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CREEP BEHAVIOUR OF SATURATED CLAY SUBJECTED TO CYCLIC LOADING IN PLAIN STRAIN CONDITION

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Abstract

Creep of clayey soils is defined as the time-dependent deformation under sustained stresses, due to viscous behavior of the soil skeletons. Creep behavior of soil depend upon main factors which include time, temperature, soil type, soil structure, stress history, stress state and drainage conditions. In this study, a series of one-dimensional oedometer laboratory creep tests were conducted on samples subjected to cyclic loading in order to study the effect of overconsolidation ratio (OCR) and duration of preloading on the secondary consolidation coefficient of the soft clay. Test results reveal that the secondary consolidation coefficient of the overconsolidated samples are smaller than that of the normally consolidated samples, and decreases with increasing OCR with a drastic drop for OCR values close to 1. On the basis of the isotache model, an equation for secondary consolidation coefficient compression index is proposed as a function of OCR and duration of preloading. The laboratory tests show that the actual measured secondary consolidation coefficient is independent of time. Therefore, to comply with this reality, the proposed equation was modified to be independent of time and laboratory test results were used to calibrate parameters of the newly proposed equation.

Keywords: clayey soils; creep behavior; overconsolidation ratio; secondary consolidation; cyclic loading.

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1. Introduction

The creep deformation of over consolidated soft soil is the main component of post-construction settlement of soft soil foundation improved with surcharge preloading. In geotechnical engineering, especially highway embankment construction on soft soils, surcharge preloading together with either perforated vertical drains (PVD) or sand drains are commonly adopted to reduce the long-term post-construction settlement of the soft ground. In most of the surcharge preloaded projects for highway embankments, vertical drains are installed to speed up the compression. Thus the primary compression for permanent constant load is completed before the removal of surcharge. Generally the surcharge could transform into effective stress entirely during preloading process, unloading surcharge can make soil layers underground over consolidated. And the post-construction settlement is mainly the result of the secondary compression (creep) of the soft soil at over consolidated state.

The creep behaviour of soft clay is nearly related to its stress history. Generally, it is reasonable to believe that the creep coefficient is constant for one sort of soft clay at normally consolidated state [1–3]. And the creep deformations of soft clay could be calculated with creep coefficient and increment of logarithm of creep time. For over-consolidated soil, the creep behaviour also has been studied by few researchers in the past. One of the viewpoints is to use a constant creep coefficient of soft clay to solve the creep problem including normally and over-consolidated state. Another one is that the creep coefficient of soft clay is different between normally and over consolidated state. Even though

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above viewpoints are different, both of them thought the creep deformations of soft clay decreases at over consolidated state relative to normally consolidated state. Typical results of the first viewpoint came from Yin and Graham [4–7] who thought that, the creep process of over-consolidated soft clay is the same as normally consolidated state at a period of creep duration beginning at equivalent time corresponding to the over-consolidated state [8–11].

However, the effects of removing the surcharge and preloading duration on the secondary compression have been rarely studied. Kassem et al., Yoshikuni et al., Kamao, Alonso [12–15] and Nguyen et al. [16] have studied the secondary compression after the removal of the surcharge and some empirical equations were proposed to predict the settlement. However, these equations didn't consider the effects of the magnitude and duration of the preloading on the secondary compression. The isotache model ([1, 2]) was initially proposed for considering the time effect on the settlement of soft soil. Based on the framework of the isotache model, many constitutive models Fukazawa, Shimomura, Murakam, Li et al. [17–20] and Yuzhou et al. [21] have been developed which use different assumptions for evaluating the starting time for the secondary compression. They proposed a one-dimensional EVP model based on the concept of "equivalent times" during time dependent straining. In order to quantify equivalent times, a reference time line was defined along which equivalent time has zero value.

As for empirical model, there are two shortcomings being ignored for years. First, the clayey deposits are needed to be preloaded by surcharge, vacuum load or combine surcharge and vacuum load before the construction of the upper pavement in order to reduce the post-construction settlement of soft foundation. This induces an over-consolidated (OC) state of clay after removing the applied load. The soft foundation may have significant deformation even ever experienced a considerable maximum past effective stress [3, 7, 10, 11, 15, 20], i.e., with a large value of over-consolidate ratio (OCR). The second issue is that the soft foundation under high embankment is of plain strain condition typically. Whereas, the current empirical models cannot directly consider the influence of both OCR and plains strain condition in their models yet. This study investigated the creep rate for OC clay under plain strain condition through laboratory tests, and a new simple empirical correlation between the creep rate and the degree of overconsolidation and principal stress ratio was proposed.

