consolidation settlement by 1D MIT-SR model with different values of β *and* R_{a0}: **a** $\beta = 0.047 = \rho_\alpha/\rho_c$; **b** $\beta = 0.03 < \rho_\alpha/\rho_c$; **c** $\beta = 0.0094 < \rho_\alpha/\rho_c$; **d** $\beta = 0 < \rho_\alpha/\rho_c$; **e** $\beta = 0 < \rho_\alpha/\rho_c$ with R_{a0} proportional to $1/H^2$

primary consolidation. If this is true, the determination of EOP will be a key factor in the creep estimation. It still needs clarification why the visco-plastic compression of the soil is dependent on the thickness of the soil layer. Besides, since the consolidation near EOP becomes extremely slow, the estimated value of EOP period naturally contains a large margin of error. Furthermore, it was also noted that some researchers [7–9, 36] have re-analysed a number of test data reported in the literature considering the stress-strain histories before the incremental loading. The results showed that many consolidation test data that were considered to reveal the unique EOP behaviour actually inclined towards non-unique EOP or the isotache-type models.

In contrast, some researchers concluded that if creep exists within primary consolidation, it will contribute to the total compression of soils before EOP and should be modelled before EOP. Therefore, the EOP strain of the same soils is dependent on the t_p. The larger the thickness of clay, the larger the value of t_p, and the larger the EOP strain. Such behaviour is predicted by the EVP model by Yin & Graham and the SSC model, as shown in Figs. 8, 9.

4.3.2 Interpretation of MIT-SR model

Special attention is given to the prediction results of the non-isotache MIT-SR model by Yuan and Whittle [50]. Figure 11 shows the simulation results by MIT-SR model considering different values of parameters β and ρ_α. The soil is an NC clay and the parameters follow the simulation by Yuan and Whittle [50], but the values of β and ρ_α are changed. It can be found that the configurations of consolidation curves are evolving with β and ρ_α adopted. In Fig. 11a, when the upper bound $\beta = \frac{\rho_\alpha}{\rho_c}$ is adopted, the EOP strain significantly increases with soil thickness, and the consolidation settlement curves converge quickly after EOP, with the same pattern as the isotache-type models. In Fig. 11b-c, as β deviates from $\frac{\rho_\alpha}{\rho_c}$ and decreases, the convergence point is delayed, the secondary compression slope becomes smaller, and the difference of EOP strains becomes smaller. However, as long as $\beta > 0$, including in the cases of Fig. 8, the strain-time curves of different thicknesses always converge, and the EOP strain always increases with thickness. For the extreme case of Fig. 11d where $\beta = 0$, the EOP strains of different thicknesses are the same, and the secondary compression curve seems non-

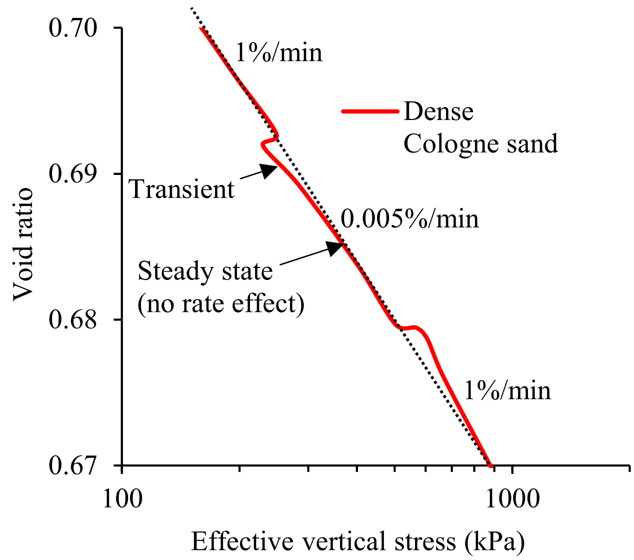


Fig. 12 CRS test results of a dense sand specimen exhibiting transient but non-isotache behaviour (after Levin et al. [24])

converged after a long period. Such results look similar to the predictions by the unique EOP theory and the Hypothesis A method.

However, $\beta = 0$ suggests that in CRS tests, the soil would not exhibit rate sensitivity at steady state. According to the calibration results on various types of reconstituted and intact clay samples from Boston, San Fransisco, and Osaka [50, 52], the values of β ranged between 0.01 and $\frac{\rho_\alpha}{\rho_c}$, but none reached zero. Therefore, the case of $\beta = 0$ in Fig. 11d is unlikely to be suitable for clays. Besides, the $e - \log\sigma'$ curves during the consolidation are influenced by the value of R_{a0}, as shown in Fig. 11d-e. Under a special case, when R_{a0} of 200-mm-specimen is 1/100 R_{a0} of 20 mm-specimen (i.e., $R_{a0} \propto \frac{1}{H^2}$), their $e - \log\sigma'$ relations overlapped. However, such results do not indicate the absence of creep during primary consolidation. If other values of R_{a0} is used, the $e - \log\sigma'$ curves will exhibit dependency of thickness, which differs from the old Hypothesis A method.

