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A review of isotache modeling and secondary consolidation behavior of soft clays

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Abstract. Secondary consolidation of soils has gained much importance as a major contributor in the long term settlements of soft clays. In the scenario where constructions on such problematic soils are increasing, accurate prediction of long term settlements is very challenging. This paper reviews the evolution of the secondary compression characteristics from the concept of $C\alpha$ to the creep isotaches. The generation of the 3 D elastic viscoplastic model from the equivalent time (isotache) approach is also discussed in this paper. This model is an extension of Maxwell's rheological model. With relevance to Indian soils, the secondary compression characteristics of Kuttanad and Cochin marine clay are also reported in this paper.

1. Introduction

Normally consolidated aged clay is a term used for 'young' clays that continue to settle when left under constant effective stress for hundreds or thousands of years. This settlement is referred to as secondary or delayed consolidation and results in a more stable configuration, i.e. greater strength and reduced compressibility [2]. A quasi preconsolidation pressure or an increased yield stress due to secondary consolidation effects, which is greater than the maximum past effective stress, is observed in such clays. This is seen as p_c in figure 1.

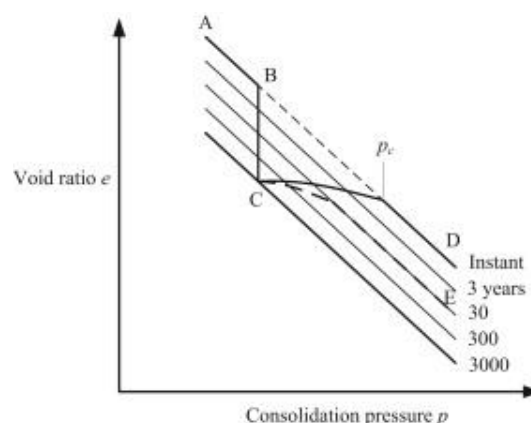


Figure 1. Quasi preconsolidation pressure due to secondary consolidation effect [2]



There are three main approaches for the practical and the theoretical evaluations of the consolidation settlement, and these can be listed as follows:

- (i) the coupling of Terzaghi's one-dimensional consolidation theory and the constant C_{ae} concept,
- (ii) the end of the primary consolidation (EOP) concept [6] and the constant C_{ae}/C_c concept[8],
- (iii) the isotache concept introduced by Šuklje in 1957 (as reported in[12])

Where C_{ae} is the coefficient of secondary compression and C_c is the compression index

1.1 Terzaghi's one dimensional theory and the constant C_{α} approach

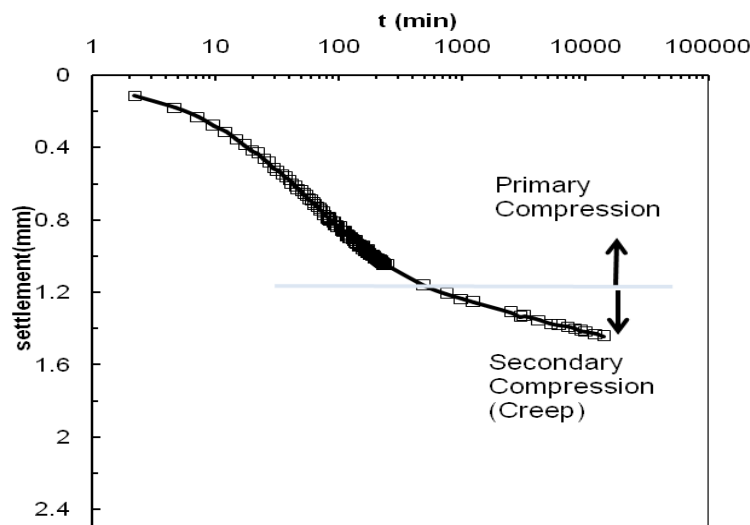


Figure 2. Typical time-settlement response of clay with large secondary compression

One of the first computations in settlement due to secondary compression is using the secondary compression index C_{α} , which is the slope of the e v/s $\log t$ curve in the secondary compression range. The time settlement curve is divided into two portions i.e. primary and secondary compressions; secondary compressions assumed to begin at the end-of-primary compressions (Fig. 2). For a constant C_{α} between t_p and t , the settlement can be written as

$$S = \frac{C_{\alpha}}{1+e_0} L_0 \log \frac{t}{t_p} \quad (1)$$

where e_0 is the void ratio corresponding to end-of-primary consolidation stage, L_0 is height of the layer corresponding to primary consolidation stage, t_p is time of primary consolidation stage.

