Geomechanics and Engineering, *Vol. 28, No. 2 (2022) 157-169*
https://doi.org/10.12989/gae.2022.28.2.157

Simplified analysis of creep for preloaded reconstituted soft alluvial soil from Famagusta Bay

Ali Hossien Basheer Garoushi* and Eris Uygar[a]

*Department of Civil Engineering, Engineering Faculty, Eastern Mediterranean University,
Famagusta, North Cyprus, via Mersin 10 Turkey*

(Received September 8, 2020, Revised December 2, 2021, Accepted December 7, 2021)

Abstract. Preloading of soft clays is a common ground stabilization method for improvement of compressibility and the undrained shear strength. The waiting period under preload is a primary design criterion controlling the degree of improvement obtained. Upon unloading the overconsolidation attained with respect to actual loads defines the long term performance. This paper presents a laboratory study for investigation of creep behavior of Famagusta Bay alluvial soft soil preloaded under various effective stresses for analysis of long term performance based on the degree of overconsolidation. Traditional one-dimensional consolidation tests as well as modified creep tests are performed on reconstituted soft specimens. Compressibility parameters are precisely backcalculated using one dimensional consolidation theory and the coefficient of creep is determined using the traditional Cassagrande method as well as two modified methods based on log cycles of time and the inflection of the creep curve. The test results indicated that the long term creep can be successfully predicted considering the proposed method. The creep coefficients derived as part of this method can also be related to the recompression index (recompression index, swelling index) considering the results of the testing method adopted in this study.

Keywords: creep; oedometer test; preloading; recompression index; secondary compression; soft soil

1. Introduction

For many years, creep deformation has been considered as a challenging problem for researchers (Yin 1999, Fox *et al.* 1999, Miao and Kavazanjian 2007, Suneel *et al.* 2008, Hong *et al.* 2010, Le *et al.* 2012, Fatahi *et al.* 2013, Azari *et al.* 2016, Zhu *et al.* 2016, Lei *et al.* 2016, Lei *et al.* 2018). In the literature, there are several theories attempting to explain the mechanism of creep such as; (a) breakdown of interparticle bonds (b) breaking of molecule bonds (c) sliding of soil particles (d) water flow from macro to microstructure and (e) the deformation due to structural viscosity (Mitchell 2005, Le *et al.* 2012, Fatahi *et al.* 2013, Yin 2015).

For saturated cohesive soils, the mechanism of creep can be explained based on the micromechanics of the settlement process (Yong and Warkentin 1966), which states that the initial orientation of the soil particles plays an important role in the compressibility behavior. Crawford (1964) have shown that the creep of Marine Clay onsets from the end of primary consolidation, which corresponds to a condition where the excess pore water pressure dissipated is equal to the applied stress increase. Using this approach Yong and Warkentin (1966) associated the primary consolidation with the breakdown of the soil structure (particle arrangements) and hence squeezing out of the resulting excess pore water pressure until an equilibrium state is reached. Based on this principle, following this stage is a further squeeze (creep), due to the reduction of pore space between individual particles, which occurs until the repulsion forces around the particles come into equilibrium under the applied load. Recently, several researchers have declared that the fundamental factors affecting the micromechanical properties of the cohesive soils are complex due to their structure, physicochemical processes, and stress history. (Yong and Warkentin 1966, Mesri *et al.* 1975, Leroueil *et al.* 1979, Burland 1990, Le *et al.* 2012, Azari *et al.* 2016)

Traditionally in practice, to simplify the analysis of settlement of saturated cohesive soils, creep settlement is considered to take place after primary consolidation is completed (Ladd 1978, Mesri and Choi 1985, Head 1998). However, there is also a common conception that the onset of creep settlement can commence earlier during the consolidation process (Bjerrum 1967, Ladd 1978, Leroueil *et al.* 1985, Fatahi *et al.* 2013). This is addressed commonly with reference to two major hypotheses namely; Hypotheses A and Hypotheses B (Jamiolkowski 1988, Mesri and Choi 1985, Kabbaj *et al.* 1988, Yin 1999, Aboshi 2004, Fatahi *et al.* 2013, Azari *et al.* 2016). These hypotheses can be classified simply in accordance with their assumption on the onset of creep as;

- Hypothesis A: the traditional simplified approach for creep assessment, which implies that the onset of creep is at the end of the primary consolidation.
- Hypothesis B: the creep occurs throughout the whole of the consolidation process onsetting from the beginning of loading.

---

*Corresponding author, Ph.D. Student
E-mail: bete1418@gmail.com
[a]Associate Professor
E-mail: eris.uygar@emu.edu.tr

ISSN: 2005-307X (Print), 2092-6219 (Online)

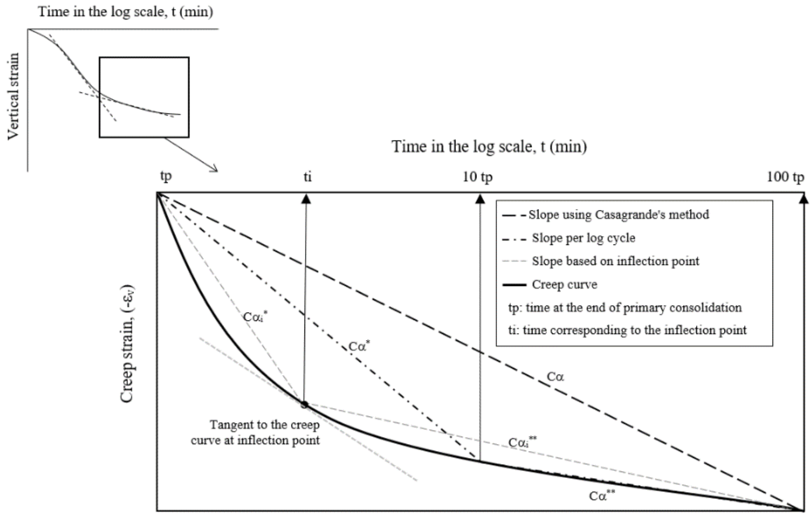


Fig. 1 Bilinear fitting for a consolidation curve

Although, mathematically well placed, the above definitions are not associated clearly with physical mechanisms and therefore, require further micromechanical investigations, which is not intended to be addressed by the authors in this study.

1.1 Variation of creep with vertical effective stress and overconsolidation ratio

Sridharan and Rao (1982) examined the mechanism of creep considering variation in void ratio, load increment ratio and pore fluid. They reported that the creep rate is indirectly proportional to the strength of the particle skeleton. In addition, they also found that a decrease in the dielectric constant of the pore fluid reduces creep as a result of an increase in the attractive forces as opposed to the repulsive forces. Nash *et al.* (1992) measured creep for both normally consolidated and overconsolidated specimens. They concluded that maximum creep occurs just after the yield stress (preconsolidation stress) in incremental loading tests stated to be associated with the breakdown of soil structure. These results are also confirmed by (Li *et al.* 2012, Luo and Chen 2014, Lei *et al.* 2016). More recently Wu *et al.* (2019) found that for reconstituted soft clay the maximum value of creep is a function of the void ratio at liquid limit and effective stress. They have also reported that the maximum creep rate occurs at a threshold stress of 2.17 yield stres beyond which a decrease is observed. This is interestingly in agreement with several other researchers who also reported that the maximum creep rate occurs approximately at twice the yield stress (Miao and Kavazanjian 2007, Suneel *et al.* 2008, Jesmani *et al.* 2012). However, it is important to note that in all these researches were the creep is assumed to be constant for a given effective stress and it is attained according to Hypothesis A.