2. Laboratory studies

2.1. Soil used and cases tested

One-dimensional laboratory creep tests were conducted to study the key factors that control the secondary compression of soft clay. The samples were taken at a depth between 3 m and 9 m below the original ground surface. The height and cross-sectional area of the soil specimens for oedometer tests are 2 cm and 30 cm^2, respectively. The basic physical properties of the soil are summarized in Table 1.

Table 1. Soil properties

Water Content ω (%)	Density ρ (g/cm^3)	Specific Gravity G_s	Void Ratio e (%)	Liquid Limit w_L(%)	Plastic Limit w_p(%)	% of Silt (0.075-0.005 mm)	% of Clay (< 0.005 mm)	Coefficient consolidation C_{v100} (m^2/s)
88~92	1.46	2.7	2.28~2.85	63	31	46	40	1.95×10^{-6}

2.2. Test procedure

Two series of creep tests were conducted and the testing conditions are summarized in Table 2. In the first series of tests (test No. 1 – 8), the creep tests were conducted in the reloading stage of the one-dimensional compression tests. The specimens were loaded initially to a maximum vertical stress ranging from 200 kPa to 400 kPa for 24 hours and then unloaded to a vertical stress of 25 kPa. After unloading for 24 hours, the specimens were then reloaded to a vertical stress ranging from 50 kPa to 300 kPa after which the creep tests were carried out. In the second series of tests (test No. 11 – 14), the creep tests were conducted in the unloading stage of the one-dimensional compression. The soil specimens were loaded initially to a vertical stress of 100 kPa for 24 hours, after which the vertical stress was unloaded to a vertical stress ranging from 90 kPa to 95 kPa and then the creep test was conducted.

Two additional creep tests (test No. 9 and 10) were conducted on soil specimens normally compressed to a vertical stress of 100 kPa as a reference for interpreting the test results of overconsolidated specimens.

The secondary consolidation coefficient ($C_{\alpha(\varepsilon)}$) in terms of strain is defined as follows:

$$C_{\alpha(\varepsilon)} = \frac{\Delta\varepsilon}{\log t_2 - \log t_1} \tag{1}$$

where $\Delta\varepsilon$ is vertical strain increment due to the creep process and t_1 and t_2 are the starting and finishing time for the creep process, the time during the post-consolidation process (secondary consolidation), specifically in the reloading stage after unloading. If the creep coefficient ($C_{\alpha(e)}$) in terms of void ratio is defined in terms of the increment in void ratio (Δe), $C_{\alpha(e)}$ becomes:

$$C_{\alpha(e)} = \frac{\Delta e}{\log t_2 - \log t_1} = C_{\alpha(\varepsilon)} \cdot (1 + e_0) \tag{2}$$

where e_0 is initial void ratio. The measured creep coefficient from the creep tests are summarized in Table 2.

Table 2. Parameters from sample creep tests on undisturbed soil samples

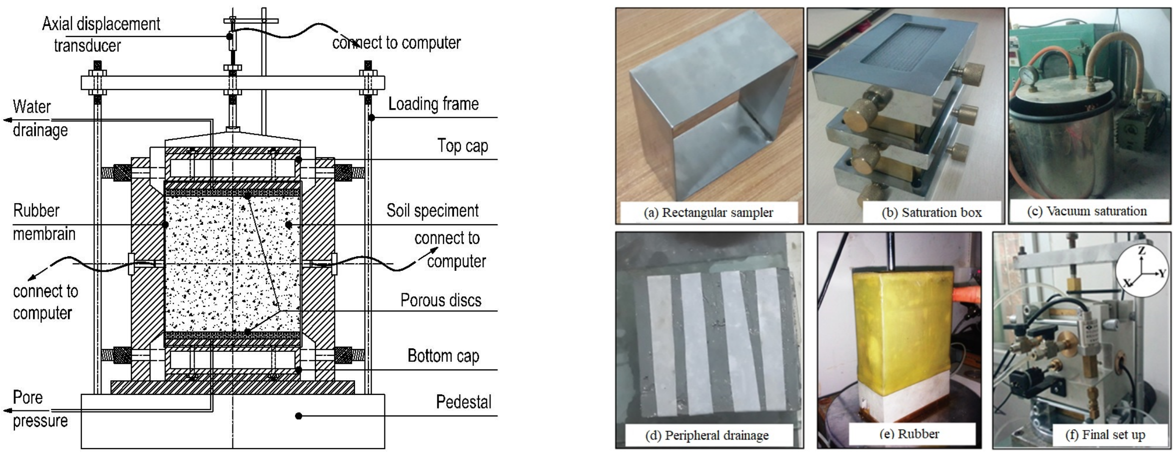
No.	OCR	$C_{\alpha(e)}$	$C_{\alpha(\varepsilon)}$	p'_c (kPa)	p'_{con} (kPa)	Load method
1	8	0.00237	0.000796	400	50	Consolidation with p'_c for 24 hours, unloaded to 25 kPa and kept for 24 hours, after that undergoing creep with constant load of p'_{con}.
2	4	0.00393	0.00134	400	100	
3	2	0.00556	0.00190	400	200	
4	1.33	0.00647	0.00221	400	300	
5	1.18	0.00978	0.00377	200	169.5	
6	1.15	0.0114	0.0044	200	173.9	
7	1.1	0.0130	0.00502	200	181.5	
8	1.05	0.0173	0.00666	200	190.5	
9	1	0.0286	0.00939	100	100	Consolidation with p'_c for 24 hours, unloaded to a constant load of p'_{con} and undergoing creep test under this loading.
10	1	0.0232	0.00760	100	100	
11	1.05	0.0160	0.00524	100	95	
12	1.05	0.0161	0.00527	100	95	
13	1.11	0.0118	0.00388	100	90	
14	1.11	0.01089	0.00353	100	90	