Therefore, results like Fig. 11d-e are limited to extreme conditions, and there is no such evidence showing $\beta = 0$ from laboratory test data on clays. In fact, one might note that the case of $\beta = 0$ coincides with the compression behaviour of sands, which usually exhibit temporal effects but no isotache-type behaviour [2, 20, 25]. Figure 12 shows the compression curves of dense Cologne sand subjected to CRS tests [23, 24]. It can be found that for the sand, the configuration of compression curves is similar to the case $\beta = 0$ in Fig. 6b. The steady-state stress-strain curve of the sand is not sensitive to strain rates. However, it did exhibit temporal effects, including the transient effect during the changes of compression rate, as well as the creep

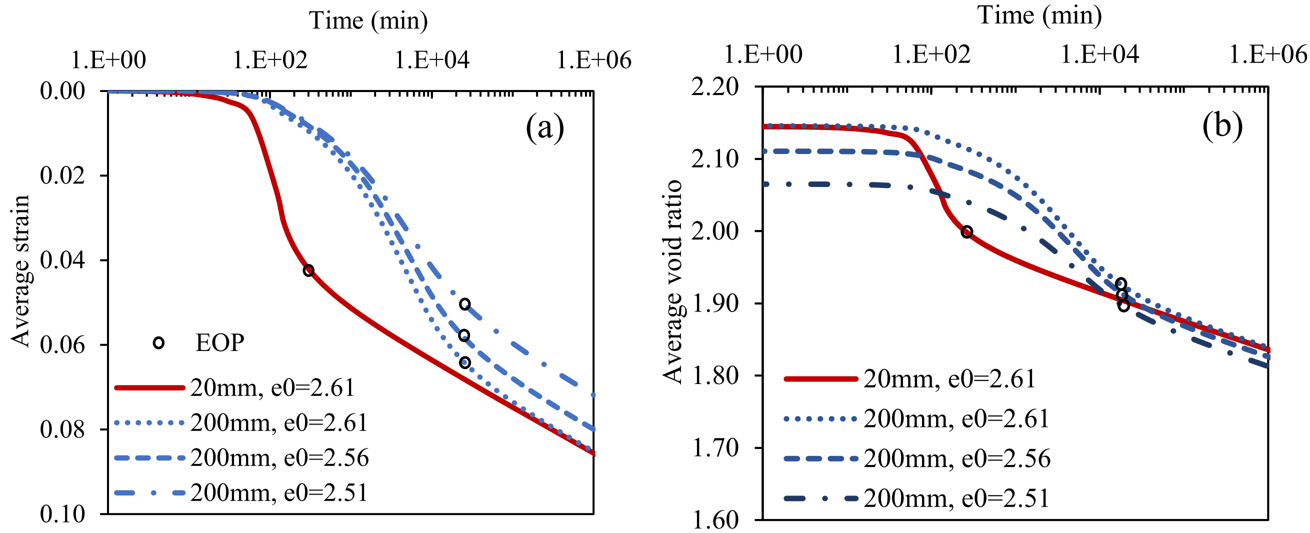


Fig. 13 Simulation of consolidation settlement by SSC model with different initial void ratios: **a** by incremental strain; **b** by void ratio

compression under constant loading and therefore $\rho_\alpha > \beta = 0$. In this regard, the MIT-SR model has achieved remarkable advances in modelling some time-dependent behaviour of different types of soils that conventional isotache-type models could not explain.

4.3.3 Interpretation with variable initial conditions

The above discussion is based on the premise that the soils with different thicknesses are uniform with the same initial conditions, such as the initial void ratio. If the initial conditions are different, the compression under the same loading will be different. Figure 13 shows the results of SSC modelling on the same clay with the initial void ratio (e_0) of the 200-mm-specimen varying between 2.51, 2.56, and 2.61. It can be found that in terms of vertical strain, the consolidation curves do not converge and exhibit some similar pattern as the curve A in Fig. 2, although the void ratio is changed by only 0.05–0.1. As soil is a natural material, such problem could have occurred in previously reported tests. The inconsistent initial void ratio can be another interpretation of test results that were similar to "curve A" in Fig. 2. But in terms of void ratio or accumulative strain, the isotache-type models will predict "curve B" only. Previous researchers have successfully used EVP models to explain this issue [7, 36].