1.2 Variation of creep with time

The variation of creep rate with time is studied by (Barden 1969), who reported that the creep isn't linear in a plot of creep compression versus log time plot. Their work is also confirmed later by other studies such as: Mesri *et al.* (1975), Mesri and Choi (1985), Yin and Graham (1989), Fox *et al.* (1992), Yin (1999), Yin (2006), Miao and Kavazanjian (2007), Hong *et al.* (2010), Le *et al.* (2012), Fatahi *et al.* (2013), Azari *et al.* (2016), Chen *et al* .(2014), Yin (2015), Lei *et al.* (2016), Zhu *et al.* (2016), Mesri *et al.* (2017), Wu *et al.* (2019). The presented methods for modeling creep in some cases of these studies are complex and difficult to apply in practice effectively.

One of the pioneering work in modeling of creep is presented by Yin and Graham (1989) who formulated an elastic-plastic model based on hypothesis B. In their constitutive model, the creep strain assumed to be linear for a given effective stress. However, Yin (1999) reported a limitation for this model, reporting an error in the prediction for the determination of long term compressibility. This error is reported to be due to the allowance of time to approach infinity, which makes the vertical strain to approach infinity as well, causing overestimation of creep strain. Yin *et al.* (2002) suggested a modification for the nonlinear equation formed for the creep strain in which zero creep strain rate is achieved at infinity. Wu *et al.* (2013) derived a non linear creep function based on hypothesis B by relating the coefficient of creep to void ratio and vertical effective stress. Recently, Le *et al.* (2015) used a unique numerical optimization procedure called Crank–Nicolson finite-difference solution to develop a one-dimensional elastic visco-plastic model. Le *et al.* (2015) stated that, although the nonlinear functions are useful in describing the compressibility behavior of soils, the estimation of the parameters for such functions isn't an easy task. However, as highlighted in many studies, amongst other factors the effective stress, overconsolidation ratio (OCR) and change of creep rate with time are crucial for the estimation of long-term settlement, which are yet to be captured completely in the available models (Walker 1969, Nash *et al.* 1992, Miao and Kavazanjian 2007, Suneel *et al.* 2008, Karunawardena *et al.*2011, Jesmani *et al.* 2012, Luo and Chen 2014, Lei *et al.* 2016, Alibrahim and Uygar 2021a).

Dhowian and Edil (1980) have shown the uncommonly curved shape of creep compression for peats, which was later addressed by Robinson (2003) and Gofar and Sutejo (2007), who indicated that the creep compression may be difficult to distinguish from the primary consolidation phase. However, Robinson (2003) and Mckinley and Sivakumar (2009) later concluded that as long as the load increment ratio of 1 or greater is used in the one dimensional oedometer testing, the creep compression is likely to onset after the completion of primary consolidation for inorganic soils. Robinson (2003) also stated that the creep compression has a nonlinear character and provided a hyperbolic fit equation for estimation of creep versus time relationship.

Sridharan and Prakash (1998) proposed a secondary compression factor method to obtain creep rate, this method is based on a log void ratio versus log time plot, along which a linear curve can be fitted to provide a practical means of obtaining creep rate.

The traditional approach or a slighlty modified version of obtaining creep is discussed frequently in the literature and also in practice such as (Sridharan and Prakash 1998, Fox 2003, Mckinley and Sivakumar 2009). In all these studies the nonlinearity of creep with logarithm of time is acknowledged and partially addressed in the analyses. However, in none of the previous works, a practical means of defining changing rate of creep with time is presented. In order to provide a simplified approach for assessing the creep versus time relationship with reasonable accuracy, the factors such as; effective stress, OCR and long term creep data need to be considered. In this study, a new practical bilinear fitting procedure is presented and the results are compared with the traditional Casagrande (1936) method. In the proposed method, the creep rate is represented with two linear slopes, which can be performed on a typical oedometer compression data obtained during any load incrementas shown in Fig. 1.

The creep data is the compression observed after the completion of the primary consolidation, which is analysed based on Hypothesis A. This point along the consolidation curve is used as the start of the bilinear fit, which breaks and changes slope at the inflection point along the creep curve. In an alternative approach, the bilinear fit is defined to be between the time intervals tp - 10tp (first log cycle after tp) and 10tp - 100tp (second log cycle after tp). For comparison, an overall fit to the data considering time up to two log cycles is also performed (Casagrande 1936).

The proposed fitting method simplifies the process of obtaining the creep rate avoiding the difficulties of assigning parameters, but at the same time, it allows for an analysis based on the change in rate of creep with time by means of a simple graphical method which can be used effectively in practice.

1.3 Backanalysis of oedometer data for estimation of compressibility parameters

The testing strategy involves preconsolidation of the soft cohesive soil specimens to various effective stresses and unloading, after which a one-step effective stress increment is performed to observe consolidation settlement and creep behaviour under various degree of overconsolidation. Hence, the oedometer consolidation curves obtained can be fitted with a backanalysis method based on the one-dimensional consolidation theory (Terzaghi 1943) to obtain the primary consolidation and recompression index. The backanalysis of the test data involved iteration of the calculation steps outlined in the folowing:

Step 1: Time required for 100% consolidation to take place, t_p, and the primary consolidation settlement is obtained graphically by Casagrande Log time method. The recompression index is backcalculated considering that the soil specimens are preconsolidated prior to compression using Eq. (1)

$$\delta_p = \frac{C_s\ H_0}{(1+e_0)} log\ (\frac{\sigma_0'+\Delta\sigma'}{\sigma_0'}) \tag{1}$$

where, C_s is recompression index; e_0 is void ratio prior to application of the effective stress increase; σ_0' is the intial effective stress; $\Delta\sigma'$ is the effective stress increase; H_0 is the half average height of the specimen for a particular load increment

Step 2: Then the time required to achieve a certian degree of consolidation is calculated using Eq. (2) for a given set of data points as

$$t = T_v\ \frac{(H_0/2\ )^2}{C_v} \tag{2}$$

where, $H_0/2$ = longest drainage height prior to application of the effective stress increase; C_v = is coefficient of consolidation, ; T_v = is the time factor corresponding to a certain degree of consolidation considering uniform dissipation of excess pore water pressure in double drainage condition calculated using Eqs. (3) and (4)

$$T_{v_i} = \frac{\pi}{4{U_i}^2} \qquad \text{For } U \leq 0.6 \tag{3}$$

$$T_{v_i} = 1.781\text{-}0.933\ log\ \ (100\ -\ U_i \times 100) \qquad \text{For } U > 0.6 \tag{4}$$

where, U_i = the average degree of consolidation given for a set of data points to develop the backcalculated curve for data fitting procedure.

Step 3: Finally using T_{v_i} obtained for the data set, corresponding consolidation settlement, δ_{v_i} at each data point is obtained using Eq. (5) using the primary consolidation obtained in Step 1 as

$$\delta_{v_i} = U_i\ \times \delta_p \tag{5}$$

and the vertical strain, ε_{v_i} is then calculated as

$$\varepsilon_{v_i} = \frac{\delta_{v_i}}{H_0} \tag{6}$$

2. Materials and methods

2.1 Soil sampling

The soil used in this research is obtained from the

Table 1 The physical properties of the tested alluvial soil

Properties	Value
Specific gravity	2.65
Liquid limit (%)	61.0
Plastic limit (%)	31.0
Plasticity index (%)	30.0
In situ moisture content (%)	33.5
Sand (> 0.075) content (%)	2.60
Silt (0.005 - 0.075) content (%)	43.4
Clay (< 0.005) content (%)	54.0
Optimum water content at Standard Proctor Effort (%)	26.5
Maximum density at Standard Proctor Effort (g/cm³)	1.54