OCR, the ratio of over-consolidation which was defined as the ratio of preconsolidation pressure (p'_c) and present overburden pressure (p'_{con})

2.3. Apparatus and sample preparation

A test equipment for creep under plain strain condition due to Li et al., [20, 22, 23] and Thang [24] was developed as illustrated in Fig. 1(a). The consolidometer can apply vertical (Z direction) load σ_z step by step using dead weight and lateral (X direction) load σ_x through pressurised water contained in a rectangular rubber-chamber to the soil samples. In Y direction, two rigid steel boards were used so that the strain in this direction can be set to zero during test. The stress in Y direction σ_y can be measured by a stress sensor. The settlement in Z direction and volume of water drained out can be measured and recorded by computer through a data logger.

The soil sample adopted in this test has a height of 100 mm in Z direction, width of 100 mm in Y direction and length of 50 mm in X direction. The undisturbed soil samples were cut by a rectangular sampler and then placed into a model box and saturated for more than 24 h before testing. Then, strip pore papers were applied on the sample surface to provide peripheral drainage, and the soil sample was set up into a rubber which has the same dimensions. Finally, the sample was put into the consolidometer and the test was started under drainage conditions. Fig. 1(b) shows the photos of the test process.



(a) Schematic of test equipment (b) Photos of the test process

Figure 1. Plain strain creep consolidometer [24]

2.4. Test results and analysis

Figs. 2 and 3 show the results of the creep tests for unloading and reloading conditions, respectively. It is shown that a linear relationship between vertical strain and change in logarithm of time is observed for the long term vertical strain - time relationship, the curves transition to straight lines that are parallel to each other.

Fig. 4 shows the relationship of $C_{\alpha(e)}$and the overconsolidation ratio (OCR) for both unloading and reloading conditions. There is a sharp reduction in $C_{\alpha(e)}$with very moderate increases of OCR, in particular, it reduces substantially for OCR smaller than 1.2 (the slope of the deformation curves in Fig. 2 is significantly greater than that in Fig. 3). Hence, applying preloading beyond the final structural load can be highly effective in minimizing creep settlement, as the stiffness of overconsolidated soil is greater than that of normally consolidated soil.

Additionally, it is worth noting that the data points obtained during both the reloading and unloading stages exhibit a similar pattern, as demonstrated in Thang [24]. Therefore there is no need to

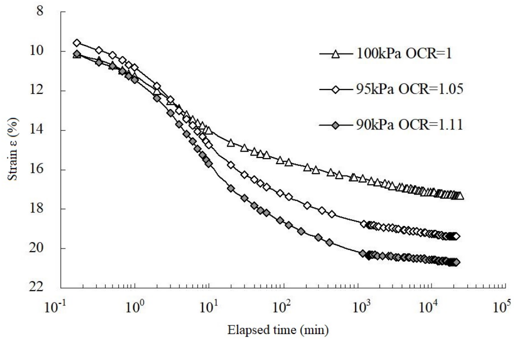


Figure 2. Unloading creep test after consolidation with 100 kPa loading for 24 hours

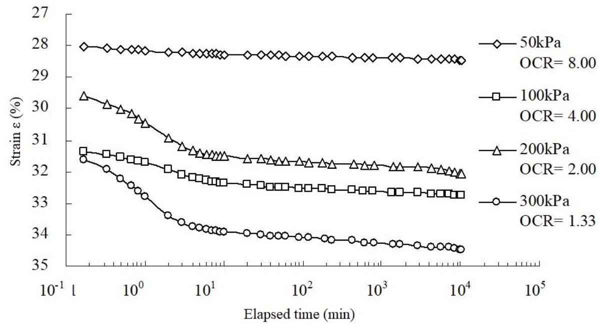


Figure 3. Reloading creep test after consolidation with 400 kPa, unloaded to 25kPa

distinguish between those two situations and in the computation of creep settlement, coefficient of creep obtained from either reloading or unloading creep test can be used. The results are consistent with the previous studies Yoshikuni et al., [13] and Alonso et al., [15].