4.3.4 Non-uniqueness of EOP and convergency of secondary compression curves

In Fig. 2, the "curve A" has a unique EOP strain and parallel secondary compression curves after EOP, while the "B curve" exhibits non-unique EOP together with immediately converging secondary compression curves with different thicknesses. It should be noted that in some cases, non-unique EOP may not fully correspond to immediately converging secondary compression curves after EOP. For example, in the predictions by MIT-SR in Fig. 11b-d, the EOP void ratios are apparently different for different thicknesses, but the secondary compression curves do not converge immediately as described in Fig. 2. In Fig. 13, the soils with different initial void ratios have non-converging secondary compression curves after EOP, but the EOP strains are still different.

4.4 Hypothesis A or Hypothesis B: a revisit

4.4.1 Current contradictions

As discussed previously, Ladd et al. [19] stated that if creep is a separate phenomenon due to the structural viscosity of soils, the EOP strain will be dependent on the primary consolidation duration. Otherwise, the EOP strain will be unique. A conceptual figure was presented with two "extreme curves", in which "curve A" describes unique EOP while "curved B" represents the results from rheologic models with increasing EOP strain with increasing thickness, as shown in Fig. 2. However, Ladd et al. [19] did not provide a direct definition of Hypothesis A and Hypothesis B.

After Ladd et al. [19], the terms "Hypothesis A" and "Hypothesis B" were firstly defined by Jamiolkowski et al. [13], co-authored by Prof. Ladd. It was concluded that Hypothesis A assumes creep occurring only after primary consolidation, while Hypothesis B assumes creep occurring during pore pressure dissipation due to structural viscosity. Such interpretations were then widely used [14, 22, 42, 45, 48], which appeared to be a very simple criterium to define these two hypotheses. With this definition, the procedure used by the conventional design method [35] corresponds to Hypothesis A, while the methods based on EVP (rheology) models by Leroueil et al. [22], Kavazanjian et al. [15], Yin and Graham [46–48], Vermeer and Neher [40], Degago et al. [9], Yuan and Whittle [50], all correspond to Hypothesis B.

Nevertheless, there was another interpretation of Hypotheses A and B, based on whether or not the EOP strain is unique for the same soils with different thicknesses [9, 36]. According to Szavits-Nossan [36], both Hypotheses A and B now accepted that creep can occur during primary consolidation, but Hypothesis A advocates unique EOP strain while Hypothesis B does not. For example, despite acknowledging creep during primary consolidation, the theories advocating unique EOP stress-strain relationship by Mesri et al. [30, 32] are also categorized and claimed to be Hypothesis A. With this hypothesis, the creep deformation during primary consolidation need not be calculated, which produces the same results as the simple assumption that creep does not exist before EOP. On the contrary, Hypothesis B, represented by most EVP models, considers that the EOP strain is not unique and will be affected by the creep compression during primary consolidation. Before the invention of MIT-SR model, the two definitions converged and their differences were seldom discussed.

As discussed previously, the MIT-SR model considers creep during primary consolidation, but can simulate "curve A" with unique EOP strain when $\beta = 0$ as shown in Fig. 11e, although $\beta = 0$ is more suitable for sands. Yuan and Whittle [50] commented that the case $\beta = 0$ corresponds to Hypothesis A behaviour while $\beta = \frac{\rho_\alpha}{\rho_c}$ corresponds to Hypothesis B behaviour, as shown in Fig. 11a. According to the simulations in this study, when $\beta > 0$ for general clayey soils, including Osaka clay, the predictions are all supportive of non-unique EOP, as shown in Fig. 11a-c. Even in the case with a very small value of β (around 0.01) in Fig. 11c, the settlement curves of the clays with different thicknesses exhibit a slower converging trend compared to $\beta = \frac{\rho_\alpha}{\rho_c}$, but still indicate "non-unique EOP" and converging trend. If the criterion of "whether EOP is unique" is followed, it seems that the MIT-SR model corresponds to Hypothesis B for clayey soils (Fig. 11a-c) but inclines towards Hypothesis A for sands (Fig. 11d). Therefore, almost all cases ($0 < \beta \leq \frac{\rho_\alpha}{\rho_c}$) for clays of MIT-SR model predictions indicate Hypothesis B, not limited to the single case of $\beta = \frac{\rho_\alpha}{\rho_c}$ in Fig. 11a.

With the development of EVP modelling from Bjerrum [3] to Yuan and Whittle [50], researchers have a growing understanding of the creep behaviour of clays. However, it can be found that there exist some contradictions regarding the usage of Hypothesis A and Hypothesis B in the literature, and such inconsistent interpretation may confuse or even mislead engineers.