In laboratory, the value of t/t_p can be very large due to the duration of primary settlement of laboratory samples being generally small. On the field, the value may range from several months to many years, and for the typical useful life of a structure, t/t_p rarely exceeds 100 and often considered less than 10. As t/t_p decreases, the secondary settlement becomes less significant. [11]

1.2 End of the primary consolidation (EOP) concept and the constant C_{α}/C_c concept

The C_{α}/C_c concept was developed based on the observation that the magnitude and behavior of C_{α} with time is directly related to the magnitude and behavior of C_c with consolidation pressure. In general, C_{α} remains constant, decreases, or increases with time, in the range of consolidation pressure at which C_c remains constant, decreases, or increases with σ'_v , respectively. The value of C_{α}/C_c together with the end-of-primary (EOP) e - $\log \sigma'_v$ curve completely defines the secondary compression behavior of any one soil [8]. The values of C_{α}/C_c are listed in Table 1.

Table 1 Values of $C\alpha/Cc$ for Geotechnical materials [11]

Material	$C\alpha/Cc$
Granular soils including rockfill	0.02 ± 0.01
Shale and mudstone	0.03 ± 0.01
Inorganic clays and silts	0.04 ± 0.01
Organic clays and silts	0.05 ± 0.01
Peat and muskeg	0.06 ± 0.01

The value of $C\alpha$ in the early stages of secondary compression from t_p to $10 t_p$ is computed from $C\alpha/Cc$ together with Cc from the EOP e vs $\log \sigma'_v$ curve (Fig. 3). Thus equation of settlement during secondary compression can be written as

$$S = \frac{C\alpha/Cc \times Cc}{1+e_0} L_0 \log \frac{t}{t_p} \quad (2)$$

The $(C\alpha, Cc)$ data pairs should be plotted in a $C\alpha$ versus Cc diagram. The slope of the best fit line through the origin defines $C\alpha/Cc$. $C\alpha$ increases, remains constant, and decreases with time at the first, second, and third consolidation pressures, respectively. Therefore, in the range of consolidation pressure at which Cc changes rapidly, care must be exercised in selecting the corresponding values of $C\alpha$ and Cc . This is especially true for highly structured clays near the preconsolidation pressure where Cc abruptly increases from the recompression to the compression range and $C\alpha$ increases with time.

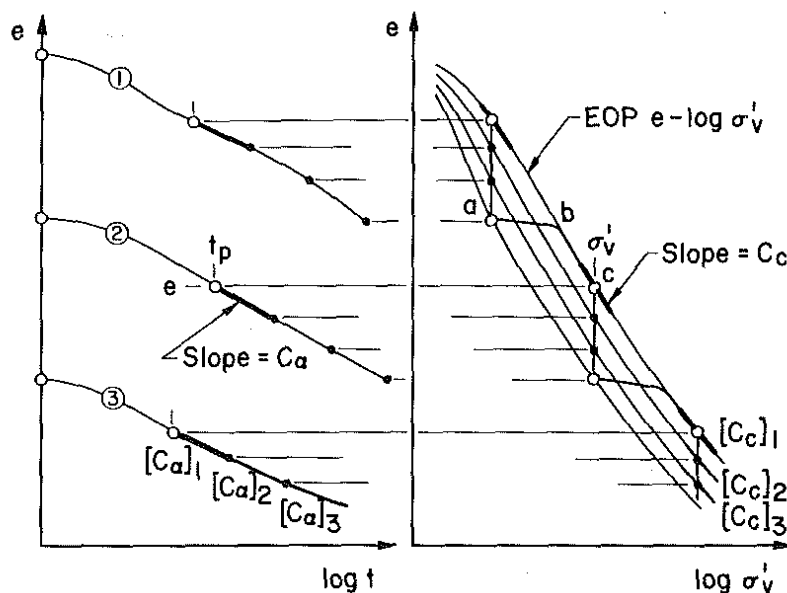


Figure 3. $C\alpha$ and Cc corresponding to any instant (e, σ'_v, t) during secondary compression [8]

1.3 Hypothesis A and B

An important question was raised in the 1970s about the appearance of secondary compression or creep in soils, whether creep acts as a separate phenomenon until all the excess pore pressures are dissipated during primary consolidation. If it does, then for a given effective vertical stress increment the end-of-primary (EOP) vertical strain depends upon the duration of primary consolidation, and hence on the thickness of the consolidating soil layer. Such a consideration led to the realisation of two possible extreme effects of sample thickness, which are summarised in Fig. 4 in terms of hypotheses A and B. In hypothesis A, the strain at EOP is assumed to be independent of the consolidation period, whereas hypothesis B predicts an increasing EOP strain with increasing consolidation period or increasing sample thickness.