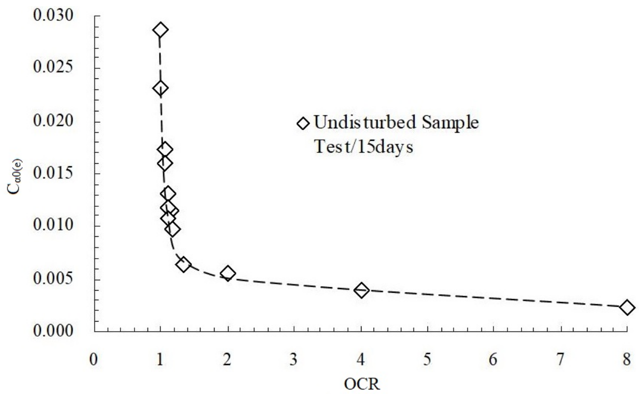


Figure 4. Variation of secondary coefficient of consolidation with OCR

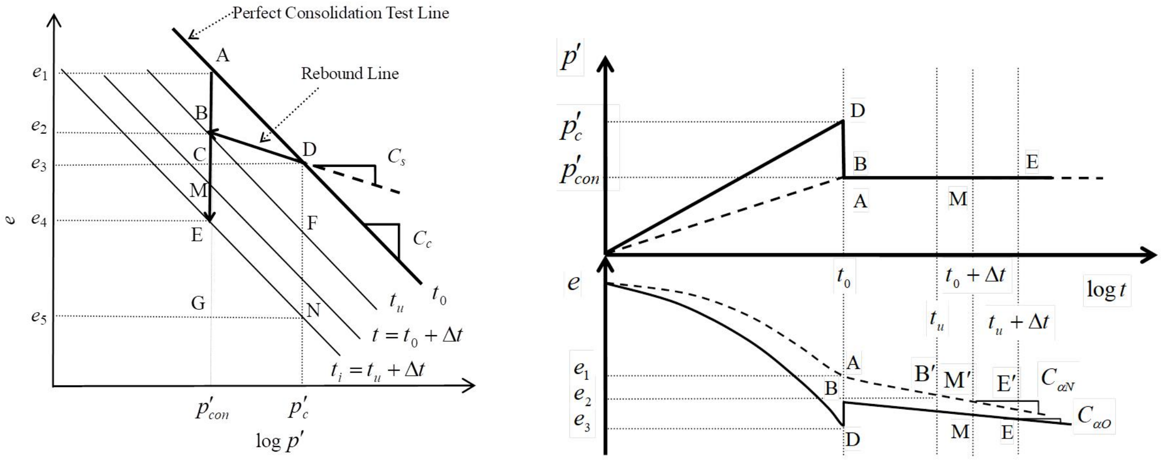
3. Modeling of creep process for over consolidated soft clay

Based on the results of oedometer tests, Crawford [2], etc., showed that the $e - \log p'$ compression curves obtained from different durations can be approximated by a series of parallel lines. Bjerrum [3] have postulated to use a series of parallel lines in $e - \log p'$ plane (see Fig. 5) to represent the time effect on the compression of soil.

Each of these lines represents the equilibrium void ratio for different values of effective overburden pressure at a specific time of sustained loading. For any given value of effective overburden pressure and void ratio there corresponds an equivalent time of sustained loading and a certain strain rate. The figure can be used to represent a unique relationship between void ratio, effective overburden pressure and time. Line AD in Fig. 5(a) represents the preloading path for normally compressed soft clay to a surcharge pressure p'_c. After completion of primary compression, the surcharge pressure is reduced to p'_{con} (line DB). Line BE represents the post-construction process after taking away the surcharge loading. Eqs. (3) to (6) are obtained from the geometric relationships shown in Fig. 5(a).

$$e_1 - e_2 = C_{\alpha N} \cdot \log\left(\frac{t_u}{t_0}\right) \tag{3}$$

$$e_2 - e_3 = C_s \cdot \log\left(\frac{p'_c}{p'_{con}}\right) = C_s \cdot \log(OCR) \tag{4}$$



(a) Parallel line about stress, deformation and time

(b) Process and time line about stress and deformation

Figure 5. Creep behaviour of soil between normally and over consolidated situation, Yin and Graham [4]

$$e_1 - e_3 = C_c \cdot \log(\frac{p'_c}{p'_{con}}) = C_c \cdot \log(OCR) \tag{5}$$

$$C_{\alpha N} \cdot \log\left(\frac{t_u}{t_0}\right) + C_s \cdot \log(OCR) = C_c \cdot \log\left(OCR\right) \tag{6}$$

where t_u is the time for the creep process path AB. Since the paths AB and DB lead to point B having the same void ratio e_2, t_u can be regarded as equivalent time for the unloading path DB; t_0 is the time for the completion of primary compression; $C_{\alpha N}$ is creep coefficient for normally compressed soil; C_c is primary compression index and C_s is swelling index, OCR is over consolidation ratio.