Table 2 Test groups and loading scheme

Specimen group	Specimen type	Test type	Initial water content (%)	Initial void ratio	Preconsolidation stress (kPa)	Creep test stress (kPa)	Representative OCR for creep test
GR 1	Soft	SOT	61.0	1.5853	-	-	-
GR 1-1-1	Soft	CT	60.1	1.5793	50	12.5	4
GR 1-1-2	Soft	CT	60.1	1.5665	50	25	2
GR 1-1-3	Soft	CT	60.1	1.5725	50	37.5	1.3
GR 1-1-4	Soft	CT	60.1	1.5681	50	50	1
GR 1-2-1	Soft	CT	60.5	1.5803	100	25	4
GR 1-2-2	Soft	CT	60.5	1.6270	100	50	2
GR 1-2-3	Soft	CT	60.5	1.5910	100	75	1.3
GR 1-2-4	Soft	CT	60.5	1.5818	100	100	1
GR 1-3-1	Soft	CT	61.0	1.6114	300	75	4
GR 1-3-2	Soft	CT	61.0	1.5903	300	150	2
GR 1-3-3	Soft	CT	61.0	1.7264	300	225	1.3
GR 1-3-4	Soft	CT	61.0	1.6126	300	300	1

Alluvial deposits extensively present along the Famagusta Bay alluvial soft soil, Cyprus (Golhashem and Uygar 2019, Golhashem and Uygar 2020, Alibrahim and Uygar 2021b). The soil samples are obtained from approximately a depth of 2 m below the ground level to enable sampling below the organic soil cover. The index properties of the soil specimens tested are summarized in Table 1.

2.2 Testing strategy

Testing involved one main group of specimens; (GR 1) soft specimens. The test group is subjected to standard oedometer testing (SOT) in the first stage of testing in accordance with (ASTM D 2435, 2011). Then, three sub-groups are formed by application of various preconsolidation stages. Finally for these subgroups, creep tests (CT) are performed under constant effective stress to achieve various degree of consolidation at the end of test as; OCR= 1, OCR= 1.3, OCR= 2 and OCR= 4. Therefore, the testing program allowed for comparison of the creep behaviour of the soft soil specimens under various effective stresses and state of degree of consolidation.

Standard oedometer tests

SOT is performed to measure the complete compressibility curve of all groups. The initial consolidation stress of 5.2 kPa is applied prior to any load increment to ensure that the specimens are fully saturated. The consolidation stresses are applied as; 25- 50- 100- 200- 400-200-100- and 50 kPa).

Creep tests

The creep measurements in the standard oedometer testing are considered to be affected by the incremental loading sequence, which ensures a different effective stress is achieved at the end of each load step. In order to focus on creep behaviour for a state of preconsolidation stress σ_p' and OCR, a creep test program is designed as presented in Table 2. The preconsolidation stress is applied immediately after the saturation stage for a 24hr period to ensure dissipation of excess pore water pressures, and then specimen is unloaded for a 24hr period prior to application of CT.

For preparation of subgroups, the preconsolidation stresses applied are; GR 1-1=50 kPa, GR 1-2=100 kPa, and

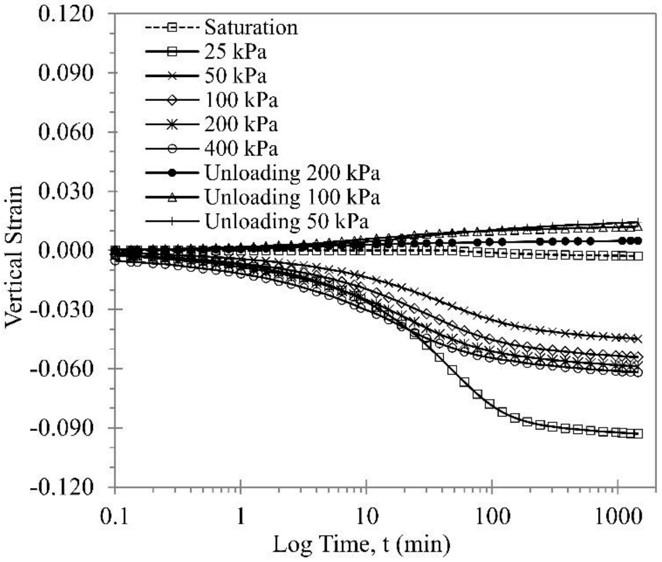


Fig. 2 Compression curves obtained in SOT

GR 1-3=300 kPa, which are selected to cover a practical range of effective stresses for foundations and embankment loading on clay soils. In all of these subgroups, four identical specimens are then loaded to an effective stress for CT to attain a match for a certain degree of overconsolidation as; OCR= 1, OCR= 1.3, OCR= 2 and OCR= 4. In creep test (CT), to ensure that the initial state of the specimens are identical, the saturation stage is continued until all specimens reach approximately to the same void ratio as they are compressed.

CT is applied for a period of 7 days to ensure that sufficient data is obtained for the analysis of variation of creep rate with respect to time. In CT, the effective stress increase is applied in one load step to avoid inclusion of the impact of effective stress build up (i e. effect of construction rate) in the creep behaviour.

2.3 Specimen preparation

In order to produce identical specimens, a cylindrical mold made of steel consisting two parts (split collar and base) is used. The inner diameter and the height of the mold are 150 mm is 60 mm, respectively. The base is a 300 mm square plate and it is manufactured to be attached and detached easily to the collar.

The soft specimens are prepared following a similar procedure with the methodology outlined in Burland (1990). The soil is used in its natural state without any drying or pulverizing. The mold is used to mix the soil with distilled water, targeting liquid limit water content to form a slurry with viscous consistency. Then, the mold is gently tapped from the bottom to minimize air bubbles and kept in a vacuum desiccator for 24 hours. After this period, a total of four oedometer specimens are extracted for testing with the use of a thin specimen tube with a wall thickness of approximately 0.5mm to reduce the disturbance to the specimens. Then, while extraction and trimming a flexible spatula and a thin wire saw are used.

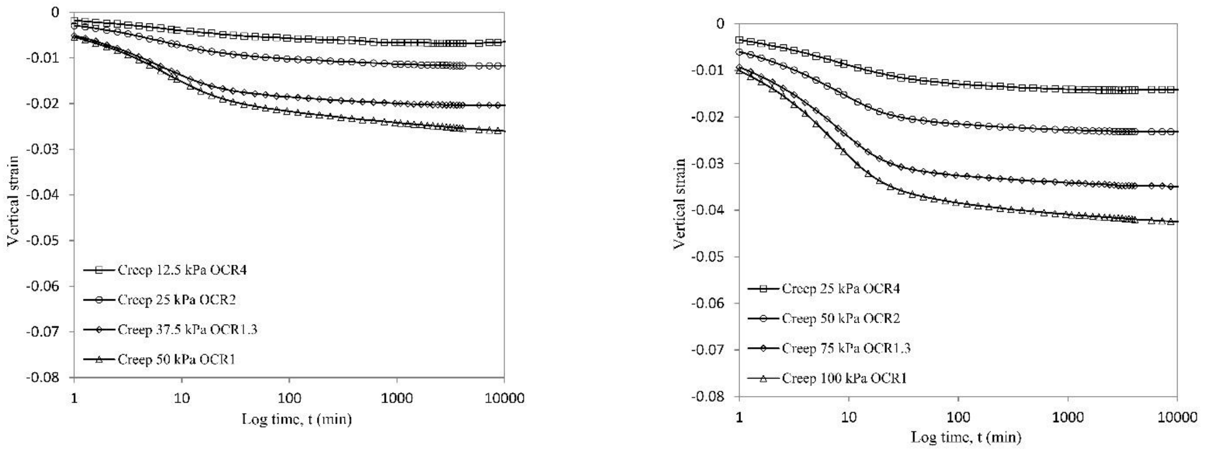
3. Results and discussion

3.1 Standard oedometer test (SOT)

The results are presented in Fig. 2. All were compressed in the saturation stage. The vertical strain is calculated separately for each stage of the tests allowing comparison of the relative change in behaviour with respect to loading and unloading. The compression attained is observed to reduce with increase in the effective stress. However, in order to make accurate conslusions on compressibility behaviour one should differentiate the completion of primary consolidation and creep (secondary consolidation) from each other. This is due to the fact that the test is carried out on the basis of a waiting period of 24hr after each loading step, which undoubtedly involves part of creep settlement, incrementally added on the total compression observed (ASTM D 2435 2011). The creep settlement not only affects the magnitude of the settlement observed numerically, but also has an impact on the compressibility behaviour of the specimen that is observed in the next loading step due to the densification (aging) it causes. The cumulative effect of aging due to creep is observed as a drop in the compressibility with the increase in the effective stress. As a result of the specimen characteristics, soft specimen didn’t reveal a yield stress, hence the loading steps applied indicated almost a linear progression of compressibility behaviour with respect to logarithm of effective stress.