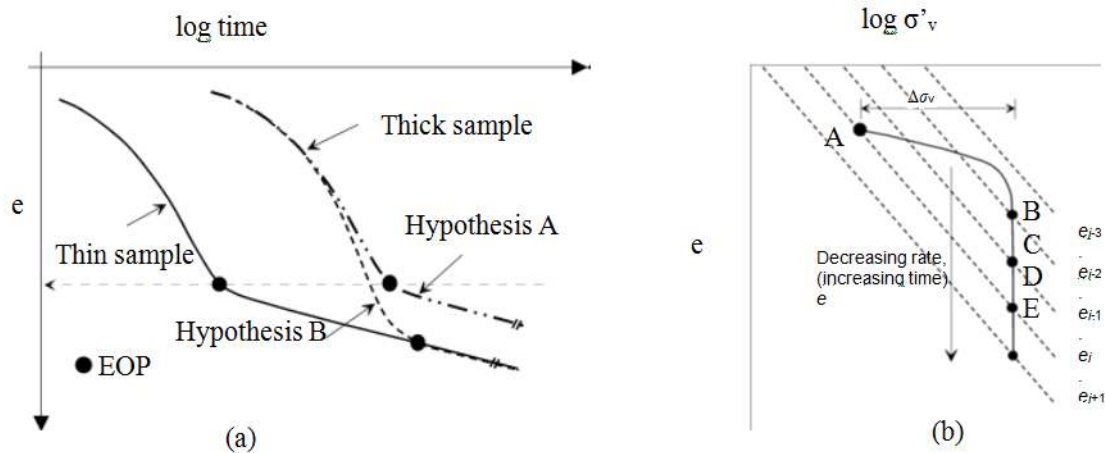


Figure 4 a) Effect of sample thickness according to creep hypothesis A and B b) Illustration of isotache concept [3]

A possible implication of the two creep hypotheses in terms of effective stress–void ratio (strain) is stated as follows. Hypothesis A predicts that the relationship between EOP void ratio (strain) and effective stress is the same for both laboratory and field conditions. This means that the EOP preconsolidation stress is identical for laboratory specimens and in the in situ condition. Hypothesis B yields a relationship between an in situ EOP strain and effective stress that is different from the corresponding laboratory curve, such that the in situ EOP preconsolidation stress is lower than that determined from an EOP laboratory test. Since 1977, this concern has continued to be a topic of active discussion among researchers, and remains an issue that needs to be resolved. A summary of some of the discussions can be found in [5] and [7].

Early work by researchers studying creep ([9],[2], [4],[10]) assumed that the creep rate was given by the current effective stress and the current void ratio (strain). In other words, any combination of void ratio (strain), effective stress and rate of change of void ratio (strain rate) is considered to be unique throughout the primary and secondary consolidation phases. These formulations can be classified as isotache models, and imply hypothesis B.

1.4 The Isotache Concept (Equivalent time lines)

The isotache concept was first proposed by Šuklje [9], which professed a unique relationship between the strains and the consolidation pressure corresponding to the strain rate in association with viscosity. Strain rate effect is a key parameter here. This approach states that the rate of change of void ratio is given by the prevailing void ratio and effective stress. The concept of isotaches can be explained using the sketch shown in Fig. 4b. The series of parallel broken lines in the figure are creep isotaches. Each creep isotache corresponds to a constant void ratio rate, \dot{e}_{j+n} . This means that any combination of void ratio, vertical effective stress and rate of change of void ratio is unique, and this remains valid during the entire soil compression process (primary and secondary consolidation phases). For instance, consider a soil element close to a draining boundary. Point A is assumed to be the initial state of this soil element. A vertical total stress increment, $\Delta\sigma_v$, is then applied and left to creep for some time. The path followed is represented by the solid line ABCDE. This path is dependent on the distance of the soil element from the drainage boundary, as the effective stress rate and strain rate are governed by the consolidation process. Depending on the duration of the applied effective stress, the final state of the soil element can be B, C, D or E.