Rearrange Eq. (6)

$$t_u = t_0 \cdot OCR^{\frac{C_c - C_s}{C_{\alpha N}}} \tag{7}$$

Compare the path AM and the path DBE, the path AM is a creep process of normally consolidated soil. From A to M, the creep duration time is Δt, and the creep deformation is Δe_{cN} Eq. (8):

$$\Delta e_{cN} = C_{\alpha N} \cdot \log\left(\frac{t_0 + \Delta t}{t_0}\right) \tag{8}$$

The path DBE includes two processes. From D to E, it is unloaded to B first then creep to E, the creep duration time also is Δt. Over consolidated soil creep deformation is Δe_{cO}. According to the Fig. 5(a), $\Delta e_{cO} = e_2 - e_4$, the following equation is obtained from the geometric relationships shown in Fig. 5(a):

$$\Delta e_{cO} = C_{\alpha N} \cdot \log\left(\frac{t_u + \Delta t}{t_u}\right) \tag{9}$$

Eq. (9) shows the creep deformation of over consolidated soil can be calculated with its equivalent time and the creep coefficient which is in normally consolidated situation. This is illustrated more deeply in Fig. 5(b) which is the schematic diagram about the creep process of clay for normally and over consolidated situation. The path *DBE* is a creep process of over consolidated soil corresponding to the deformation time curve *DBME*. The path *AME* is a creep process of normally consolidated soil corresponding to the deformation time curve *AB'ME'*. It is shown, in fact, Eq. (9) is a creep process for a normally consolidated soil, not over consolidated situation. As a result of the postulate that is a series of time line are parallel, the $B'E'$ process in normally consolidated situation is equivalent to the BM process in over consolidated situation because they create the same creep deformations. The $B'E'$ process and the BM process take the same length of time, but they start at different time and develop at different rate. The $B'E'$ process beginning time is t_0 and the slope of deformation time line is $C_{\alpha N}$, the BM process is t_u and $C_{\alpha w}$. The t_u is far more than the t_0 in general, and also $C_{\alpha N}$ is more than $C_{\alpha w}$.

Rearrange Eq. (9) and substitute Eq. (7) into Eq. (9):

$$\Delta e_{cO} = C_{\alpha N} \cdot \log\left(\frac{t_u + \Delta t}{t_0}\right) - (C_c - C_s) \cdot \log(OCR) \tag{10a}$$

$$\Delta e_{cO} = C_{\alpha N} \log\left(OCR^{\frac{C_c - C_s}{C_{\alpha N}}} + \frac{\Delta t}{t_0}\right) - (C_c - C_s) \cdot \log(OCR) \tag{10b}$$

Let $t = t_0 + \Delta t$, substitute $t = t_0 + \Delta t$ into Eq. (8) and Eq. (10b)

$$\Delta e_{cN} = C_{\alpha N} \cdot \log(t/t_0) \tag{11}$$

$$\Delta e_{cO} = C_{\alpha N} \cdot \log\left(\left(OCR^{\frac{C_c - C_s}{C_{\alpha N}}} - 1\right) + \frac{t}{t_0}\right) - (C_c - C_s) \cdot \log(OCR) \tag{12}$$

Rearrange Eq. (12)

$$\Delta e_{cO} = C_{\alpha N} \cdot \log\left(\left(OCR^{\frac{C_c - C_s}{C_{\alpha N}}} - 1\right) + 10^{\log(t/t_0)}\right) - (C_c - C_s) \cdot \log(OCR) \tag{13}$$

Comparing Eq. (11) and (13), it is found that, when OCR is 1, Eq. (13) becomes Eq. (11). Eq. (13) is total formula for the creep deformation of soil with elapse time including normally consolidated and over consolidated situation. And it also shows that the creep deformation of soil could be calculated with its OCR and other parameters obtained in normally consolidated situation. When OCR is 1, the relationship between the creep deformations and $\log(t/t_0)$ is linear whereas, it is nonlinear for OCR greater than 1. This is shown in Fig. 6.