4.4.2 A unified interpretation: hypothesis, methodologies, and phenomena

Figure 14 shows the relationships between the key concepts in the modelling of soil creep, which is associated with three levels: hypotheses, methodologies, and phenomena. The first set of hypotheses concerns whether creep can happen during primary consolidation. The second set of hypotheses concerns whether creep during primary consolidation can affect the EOP stress-strain relationship. Combining the current understandings among researchers reviewed before, Hypothesis A should refer to the assumption that the EOP compression is unique and uninfluenced by creep, while Hypothesis B should refer to the idea that the compression during the whole process can be influenced by creep, to different degrees dependent on the specific viscosity parameters. Hypothesis A requires t_p as a parameter while Hypothesis B does not. Therefore, the three EVP models discussed here can be categorized to Hypothesis B.

For the methodologies, Hypothesis A is achieved by a unique EOP theory with the secondary compression calculated by Eq. (2). Separation of primary and secondary consolidation is needed in this method, which implies that creep does not exist or need not be considered during primary consolidation. Hypothesis B is achieved by hydro-mechanical coupled analysis using an elastic visco-plastic model. Different models were developed from the relatively simple isotache-type models [40, 46] to the more complex model by Yuan and Whittle [50]. These EVP models are developed based on continuum mechanics, without direct assumptions on the position of EOP, and could be referred to as "rigorous" Hypothesis B [44].

Different results can be described by different methods and hypotheses. For the old Hypothesis A method, it is always predicted that the EOP strain of the same clay with different thicknesses is a constant, and that the secondary compression curves are parallel. For the EVP models, the EOP void ratio is dependent on many factors, including the soil parameters (such as β, ρ_α, ψ), the thickness, and the duration for primary consolidation. When $\beta = 0$ in the MIT-SR model, it can predict unique EOP strain, which however is an extreme case that probably only works for sands.