Watabe et al. [12] proposed a simplified method with the isotache concept using a reference compression curve and a function of the strain-rate dependency of the consolidation yield stress (preconsolidation pressure) obtained from both constant rate of strain one-dimensional consolidation (CRS) tests and long-term consolidation (LT) tests under a constant applied stress. The isotache

parameters used in this method was commonly determined for the Osaka Bay clays retrieved from various depths upto 300m below the seabed at the Kansai International Airport. This method was useful, as it was not necessary to determine the parameters at each depth.

2. Modelling the time dependent stress-strain characteristics of soils

The equivalent time concept was used to derive a 1 D elastic viscoplastic (EVP) model. It is generalized into a 3D EVP model based on modified cam clay and viscoplasticity[19] The four fundamental concepts used in the EVP model under 1D straining are i) equivalent timeline, t_e ii) Instant time line (κ line) iii) reference timeline (λ line) iv) limit timeline

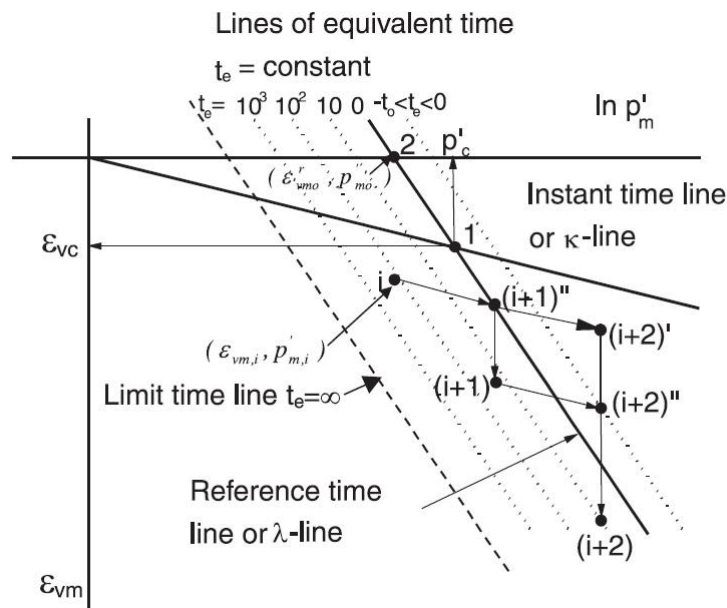


Figure 5. Illustration of equivalent timeline, instant time line, reference time line and limit time line [19]

(i) *Equivalent times.* The equivalent time t_e is defined as the time needed to creep from a reference time line to the current value of ϵ_{vm} , and p'_m under constant effective stress. In the normally consolidated range of multistage loading tests with constant load increment ratio and constant load durations, equivalent times are usually close to the duration of the increments. However, in the overconsolidated range, equivalent times and load durations may be quite different, depending on the overconsolidation ratio. An equivalent time is related to a unique creep rate, with larger equivalent times being associated with smaller creep rates ϵ'_{vm} . The relationship $\epsilon_{vm} - p'_m - t$ (or $\epsilon_{vm} - p'_m - \epsilon^{\circ}_{vm}$) is unique and independent of loading history

(ii) *Instant time line (κ line).* The instant timeline is used to define the instantaneous volume strain at any point above or below the limit timeline in Fig. 5. The volume strain on the instant timeline can be expressed as

$$\epsilon_{vm}^e = \epsilon_{vm0}^e + \frac{\kappa}{V} \ln\left(\frac{p'_m}{p_u}\right) \quad (3)$$

where p_u is a unit-reference mean stress, p'_m is the mean effective stress under isotropic stressing, ϵ_{vm0}^e = volume strain at $p'_m = p_u$, $V = 1 + e_0$ is the specific volume, κ/V is a material parameter used in the same way as in modified cam clay model. The instant timeline is actually a set of lines with a slope of κ/V in the $p'_m - \epsilon_{vm}$ plane. For example, if a soil in a state at point i in Fig. 5 is loaded to $p'_{m,i+1}$, the state of the soil will first instantly move from i to $(i+1)''$ along an instant timeline (the same slope as that of the κ line), and then it will experience creep, i.e., delayed deformation from $(i+1)''$ to $(i+1)$.