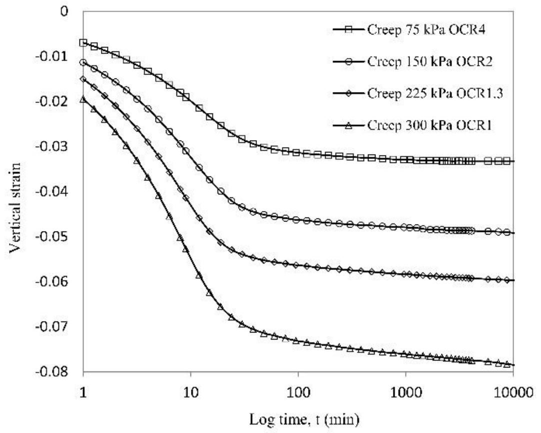
3.2 Creep test (CT)

The creep test results for the soft specimens are presented in Figs. 3(a)-3(c). It is observed that the slope of vertical strain curve is initially greater, gradually decreasing with time, reflecting a highly nonlinear progression. In addition, there is a decreasing trend in the magnitude and rate of creep observed as the OCR is increased. This is in a



(a) CT results for specimens preconsolidated to an effective stress of 50 kPa

(b) CT results for specimens preconsolidated to an effective stress of 100 kPa



(c) CT results for specimens preconsolidated to an effective stress of 300 kPa

Fig. 3 (a)-(c) The creep test curves

good agreement with the results observed previously by Nash *et al*. (1992), Suneel *et al*. (2008), Azari *et al*. (2014) and Luo and Chen (2014). It is interesting to note that for OCR= 4 the rate of creep compression is reduced significantly at 10000 minutes. The influence of effective stress on the creep rate is such that, for the same OCR, as the effective stress is increased the nonlinearity of the creep curve is also increased, indicating a greater reduction in the creep rate.

The rate of creep is reduced due to allowance for reconfiguration of their fabric with the provision of greater space in their interlayer regions. The high compressibility of the soft specimens is likely to have led to a significant improvement of their initial compressibility behaviour during preconsolidation stage, which affected their initial response.

3.2.1 The relationship between coefficient of secondary compression with effective stress and OCR

The results from Cassagrande's method of estimation for coefficient of creep from secondary compression curves are presented in Fig. 4. It is evident that the creep rate drops significantly with respect to OCR and there is an indirect relationship between creep rate and the effective stress.

When the creep rate is considered as a function of the log cycles Fig. 5, it is observed that this drop in the creep rate is continuous.

In Fig. 5 the data indicate that the drop in creep rate with respect to OCR is similar in the first and second log cycles. However, most of the drop occurs between OCR:1 and OCR: 2 and thereonwards variaton is contained within a tighter range especially in the second log cycle. Overall, the test results highlighted a significant relationship between stress history and creep rate. In the dataset obtained, the bilinear fit showed that Cassagrande's method might underestimate the secondary compression in the first log cycle with a factor of 0.5 to 0.8 for OCR:4 to 1, respectively, whereas it might overestimate it in the second log cycle with a factor of 1.3 to 1.6 within the same range of OCR. Hence, considering predictions in a preloading project for ground stabilisation early on in a design scenario, the differences attained are quite significant to rely on for prediction of long term behaviour.

The bi-linear fitting approach considering log-cycles of time during creep is useful and quite practicable for defining nonlinearity in rate of creep, such that the equation for calculating creep settlement (δ_{creep}) for time period greatear than one log cycle after the primary consolidation

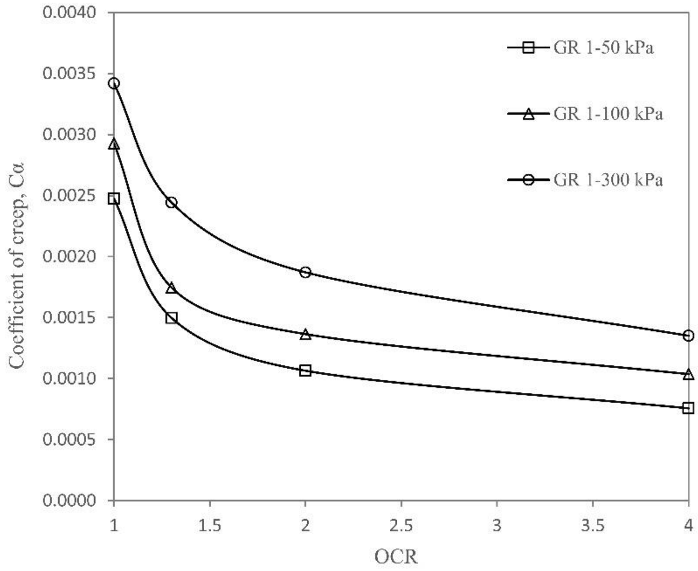


Fig. 4 Coefficient of creep obtained as a single slope using Cassagrande's method with respect to OCR

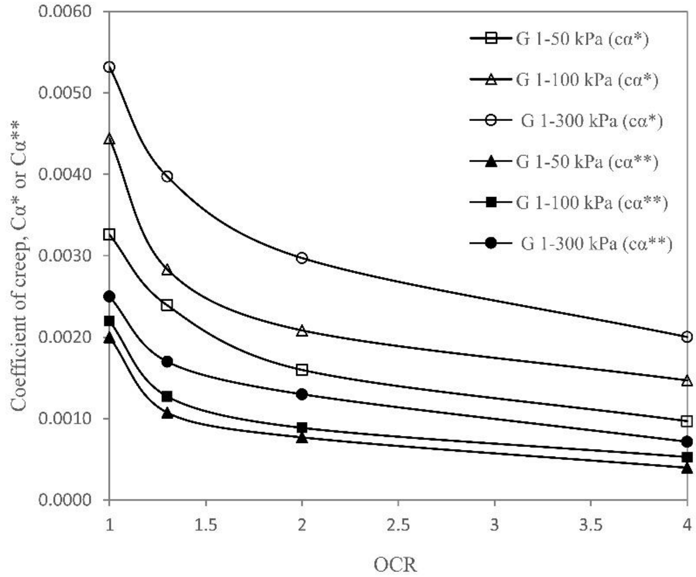


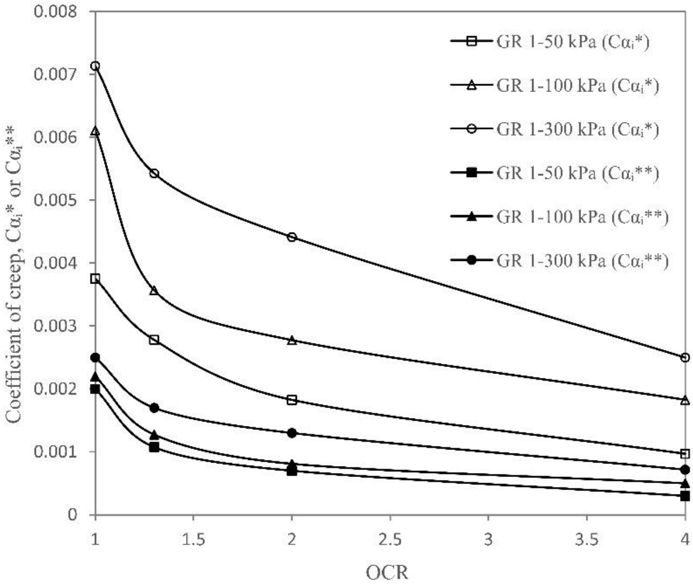
Fig. 5 Coefficient of creep obtained from bilinear fitting to first (C_α^*) and second log cycles (C_α^{**}) of time with respect to OCR