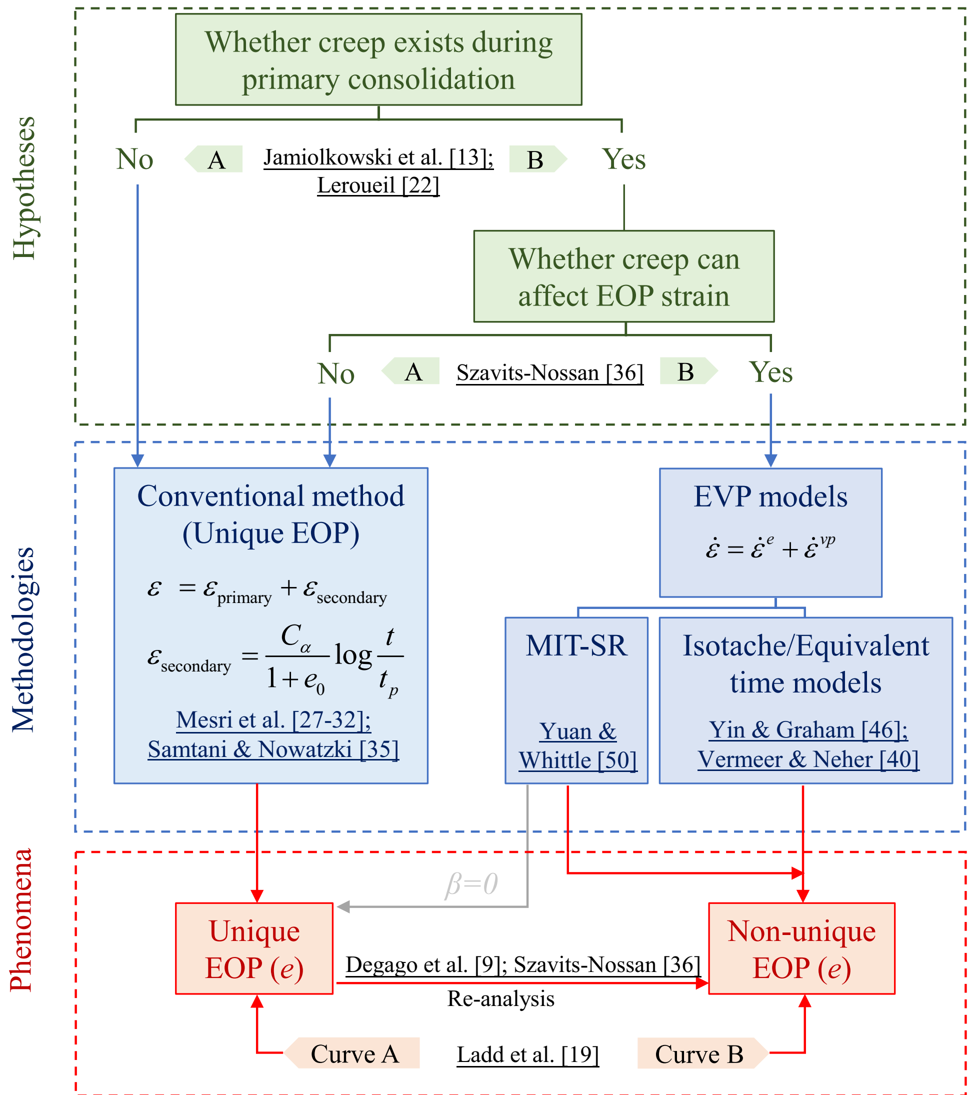


Fig. 14 Correlations between different concepts related to consolidation and creep of clays

4.4.3 Suggestion for design practice

Although the complex mechanisms of clay creep are not fully understood yet, it has gradually been an academic consensus on using elastic visco-plastic models for clay behaviour [50]. There are two reasons to improve the old Hypothesis A method in design codes.

Firstly, from the experimental side, the interpretations of many consolidation settlement data supporting unique EOP have been challenged and re-interpreted to show the contrasting behaviour after considering the different initial conditions [9, 36]. From the interpretation by MIT-SR model, clayey soils tend to have non-zero β, which is also in line with the Hypothesis B.

Secondly, from the theoretical side, existing EVP models have been developed based on continuous mechanics, with much more versatility in describing the behaviour of clays in various hydro-mechanical conditions, without estimation of t_p and subjective assumptions on EOP strain. For simple analysis, the de-coupled simplified Hypothesis B method has been validated with a number of real cases, showing comparable accuracy [44, 45]. In recent years, several countries like Canada and Norway have updated their design guidelines to include EVP modelling and Hypothesis B in the settlement analysis of clays [6, 34]. Among three models, the isotache-type models by Vermeer and Neher [40], Yin and Graham [46–48] (including the simple B method [44, 45]) have simpler forms and are easier to use, while the MIT-SR model [50] contains more versatility but also more complicated implementation.

5 Conclusions and suggestions

In this study, a comparison is conducted among four existing methods for modelling the 1D consolidation of clays exhibiting creep: the unique EOP method, Yin and Graham's 1D EVP model, Vermeer and Neher's SSC model, Yuan and Whittle's 1D MIT-SR model. Numerical simulation results on typical cases are presented and key issues of researchers' concern are discussed and revisited. The major conclusions are summarized below.

(a) Most researchers agree that clays are viscous materials and that visco-plastic compression exists during the consolidation process. Creep is not an isolated phenomenon but is coupled with the dissipation of excess porewater pressure.

(b) The terminologies of Hypothesis A and Hypothesis B have been frequently abused, and therefore a more precise definition is required. Hypothesis A is an assumption that the relationship between EOP void ratio and effective stress is unique and uninfluenced by the creep of soils. Hypothesis B considers that the visco-plastic compression of clays can affect the EOP strain and should be considered during primary consolidation.

(c) For typical clayey soils, the interpretations of thickness effects by existing EVP models (including the isotache-type models and the MIT-SR model) are in line with Hypothesis B, with non-unique EOP behaviour. Hypothesis B is not limited to an "extreme" case or "isotache" models only.

(d) All three EVP models have good performance in calculating the settlements of clays. The two isotache-type models are easier to implement, while the MIT-SR model might be more versatile in describing the creep behaviour.

(e) Existing EVP models can partly explain the phenomenon that consolidation settlements exhibit similar EOP or non-converging trends with different thicknesses, without assuming unique EOP in Hypothesis A. In the MIT-SR model, a smaller β will cause the slower convergence of settlement curves, and the extreme case β=0 will generate consolidation curves with unique EOP, although it may be suitable for sands only. In all EVP models, the in-consistency of the initial void ratio of clay can also explain the consolidation settlement curves with different thicknesses exhibiting non-converging trends.

(f) The old Hypothesis A method stands on a weak foundation in both experimental and theoretical interpretation. It is necessary and convenient to adopt visco-plastic models for calculating clay settlement for safer design rather than the old method.

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Author contributions Chen ZJ, Li PL and Song DB contributed to the conception, analysis, and interpretation of the data and drafted work. Yin JH and Wu PC revised the manuscript and contributed to the interpretation of the data.

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Data availability No datasets were generated or analysed during the current study.

Declarations

Competing interests The authors declare no competing interests.

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