(iii) *Reference timeline (or λ line).* The reference timeline is written as

$$\varepsilon_{vm}^r = \varepsilon_{vmo}^r + \frac{\lambda}{V} \ln\left(\frac{p_m'}{p_{mo}'}\right) \quad (4)$$

where ε_{vmo}^r is the strain at mean effective stress $p_m' = p_{mo}'$; and ε_{vmo}^r , λ/V , and p_{mo}' are three parameters. The term λ/V is similar to that used in the modified Cam-Clay model for defining the elastic-plastic line for isotropically consolidated specimens in a normally consolidated stress range. It has been shown by Yin and Graham [15] that if the viscous nature is ignored, the reference timeline in eq. [4] expresses the elastic-plastic compression line as in the modified Cam-Clay model. The two parameters ε_{vmo}^r and p_{mo}' determine a point which the λ line passes through, i.e., they are used to fix the λ line. The λ line corresponds to zero equivalent time, and hence provides a reference for counting equivalent time te , which is discussed later in this section. As shown in Fig. 5, te is negative above the λ line and positive below the λ line. Once the reference timeline is determined, a unique relation of $p_m' - \varepsilon_{vm} - te$ is then established.

(iv) *Limit time line.* The logarithmic function has been commonly used to fit the creep strain ε_{vm}^{vp} versus time or secondary compression versus time. [17] proposed a nonlinear creep function with a limit and found that this function can fit creep test data well. The nonlinear creep function is expressed as

$$\varepsilon_{vm}^{vp} = \frac{\psi}{V} \ln\left(\frac{t_o + t_e}{t_o}\right) \quad (5)$$

where

$$\frac{\psi}{V} = \frac{\frac{\psi_o}{V}}{1 + \frac{\psi_o}{V \varepsilon_{vml}^{vp}} \ln \frac{t_o + t_e}{t_o}} \quad (6)$$

ψ/V , t_o , and ε_{vml}^{vp} are three constant parameters. If considering $\ln[(t_o + te)/t_o]$ together as a variable, eq. [5] is in fact a hyperbolic function. A hyperbolic function using time t (or te) directly cannot provide a good fit to the creep data. In eq. [6], if $\varepsilon_{vml}^{vp} = \infty$, the parameter $\psi/V = \psi_o/V$. Consequently, eq. [6] would be reduced to the commonly used logarithmic function. From eq. [6], it is also found that when $te = \infty$, $\varepsilon_{vm}^{vp} = \varepsilon_{vml}^{vp}$. This implies that there is a limit for the viscoplastic (creep) volume strain.

3. Development of the 3d elastic visco-plastic (evp) model

Bjerrum [2] was the first to suggest that strains in an oedometer can be decomposed into instant and delayed strains. This concept has been widely accepted in both isotropic and general 3D stress states. Consequently the total strain rates are the sum of elastic strain rates and viscoplastic strain rates.

$$\dot{\varepsilon}_{ij} = \dot{\varepsilon}_{ij}^e + \dot{\varepsilon}_{ij}^{vp} \quad (7)$$

where subindices $I = 1, 2, 3$ and $j = 1, 2, 3$; and the overdots indicate the rate of a variable

For the 1D oedometer condition, the composition of eq. [7] is simplified as

$$\dot{\varepsilon}_z = \dot{\varepsilon}_z^e + \dot{\varepsilon}_z^{vp} \quad (8)$$

Here, the instant strains are assumed to be elastic, time-independent, and recoverable, but the delayed strains are viscoplastic, time-dependent, and irrecoverable. This model is related to the classic Maxwell's linear rheological model. Yin and Graham [14,15] devised the constitutive equation of a 1D EVP model using the equivalent time concepts.