is completed can be given as

$$\delta_{creep} = H_c C_\alpha^* log\frac{10t_p}{t_p} + H_c C_\alpha^{**} log\frac{t}{10t_p} \qquad (7)$$

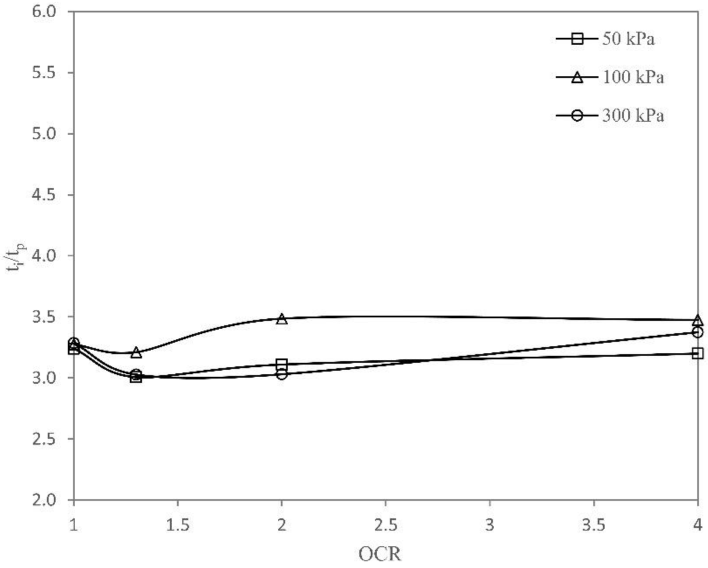
where; H_c: thickness of the clay, t_p: time required for completion of primary consolidation settlement, t: time at which creep is calculated ($t > t_p$).

It is clearly seen from here that, consideration of a simple approach such as bilinear fitting based on the log cycles of time alone can help provide a significant improvement on the creep settlement predicted, which for the above example given, resulted to a drop of 25% for the results of soft specimens, compared to the case where creep is taken as constant in log time scale (i.e., C_α^*). However, it is also evident from the creep curves that one might obtain a more accurate estimate on change of rate of creep with time, should the fitting is applied based on the inflection observed along the creep versus time curves. The results of the bi-linear fitting considering the inflection points along the creep curves are presented in Fig. 6, which indicates that the rate of creep is significantly greater up to the inflection point along the creep curves and vary afterwards depending on the effective stress and OCR. It can be stated that as the effective stress and OCR is increased, the creep rate is observed to be greater after the inflection point ($C^{**}_{\alpha_i}$) compared to the assessment carried out using the log cycles approach. Where the nonlinearity of the creep curves are



8

Fig. 6 Coefficient of creep obtained from bilinear fitting based on inflection point along creep curves with respect to OCR



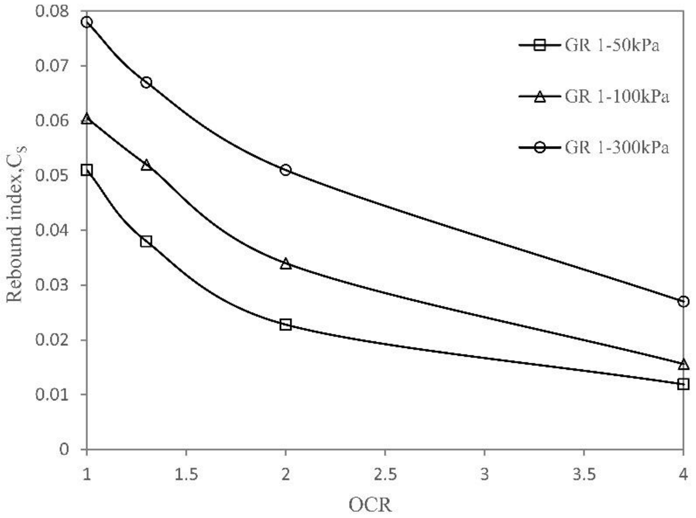
8

Fig. 7 The ratio of t_i/t_p with respect to effective stress and OCR

less, the two bi-linear fit approaches used seem to converge. For such cases, it can also be stated that the Cassagrande approach can also provide a good fit.

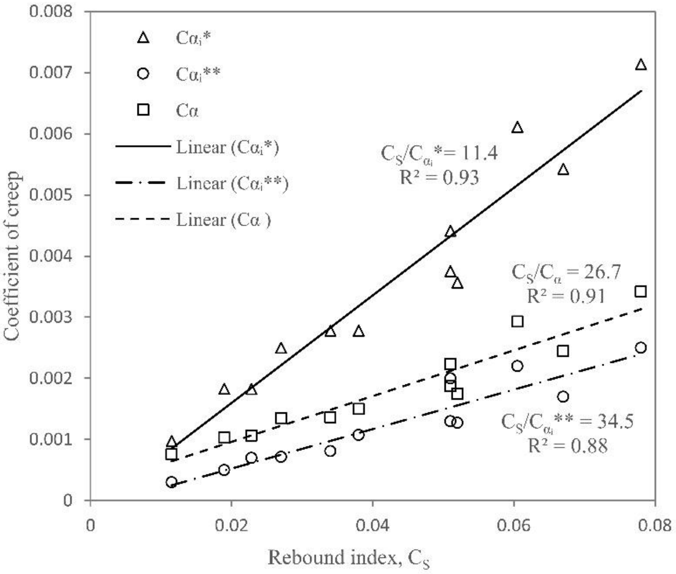
Although, the bi-linear fitting based on inflection point along the creep curve provides a more accurate prediction of creep rate, one would argue its appicability in the absence of long term data. The key information in this procedure which is the estimation of the inflection point can not be determined without long term data. However, it is observed from the dataset obtained that it is likely to develop a relationship between the change of creep rate (in terms of time required to attain inflection in the creep curve, t_i) with compressibility characteristics namely; the time required for completion of primary consolidation, t_p and recompression index, C_S (also known as rebound index or swelling index). The former of these two parameters is likely to have strong links with the creep rate as both are affected by drainage characteristics, whereas the latter has the advantage of including the influence of preconsolidation stress. The data obtained from the analysis of the relationship between t_p and t_i is presented in Fig. 7. An almost asymptotic relationship can be observed, which seems to be independent of the effective stress and OCR, for soft specimens, indicating a t_i/t_pratio of approximately 3.0-3.5. Therefore, Eq. (7) can be further modified as

$$\delta_{creep} = H_c C^{*}_{\alpha_i} log \frac{t_i}{t_p} + H_c C^{**}_{\alpha_i} log \frac{t}{t_i} \quad (8)$$