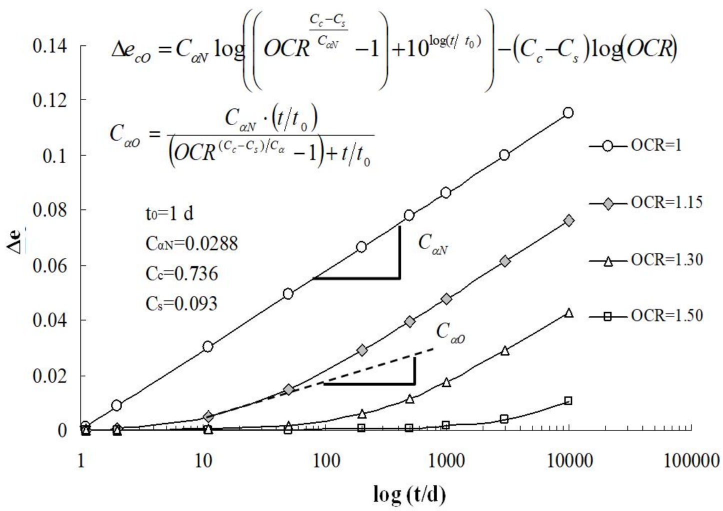


Figure 6. Creep process of normally consolidated and over consolidated soil

When OCR is more than 1, the curve develops with logarithm of elapse time nonlinearly at early duration and becomes linear gradually at later duration time. Fig. 6 demonstrates that the creep behaviour of over consolidated soft clay is different from normally consolidated soft clay for only a short duration of creep time relative to whole creep process of clay in engineering. From the graph it is observed that, long term creep behaviour of normally and lightly over consolidated soft clay is independent of OCR. Because all the curves attain a constant slope equal to the slope of the normally consolidated case at later time duration irrespective of the OCR.

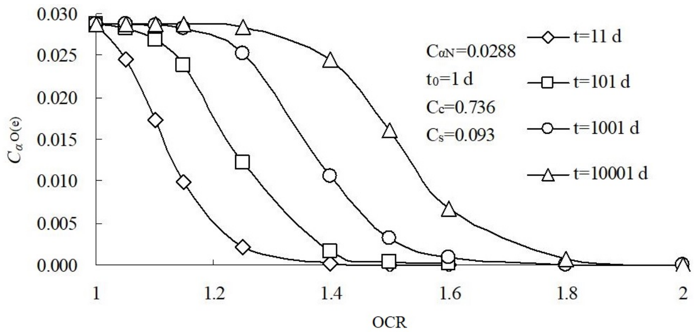
Let $C_{\alpha O} = \dfrac{d(\Delta e_{cO})}{d(\log(t))}$ and from Eq. (13):

$$C_{\alpha O} = \frac{C_{\alpha N} \cdot \frac{t}{t_0}}{\left(OCR^{\left(\frac{C_c - C_s}{C_{\alpha N}}\right)} - 1\right) + \frac{t}{t_0}} \tag{14}$$

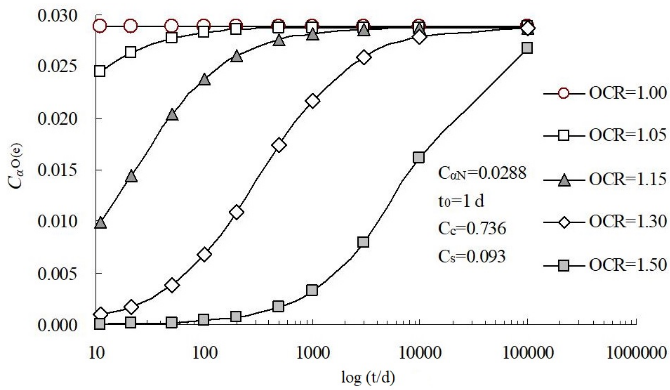
Eq. (14) shows when OCR is 1 the $C_{\alpha w}$ is equal to $C_{\alpha N}$ a constant. Eq. (14) also is a total formula for the slope of the creep behaviour of clay including normally and over consolidated situation. This is shown in Fig. 6, when OCR is more than 1 the slope of the curve is less than the slope when OCR is 1, also not constant at early duration. With the time of creep process getting longer the slope of the creep curve of clay become bigger and bigger gradually and approach to a constant equaled to the creep coefficient of normally consolidated clay. Relationships between the creep coefficient of clay and OCR, logarithm of elapse time are illustrated in details in Fig. 7(a) and Fig. 7(b). The creep coefficient of clay decreases with increasing OCR, and increases with increasing the logarithm of elapse time which ultimately reaches the creep coefficient of normally consolidated clay $C_{\alpha N}$. It is worth of noticing the creep coefficient of clay is time-dependent. This is different from the previous research results about the creep behaviour of soft clay.