$$\dot{\varepsilon}_z = \frac{K}{V} \frac{\dot{\sigma}_z}{\sigma_z} + \frac{\psi}{V t_0} \exp\left[-(\varepsilon_z - \varepsilon_{zr}) \frac{V}{\psi}\right] \left[\frac{\sigma_z'}{\sigma_{zr}'} \right]^{\frac{\lambda}{\psi}} \quad (9)$$

where $\dot{\sigma}_z'$ and σ_z' are vertical effective stress rate and stress; κ/V = a constant related to elastic compression; and λ/V = a constant related to the reference time line. The variables σ_{zr}' and ε_{zr} are defined as points where the reference time line passes; ψ/V and t_0 (in units of time) are two constants related to the creep of the soil. The elastic strain rate and viscoplastic strain rate is given as follows

$$\dot{\varepsilon}_z^{*e} = \frac{\kappa}{V} \frac{\dot{\sigma}_z'}{\sigma_z'} \quad (10)$$

$$\dot{\varepsilon}_z^{*vp} = \frac{\psi}{Vt_0} \exp\left[-(\varepsilon_z - \varepsilon_{zr}) \frac{V}{\psi}\right] \left[\frac{\dot{\sigma}_z'}{\sigma_{zr}'}\right]^{\frac{\lambda}{\psi}} \quad (11)$$

It can be observed from eq. [11] that the viscoplastic strain rate (the creep-strain rate) is a function of the stress-strain state (σ_z' , ε_z), and how this stress-strain rate is reached is not a matter of concern. The classic Maxwell's rheological model is expressed as

$$\dot{\varepsilon}_z = \frac{\dot{\sigma}_z'}{E} + \frac{\sigma_z'}{\eta} \quad (12)$$

where E = elastic modulus and η = viscous constant. Maxwell's rheological model that is derived from a series connection of a linear spring and a linear dashpot divides total strain rate into elastic strain rate and a viscoplastic strain rate. As the strains due to the dashpot are not recoverable, Maxwell's rheological model is not a viscoelastic model but a linear elastic and linear viscoplastic model. On comparison with Yin and Graham's 1D EVP, it can be considered to be an extension of the classic Maxwell's linear rheological model [18].

Yin and Graham [17] derived a 3D EVP model using the separation of elastic and creep strains and the modified Cam-Clay model:

$$\dot{\varepsilon}_{ij} = \dot{\varepsilon}_{ij}^{*e} + \dot{\varepsilon}_{ij}^{*vp} \quad (13)$$

$$\dot{\varepsilon}_{ij} = \frac{1}{2G} \dot{S}_{ij} + \frac{\kappa}{3V} \frac{p'}{p'} \dot{\delta}_{ij} + \frac{\psi}{Vt_0} \exp\left[\left(\varepsilon_{vm}^r + \frac{\lambda}{V} \ln \frac{p_m'}{p_{m0}'} - \varepsilon_{vm}\right) \frac{V}{\psi}\right] \frac{1}{(2p' - pm')} \frac{\partial F}{\partial \sigma_{ij}'} \quad (14)$$

Where σ_{ij}' = effective stress; the mean effective stress p' is defined as $p' = \sigma'_{kk}/3$; s_{ij} = deviator stress rate; the deviator stress s_{ij} is defined as $s_{ij} = \sigma'_{ij} - \delta_{ij} \sigma'_{kk}/3$, where $\delta_{ij} = 0$ if $i \neq j$, $\delta_{ij} = 1$ if $i = j$; G = elastic shear modulus; and κ/V (V is specific volume), ψ/V , t_0 , λ/V , pm'_0 , and ε_{vm0}^r are model parameters.

The F in previous equation is a function describing the viscoplastic flow surface [17]

$$F = p'^2 + \frac{q^2}{M^2} - p' p_m' = 0 \quad (15)$$

where M = slope of the critical state strength envelope in the plane, and q = generalized deviator stress $\sqrt{(3/2)s_{ij}}$, pm'_0 is the mean effective stress at which the flow surface intercepts the p' -axis in the $q - p'$ plane. The index m represents the mean stress or volume strain under isotropic stressing conditions, that is, $p' = pm'_0$ with $q = 0$.