8

Fig. 8 The effect of OCR on the recompression index



8

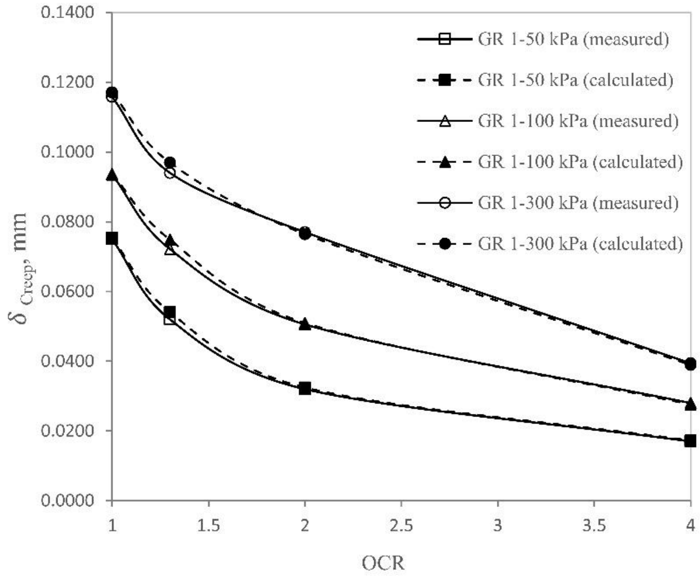
Fig. 9 The relationship between the coefficient of creep and recompression index

3.3 Relationship between coeffients of creep and recompression index (C_S)

Fig. 8 presents the effect of OCR and σ_p' on the recompression index (which is obtained from the results of the same set of specimens using the methodology described in section 1.2). It is evident that both of these parameters induce a reduction in the recompression index, which shows reduction in the comressibility due to overconsolidation achieved. The effective stress applied during testing had a remarkable impact as well with a tendency of reflecting a lower recompression index when the applied effective stress is lower, which is considered to be in accordance with the stress history concept in soil mechanics.

Further to the above, the relationship between the coefficient of creep and recompression index C_S is also presented in Fig. 9. In spite of the scattered data, a directly proportional linear relationship can be clearly observed between these paramters, similar observation has been found by *(Deng et al.* 2012) . This relationship is obtained for the creep coefficients defined for the modified method in this study as well as for the traditional creep coefficient for comparison, as displayed with regression results in Fig. 9.

Based on these findings the Eq. (8) can be updated to be written in terms of the recompression index, which is a paramter that can be obtained in a typical oedometer test.



8

Fig. 10 A comparison between the measured creep and calculated creep

$$\delta_{creep} = \; H_c \frac{C_s}{a} log \frac{t_i}{t_p} + H_c \frac{C_s}{b} log \frac{t}{t_i} \qquad (9)$$

In Eq. (9), coefficients a and b are obtained through the relationships in Fig. 9, where a corresponds to $C_S/C^*_{\alpha_i}$ and b corresponds to $C_S/C^{**}_{\alpha_i}$. For the tested (Famagusta Bay) alluvial soft soil, these coefficients are observed to be related such that *a/ b = 0.33*, which leads to the following modification of Eq. (9) (also considering that average $t_i/t_p = 3.25$)

$$\delta_{creep} = \; H_c \frac{C_s}{a} log \frac{3.25\ t_p}{t_p} + H_c \frac{C_s}{3.03\ a} log \frac{t}{3.25\ t_p} \qquad (10)$$

$$\delta_{creep} = \; H_c \frac{C_s}{a}\left(0.51188 + 0.33\ log\ \frac{t}{3.25\ t_p}\right) \qquad (11)$$

In order to test the proposed simplified method of estimating creep settlement Eq. (9), the laboratory creep test results obtained in this study are used. The methodology suggested is expressed in general form in Eq. (9), however, it is also modified to suit for the purposes of backcalculating creep settlement for this study in Eqs. (10) and (11). The simplified methodology can be used to obtain long term creep using laboratory results provided that the inflection point along the creep curve is exceeded during the test. Fig. 10 displays a comparison between the measured creep settlement and calculated creep from Eq. (11) with respect to OCR for time t = 10000 minutes, which indicated an excellent fit based on the data from laboratory creep tests.

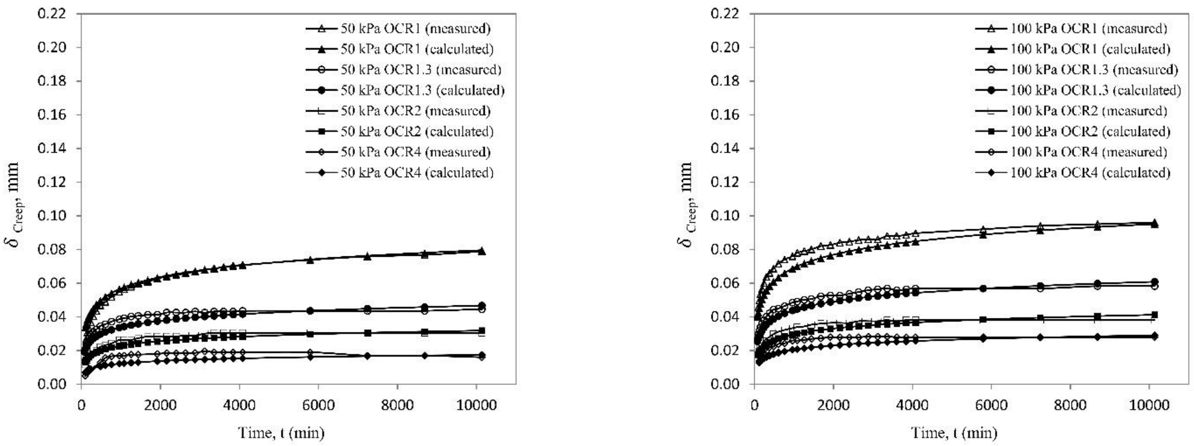
Comparisons between the measured and calculated creep settlement with respect to time are plotted in Fig. 11, in which Eq. (11) is employed. The results have shown that, a reasonable match to the general trend of the creep settlement is achieved at any time, regardless of the stress history of the soil and the applied effective stress. The nonlinearity of the creep curve is also well accounted with Eq. (11), which has a simple expression well connected to the other parameters of the compressibility curve. In the case of preloading works for insitu ground modification, this form of creep settlement equation is likely to be very useful for long term settlement predictions after removal of surcharge and reloading due to planned construction works. The equation can also be implemented to provide fitting curve for oedometer test results.

The proposed prediction equation for soft soil sample tested would be considered for all soft cohesive soils, however, the relationship between the inflection point along the creep curve and the time for the completion of primary consolidation will have to be confirmed with laboratory testing. In addition, the ratios of recompression index to the creep curve slopes defined are also likely to be specific to the properties of the soil under consideration. Regardless of these however, the methodology defined for expressing the nonlinear creep settlement curve is a simple, practical and efficient means of providing settlement predictions for long term, which is most of the times a challenging task in geotechnical design.

4. Conclusions

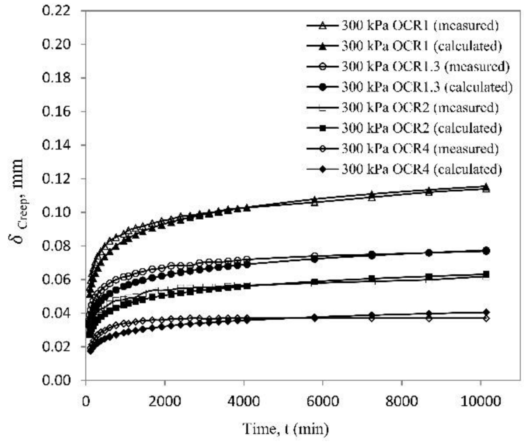
In this paper, one-dimensional consolidation and creep (secondary compression) behavior of soft alluvial clay of Famagusta Bay, Cyprus, are investigated. The paper also proposes a fitting method following a simplified process for claculation of nonlinear creep settelement with respect to time.