Many test results for soft clay creep behaviour have illustrated that the creep deformations increase linearly with the increment of logarithm time both for normally consolidated situation and over consolidated situation. This is true for creep tests conducted by unloading to constant load which is less than pre-consolidation pressure as shown in Fig. 2 or by reloading to constant load as in Fig. 3. But why does so big difference exist between the soft clay creep behaviour obtained from test results and the theoretical results deduced on the basis of the isotache model. In general, the creep tests of soft clay were conducted for about 10 days every load stage. Therefore, it could not be observed what characters the creep behaviour of soft clay would have shown at later time. That is to say the long term function of surcharge for controlling soft clay creep deformation is indeterminate in creep tests. But at earlier time after unloading the function of surcharge is obvious. In summary, it is difficult to estimate which is closer to the real situation of long term creep of soft clay between the theoretical equation (13) and creep test results. If considering the creep deformation in a shorter period began at the initial time of creep, the curve plotted from theoretical resolution equation (13) is a straight line approximately. So, the behaviour reflected in creep tests is similar to the result in Eq. (13) for early duration of soft clay creep.

If the results of creep test of soft clay for OCR greater than 1 are thought to be adequate to meet the engineering requirement, the simplest method for calculating the creep deformation of over consolidated soft clay is to use Eq. (11) same as normally consolidated soft clay. And the creep coefficient of soft clay in this situation is required at OCR being more than 1. For Eq. (14), if the time parameter t is fixed at a smaller value, the relationship between $C_{\alpha w}$ and OCR in Fig. 7(a) looks like the creep test result in Fig. 4. Compare Eq. (14) and the creep test result at early duration of creep process in Fig. 8.



(a) $C_{\alpha(e)}$ and OCR after different unloading stages



(b) $C_{\alpha(e)}$ and log(time) at different OCR values

Figure 7. Relationship between the creep coefficient and OCR, log (time)

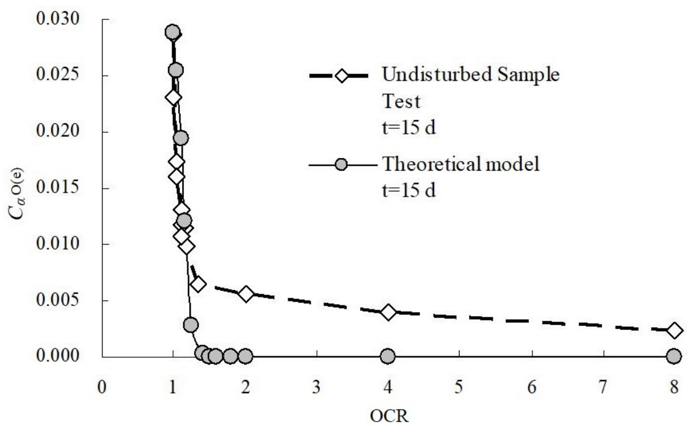


Figure 8. Creep coefficient from theoretical formula and creep test

Fig. 8 demonstrates that $C_{\alpha w}$ decrease with increasing OCR including test result and theoretical formula. And the developing tendencies of these two curves are consistent approximately. But the difference existed is also obvious i.e. theoretical curve decreases with increasing OCR more quickly to a value as low as zero at OCR of less than 2. Therefore, Eq. (14) can not be used to calculate the creep coefficient of over consolidated soft clay directly. Even though Eq. (14) is different from the creep test result, this function can be applicable for the whole range of OCR and time. In spite of the bigger difference, it is a better function to describe the relationship between $C_{\alpha w}$, OCR and creep time. Particularly, it has a strong theoretical basis.

In order to consider the unloading creep test in laboratory, the theoretical creep coefficient model equation (14) of soft clay should be modified to fit the test curve in laboratory. For the test duration used in the laboratory creep tests, a constant $C_{\alpha O}$ (see Figs. 2 and 3) rather than a time-dependent $C_{\alpha O}$ was observed and it is impossible to reach zero with increasing OCR which is greater than 2. Thus Eq. (12) is modified as follows to fit the experimental data presented in Figs. 2, 3 and 4.

$$C_{aO} = \frac{C_{\alpha N} \cdot \beta}{(OCR^{\alpha} - 1) + \beta} = C_{\alpha N} \cdot \frac{1}{1 + \frac{OCR^{\alpha} - 1}{\beta}} \tag{15}$$

where

$$\beta = b\frac{t}{t_0}; \quad \alpha = a\frac{C_c - C_s}{C_\alpha}$$

Rearrange Eq. (13) to

$$\log\left[\beta \cdot \left(\frac{C_\alpha}{C_{\alpha O}} - 1\right) + 1\right] = \alpha \cdot \log(OCR) \tag{16}$$

Eq. (16) is used to fit the experimental data shown in Fig. 4. And the fitted values for α and β are 0.135 and 0.01 respectively. The best fit curve using the proposed model with the calibrated parameters is shown in Fig. 9, where, in spite of a slight deviation between the model and experimental results for $OCR > 2$, the model exactly fits in the range of OCR very close to the normally consolidated zone. From practical and economic point of view, it is within the lower values of OCR designers play during surcharge preloading.