4. A simplified method to calculate consolidation settlement of a clayey soil with creep

Ying and Feng [20] devised a simplified method based on hypothesis B by spreadsheet calculation for the consolidation settlement of a single layer of clayey soil with creep. The total consolidation settlement is given by the equation

$$S_{total} = S_{primary} + S_{creep} \quad (16)$$

$$S_{primary} = U_V S_f \quad (17)$$

$$S_{creep} = \alpha S_{creepf} + (1 - \alpha) S_{secondary} \quad (18)$$

$$S_{creepf} = \frac{c_{ae}}{1+e_0} \log \frac{(t_0+t_e)}{t_0} \quad (19)$$

$$S_{secondary} = \frac{c_{ae}}{1+e_0} \log \frac{t}{t_{E0Pfield}} \quad (20)$$

where α is the parameter to reasonably consider the creep compression coupled with consolidation; $S_{creep,f}$ is the creep settlement under the final effective vertical stress without excess porewater pressure coupling. $S_{secondary}$ is the same as that in eq. [20]. The value of α varies between 0 and 1. When $\alpha=0$, the equation follows Hypothesis A method. By introducing α value, secondary compressions are assumed to occur along with primary consolidation settlements. Ying and Feng [20] determined a suitable value of α by comparing the settlements calculated using eqn.16 with settlements from finite element model simulations.

The new simplified Hypothesis B method in eq. [16] with a suitable value of α takes a more accurate approach to calculate the creep settlement during and after primary consolidation. With a value of $\alpha=0.8$, the settlements calculated using the simplified hypothesis B are closer to the settlements from a fully coupled consolidation modeling. This value has been verified for three Hongkong marine clays and test results from Berre and Iversen [1]

5. Compressibility studies on Cochin marine clay and Kuttanad clay

Secondary compression studies have not been much studied on Indian clays. The marine clays of India like Bombay marine clay, Cochin marine clay, Kuttanad clay are well known for post-constructional settlements. This section presents some compressibility studies done on Cochin marine clay- CMC (Vallarpadam) and Kuttanad clay (Pulinkunnu). The secondary compression characteristics are determined and compared in the undisturbed and remoulded states for CMC. The compression curves obtained for Cochin marine clay in undisturbed, remoulded, slurry conditions are shown in Fig. 6. The time period of each load was either 24 hours or end-of-primary compression. The samples showed much variation in their initial void ratios. The presence of non-uniform sand seams in these samples could be a reason. The c_v (Coefficient of consolidation) values range from 10^{-4} to 10^{-3} cm²/s. The compression index value ranges between 0.8 to 1.4. The coefficient of secondary compression values are provided in Table 2. The c_d/c_c values lie in the range of 0.003 to 0.046 that are characteristic of organic clays.