- A new test method employing one-dimensional form of creep testing is designed, which enables measurement of settlement behavior of preconsolidated soft clay, simulating preloading, unloading and reloading stages in ground modification projects. The nonlinear nature of the creep



(a) GR 1.1, soft soil specimen

(b) GR 1.2, soft soil specimen



(c) GR 1.3, soft soil specimen

Fig. 11 A comparison between the measured creep and calculated creep with respect to time

settlement curve is analysed with a modified simple graphical approach to define change in creep slope with time and equations for prediction of creep settlement in the long term are developed.

- The test results indicated that, amongst other factors, the creep rate is dependent on the stress history, the applied effective stress and the soil structure. The creep curves reflected a nonlinear progression, which can be simply represented using a bilinear fit. The bi-linear fitting approach considerıng log-cycles of time during creep is useful and quite practicable for defining nonlinearity in the rate of creep. The influence of effective stress on the creep rate is such that, for the same OCR, as the effective stress is increased the nonlinearity of the creep curve is also increased, indicating a greater reduction in the creep rate.
- For soft specimens, the time corresponding to the inflection point strongly depends on the time corresponding to the end of primary consolidation at which t_{i/t_p} ranges from 3 to 3.5.
- The recompression index is observed to have a strong relationship with the creep coefficients. A directly proportional linear relationship between the recompression index and creep coefficients are obtained, which enabled formulation of the creep curve using the recompression index. This is quite significant as one can deduct the recompression index parameter easily using the field or laboratory data for ground modification works employing preloading. Hence, in such a case, the short term data on recompression index and the methodlogy used in this paper can be used to make long term predictions.

References

Aboshi, H. (2004), “Long-term effect of secondary consolidation on consolidation settlement of marine clays”, *Proceedings of the Advances in Geotechnical Engineering: The Skempton Conference - Proceedings of a Three Day Conference on Advances in Geotechnical Engineering, Organised by the Institution of Civil Engineers*, London, UK, on 29-31 March 2004.

Alibrahim, B. and Uygar, E. (2021a), “Nonlinear calculation method for one-dimensional compression of soils”, *Arab. J. Sci. Eng*., https://doi.org/10.1007/s13369-021-06270-7.

Alibrahim, B. and Uygar, E. (2021b), “Influence of compaction method and effort on electrical resistivity and volume change of cohesive soils”, *KSCE J. Civ. Eng.*, **25**(7), 2381-2393. https://doi.org/10.1007/s12205-021-0419-9.

ASTM D 2435. (2011), “Standard test methods for one-dimensional consolidation properties of soils using incremental loading", *The Annual Book of ASTM Standards*.

Azari, B., Fatahi, B. and Khabbaz, H. (2016), "Assessment of the elastic-viscoplastic behavior of soft soils improved with vertical drains capturing reduced shear strength of a disturbed zone", *Int. J. Geomech*., **16**(1), B4014001. https://doi.org/10.1061/(ASCE)GM.1943-5622.0000448.

Barden, L. (1969), "Time dependent deformation of normally consolidated clays and peats", *J. Soil Mech. Found. Div*., **95**(1). https://trid.trb.org/view/127300.

Bjerrum, L. (1967), "Engineering geology of Norwegian normally-consolidated marine clays as related to settlements of buildings", *Geotechnique*., **17**(2), 83-118. https://doi.org/10.1680/geot.1967.17.2.83.

Burland, J.B. (1990), "On the compressibility and shear strength of natural clays", *Geotechnique*, **40**(3), 329-378. https://doi.org/10.1680/geot.1990.40.3.329.

Casagrande, A. (1936), "The determination of pre-consolidation load and its practical significance", *Proceedings of the 1st International Conference on Soil Mechanics and Foundation Engineering*, Cambridge, Mass.

Chen, X., Luo, Q. and Zhou, Q. (2014), "Time-dependent behaviour of interactive marine and terrestrial deposit clay", *Geomech. Eng*., **7**(3), 279-295. https://doi.org/10.12989/GAE.2014.7.3.279.

Crawford, C.B. (1964), "Interpretation of the consolidation test", *J. Soil Mech. Found.*, **91**(5), 146-147.

Deng, Y.F., Cui, Y.J., Tang, A.M., Li, X.L. and Sillen, X. (2012), "An experimental study on the secondary deformation of Boom clay", *Appl. Clay Sci*., **59**, 19-25. https://doi.org/10.1016/j.clay.2012.02.001.

Dhowian, A.W. and Edil, T.B. (1980), "Consolidation behavior of peat", *Geotech. Test. J*., **3**(3), 105-114. https://doi.org/10.1520/GTJ10881J.

Fatahi, B., Le, T.M., Le, M.Q. and Khabbaz, H. (2013), "Soil creep effects on ground lateral deformation and pore water pressure under embankments", *Geomech. Geoeng*., **8**(2), 107-124. https://doi.org/10.1080/17486025.2012.727037.

Fox, P.J. (2003), Consolidation and Settlement Analysis, The Civil Engineering Handbook 2, (Eds., Chen, W.F. and Liew, J.Y.R.), Washington, D.C, USA.

Fox, P.J., Roy-Chowdhury, N., Edil, T.B., Juárez-Badillo, E., Mesri, G., Stark, T.D. and Chen, C.S. (1999), "Discussions and closure: secondary compression of peat with or without surcharging", *J. Geotech. Geoenviron. Eng*., **125**(2), 160-165. https://doi.org/10.1061/(ASCE)1090-0241(1999)125:2(160).

Fox, P.J., Edil, T.B. and Lan, L.T. (1992), "Cα/Cc concept applied to compression of peat", *J. Geotech. Eng*., **118**(8), 1256-1263. https://doi.org/10.1061/(ASCE)0733-9410(1992)118:8(1256).

Gofar, N. and Sutejo, Y. (2007), "Long term compression behavior of fibrous peat", *Malaysian J. Civil Eng*., **19**(2), 14-26.

Golhashem, M.R. and Uygar, E. (2019), "Improvement of internal stability of alluvial clay from Famagusta Bay, Cyprus, using copolymer of butyl acrylate and styrene", *Environ. Eng. Geosci*., **25**(4), 289-300. https://doi.org/10.2113/EEG-2205 .

Golhashem, M.R. and Uygar, E. (2020), "Volume change and compressive strength of an alluvial soil stabilized with butyl acrylate and styrene", *Constr. Building Mater*., **255**, 119352. https://doi.org/10.1016/j.conbuildmat.2020.119352.

Head, K.H. (1998), Manual of Soil Laboratory Testing: Effective Stress Tests, John Wiley & Sons Ltd, Chichester.

Hong, Z.S., Yin, J. and Cui , Y.J. (2010), "Compression behaviour of reconstituted soils at high initial water contents", *Geotechnique*, **60**(9), 691-700. https://doi.org/10.1680/geot.09.P.059.

Jamiolkowski, M. (1988), "New developments in field and laboratory testing of soils", *Proceedings of the 11th international conference on soil mechanics and foundation engineering*, San Francisco, California, USA, August.

Jesmani, M., Vaezi , R. and Kamalzarem, M. (2012), "Correlation between Cα/ Cc ratio and index parameters of soils", *Q. J. Eng. Geol. Hydroge*., **45**(2), 207-220. https://doi.org/10.1144/1470-9236/09-060.

Kabbaj, M., Tavenas, F. and Leroueil, S. (1988), "In situ and laboratory stress-strain relationships", *Geotechnique*, **38**(1), 83–100. https://doi.org/10.1680/geot.1988.38.1.83.