Conducting sensitivity analysis of the model parameters β and α, it is found out that α is the most sensitive parameter. There for it is recommended to fix β to a value close to 0.01 and calibrate α. As can be seen from the curve in Fig. 9 the coefficient of secondary consolidation shows a drastic drop close to the pre-consolidation pressure i.e $OCR = 1.2$ which complies with Alonso et al., [15].

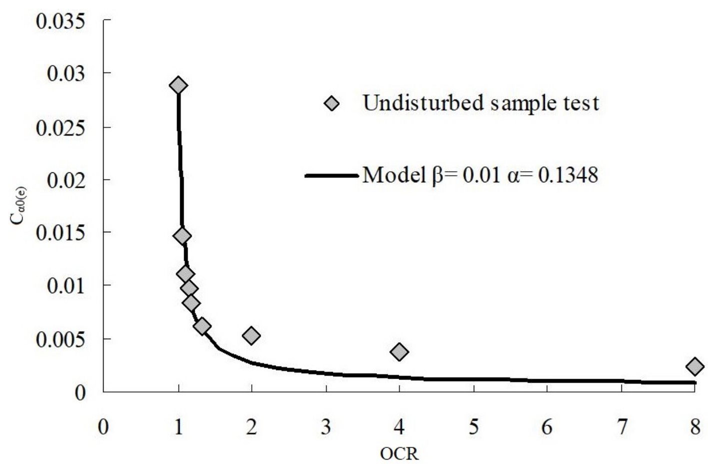


Figure 9. Creep test curve modified to suit the model

However, the proposed model is still valid for highly overconsolidated soft clays with a decrease in $C_{\alpha o}$ at a very slow rate with increasing OCR unlike that of Alonso's empirical formula that proposes an approximately constant $C_{\alpha o}$ for high values of OCR ($OCR > 1.5$) Fig. 10 which is unrealistic. The most important feature of the proposed model is the model parameters α and β can be fixed with only four testes where one of the tests is an ordinary consolidation test for normally consolidated clays and the rest three are special creep tests that involve unloading. This makes the model more efficient and economical due to the fact that, creep tests take long laboratory time Fig. 10.

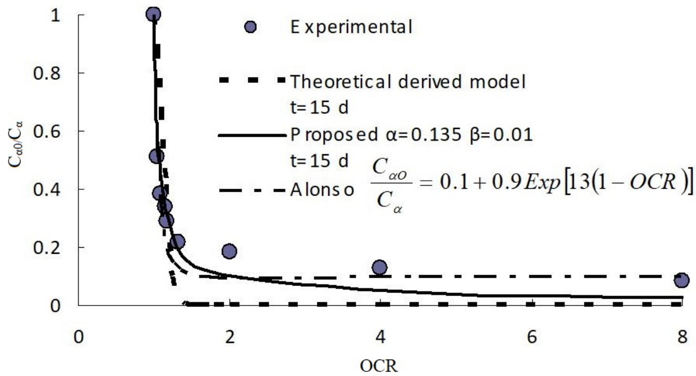


Figure 10. Comparison of proposed model and Alonso et al., [15]

The limitation of this proposed experimental model corresponds to low *OCR* values, indicating that the pre-consolidation stress during the initial consolidation stage is not significantly greater than the post-consolidation stress, specifically in this case, 1.2. In cases where the preloading stress is greater than the post-consolidation stress, the *OCR* value varies within a wider range, and the proposed formula by Alonso provides more appropriate results.

4. Conclutions

The effects of *OCR* on the creep behaviour of soft clay were studied by conducting laboratory creep tests in cyclic loading conditions. The test results conducted on undisturbed samples showed that the creep coefficient decreases with increasing *OCR* and there is no difference between reloading and unloading of creep test.

The creep coefficient of soft clay decreases drastically when *OCR* slightly exceeds 1. From practical and economic point of view, foundation soils are made lightly over consolidated during preloading to minimize the post construction settlement. But for the sake of completeness, the behavior of creep coefficient for highly over consolidated soils has been studied and the proposed model is still valid for the whole range of *OCR* with a slight deviation for $OCR > 2$. For the test duration in laboratory studies, the creep coefficient is constant with time which is different from the proposed theoretical model derived from the isotache model. Consequently, the theoretical model was modified by introducing two empirical parameters which can be calibrated by the laboratory creep test.

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