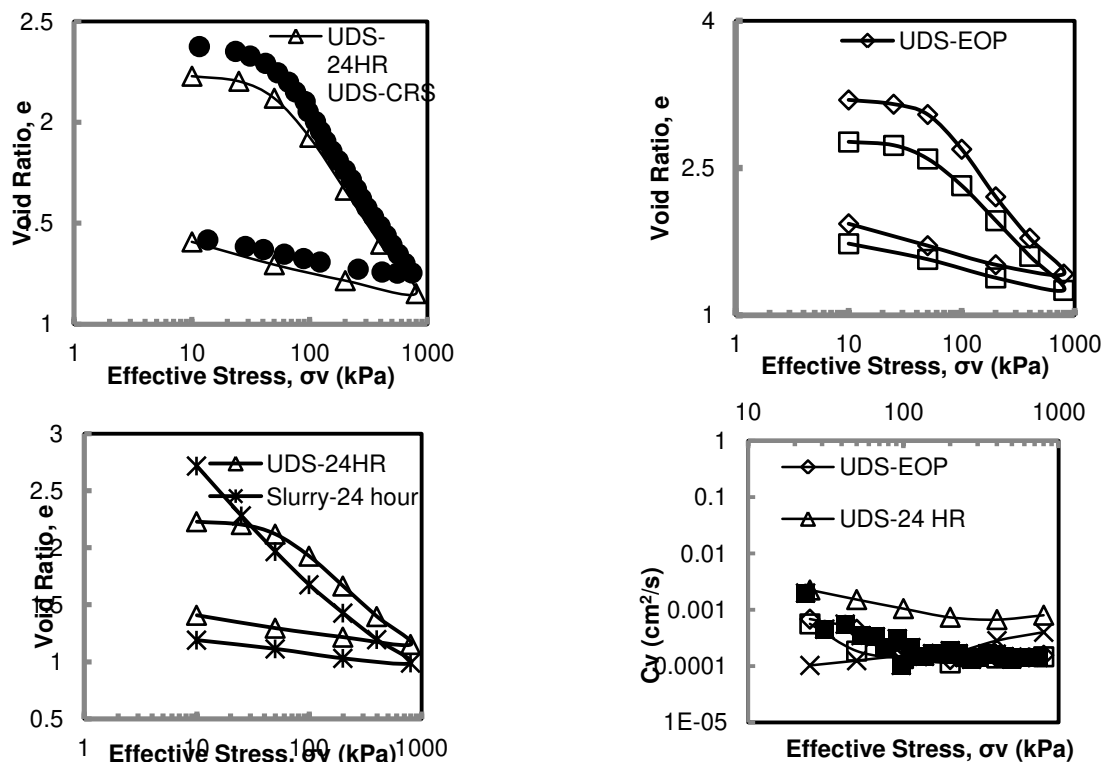


Figure 6. Compression curves for Cochin marine clay

Table 2. Compression characteristics of Cochin Marine Clay

	Cc	Cr	Cc/Cr	UDS(24 hr)			Remoulded(24 hr)		
				Pressure	C α	C α /Cc	Pressure	C α	C α /Cc
EOP	1.3804	0.2752	5.01						
UDS(24 hr)	0.8061	0.1152	6.997	25	0.00023	0.000303	25	0.004	0.003546
				50	0.0076	0.01	50	0.023	0.02039
Remoulded	1.128	0.2554	4.417	100	0.012206	0.016061	100	0.044678	0.039608
UDS(CRS)	1.0769	0.1278	8.425	200	0.033163	0.043636	200	0.048593	0.043079
				400	0.032242	0.042424	400	0.036157	0.032054
Slurry(24 hr)	0.8982	0.0921	9.752	800	0.020727	0.027273	800	0.039151	0.034708

The compression curves for Kuttanad clay are provided in Fig. 7. Slurry samples were consolidated for various time periods for each pressure. The consecutive loads in each test were added at end-of-primary consolidation, 24 hours and 10 day time period, respectively. With increasing time of loading period, the compression under each load is more, even though the starting void ratio is the same. This indicates the secondary compression behaviour. Samples were also air dried and oven dried and then remixed to slurry form. The compression curves of these samples are also plotted in the same figure

The compression characteristics are shown in Table 2. The c_{α}/c_c values range from 0.02 to 0.06. These fall in the category of soils with high secondary compression. The c_c values range between 1.1-1.4.

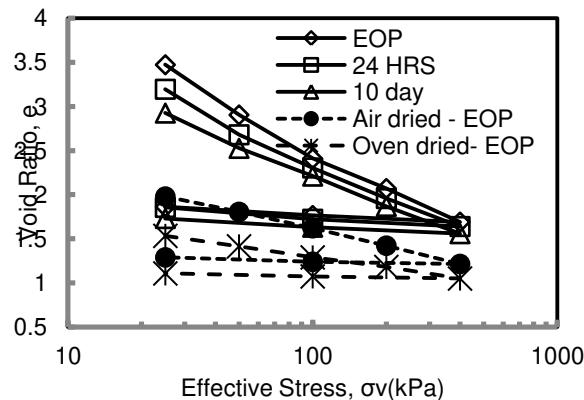


Figure 7. Compression curves for Kuttanad clay in different conditions

Table 3. Compression characteristics of Kuttanad clay with different time period of loading

	EOP	24 HOUR	10 DAY
c_c	1.22	1.34	1.13
c_r	0.21	0.18	0.18
c_{α}/c_r	5.9	7.3	6.76
c_{α}	-	0.0281-0.0702	0.034-0.0451
c_{α}/c_c	-	0.0181-0.0596	0.0321-0.0416

6. Summary and conclusions

With increasing infrastructure developments on problematic soils especially soft clays, the need to estimate consolidation settlements (mainly post construction settlements) accurately is a necessity. Secondary (or creep) settlements calculations have evolved through Hypothesis A and B; the former assumes secondary compression to begin after end-of-primary consolidation while the latter assumes it occurs along with primary consolidation. While the parameters C_{α} and C_{α}/C_c are useful in indicating the nature of creep a material can exhibit, it may not accurately estimate the secondary consolidation

settlements. The isotache approach, which is now a widely discussed topic in academia, has potential to closely represent the secondary compression behavior of soft clays by elastic viscoplastic modeling. With reference to Indian soils, Cochin marine clay and Kuttanad clay are soils with high secondary compression characteristics. The application of the isotache approach is deemed useful for these soils.

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