Karunawardena, A., Oka, F. and Kimoto, S. (2011), "Elasto-viscoplastic modeling of the consolidation of Sri Lankan peaty clay", *Geomech. Eng*., **3**(3), 233–254. https://doi.org/10.12989/gae.2011.3.3.233.

Ladd, C.C., Foott, R. and Ishihara, K. (1978), "Stress-deformation and strength characteristics. state-of-the-art report", *Int. J. Rock Mec. Min. Sci. Geomech. Abstracts*, **15**(2), 421-494. https://doi.org/10.1016/0148-9062(78)91692-3.

Le, T.M., Fatahi, B. and Khabbaz, H. (2012), "Viscous behaviour of soft clay and inducing factors", *Geotech. Geological Eng*., **30**(5), 1069-1083. https://doi.org/10.1007/s10706-012-9535-0.

Le, T.M., Fatahi, B. and Khabbaz, H. (2015), "Numerical optimisation to obtain elastic viscoplastic model parameters for soft clay", *Int. J. Plasticity*, **65**, 1-21. https://doi.org/10.1016/j.ijplas.2014.08.008.

Lei, H., Feng, S. and Jiang, Y. (2018), "Geotechnical characteristics and consolidation properties of Tianjin marine clay", *Geomech. Eng*., **16**(2), 125-140. https://doi.org/10.12989/gae.2018.16.2.125.

Lei, H., Wang, X., Chen, L., Huang, M. and Han, J. (2016), "Compression characteristics of ultra-soft clays subjected to simulated staged preloading", *KSCE J. Civil Eng*., **20**(2), 718-728. https://doi.org/10.1007/s12205-015-0343-y.

Leroueil, S., Kabbaj, M., Tavenas, F. and Bouchard, R. (1985), "Stress-strain-strain rate relation for the compressibility of sensitive natural clays", *Geotechnique*, **35**(2), 159-180. https://doi.org/10.1680/geot.1985.35.2.159.

Leroueil, S., Tavenas, F. and Brucy, F .(1979), "Behavior of destructured natural clays", *Int. J. Rock Mech. Min. Sci. Geomech. Abstracts*, **105**(6), 759-778. https://doi.org/10.1016/0148-9062(79)90037-8.

Li, Q., Ng, C.W.W. and Guo-bin, L. (2012), "Low secondary compressibility and shear strength of shanghai clay", *J. Central South Univ*., **19**(8), 2323-2332. https://doi.org/10.1007/s11771-012-1278-9.

Luo, Q. and Chen, X. (2014), "Experimental research on creep characteristics of Nansha soft soil", *Scientific World J*., **2014**. https://doi.org/10.1155/2014/968738.

Mckinley, J.D. and Sivakumar , V. (2009), "Coefficient of consolidation by plotting velocity against displacement", *Geotechnique*, **59**(6), 553-557. https://doi.org/10.1680/geot.7.00130.

Mesri, G., Ajlouni, M.A., Feng ,T.W. and Lo, D.O.K. (2017), "Surcharging of soft ground to reduce secondary settlement", *Proceeding of the 3rd Int. Conf. on Soft Soil Engineering*, Hong Kong, December 2001.

Mesri, G., and Choi, Y.K. (1985),. "Settlement analysis of embankments on soft clays", *J. Geotech. Eng*., **111**(4), 441-464. https://doi.org/10.1061/(ASCE)0733-9410(1985)111:4(441).

Mesri, G., Rokhsar, A. and Bohor, B.F. (1975), "Composition and compressibility of typical samples of mexico city clay", *Geotechnique*, **25**(3), 527-554. https://doi.org/10.1680/geot.1975.25.3.527.

Miao, L. and Kavazanjian, E. (2007), "secondary compression features of Jiangsu soft marine clay", *Mar. Georesour. Geotec*., **25**(2), 129-144. https://doi.org/10.1080/10641190701380258.

Mitchell, J.K. (2005), Fundamentals of Soil Behavior, (3rd Ed.), New York, John Wiley & Sons, inc, Hoboken, New Jersey, Canada.

Nash, D.F.T., Sills , G.C. and Davison, L.R. (1992), "One-dimensional consolidation testing of soft clay from bothkennar", *Geotechnique*, **42**(2), 241-256. https://doi.org/10.1680/geot.1992.42.2.241.

Robinson, R.G. (2003), "A Study on the beginning of secondary compression of soils", *J. Test. Eval.*, **31**(5), 388-397. https://doi.org/10.1520/JTE12362J .

Sridharan, A. and Prakash, K.(1998), "Characteristic water contents of a fine-grained soil-water system", *Geotechnique*, **48**(3), 337-346. https://doi.org/10.1680/geot.1998.48.3.337.

Sridharan, A. and Rao, A. S. (1982), "Mechanisms controlling the secondary compression of clays", *Geotechnique*, **32**(3), 249-260. https://doi.org/10.1680/geot.1982.32.3.249.

Suneel, M., Park, L.K. and Im, J.C. (2008), "Compressibility characteristics of Korean marine clay", *Mar. Georesour. Geotec.*, **26**(2), 111-127. https://doi.org/10.1080/10641190802022478.

Terzaghi, K. (1943), Theoretical Soil Mechanics, JohnWiley & Sons, New York.

Walker, L.K. (1969), "Undrained creep in a sensitive clay", *Geotechnique*, **19**(4), 515-529. https://doi.org/10.1680/geot.1969.19.4.515.

Wu, Z.X., Jin, Y.F. and Yin, Z.Y. (2013), "Nonlinear creep behavior of normally consolidated soft clay", In *Constitutive Modeling of Geomaterials* (pp. 145-148). Springer, Berlin, Heidelberg, Germany. https://doi.org/10.1007/978-3-642-32814-5_16.

Wu, Z., Deng, Y., Cui, Y., Zhou, A., Feng, Q. and Xue, H. (2019), "Experimental study on creep behavior in oedometer tests of reconstituted soft clays", *Int. J. Geomech.*, **19**(3), 04018198. https://doi.org/10.1061/(ASCE)GM.1943-5622.0001357.

Yin, J.H. (2006), "Elastic visco-plastic models for the time-dependent stress–strain behaviour of geomaterials", *In Modern trends in geomechanics* (pp. 175-190), Springer, Berlin, Heidelberg, Germany.

Yin, J.H. (1999), "Non-Linear creep of soils in oedometer tests", *Geotechnique*, **49**(5), 699-707. https://doi.org/10.1680/geot.1999.49.5.699.

Yin, J.H. and Graham, J. (1989), "Viscous-elastic-plastic modelling of one-dimensional time-dependent behaviour of clays", *Can. Geotech. J.*, **26**(2), 199-209. https://doi.org/10.1139/t89-029.

Yin, J.H. (2015), "Fundamental issues of elastic viscoplastic modeling of the time-dependent stress-strain behavior of geomaterials", *Int. J. Geomech.*, **15**(5), A4015002. https://doi.org/10.1061/(ASCE)GM.1943-5622.0000485.

Yin, J.H, Zhu, J.G. and Graham, J. (2002), "A new elastic viscoplastic model for time-dependent behaviour of normally and overconsolidated clays: theory and verification", *Can. Geotech. J.*, **39**(1), 157-173. https://doi.org/10.1139/t01-074.

Yong, R.N. and Warkentin B.P. (1966), Introduction to Soil Behavior, Macmillan, New York.

Zhu, Q.Y., Yin , Z.Y., Hicher, P.Y. and Shen, S.L. (2016), "Nonlinearity of one-dimensional creep characteristics of soft clays", *Acta Geotechnica*, **11**(4), 887-900. https://doi.org/10.1007/s11440-015-0411-y.

IC