Research Article

A Practical Design Method to Determine the Replacement Time of Light Embankment

Fei Zhou,1 Fan Yu,2 Tangdai Xia,1 Bingqi Yu,1 and Yingtao Hu3,4

1Research Center of Coastal and Urban Geotechnical Engineering, Zhejiang University, Hangzhou 310058, China
2Zhejiang Provincial Institute of Communications Planning, Design & Research, Hangzhou, Zhejiang 310006, China
3MOE Key Laboratory of Soft Soils and Geoenvironmental Engineering, Zhejiang University, Hangzhou 310058, China
4School of Engineering, Zhejiang University City College, Hangzhou, Zhejiang 310015, China

Correspondence should be addressed to Yingtao Hu; yingtao_hu@zju.edu.cn

Received 29 April 2022; Accepted 11 July 2022; Published 2 August 2022

1. Introduction

Vehicle bumping at bridgehead is a complicated technical problem, the main reason of which is the creep of soil. For soft clay, the creep change is small but significant. In soil mechanics, the consolidation process of soil is divided into primary consolidation stage and secondary consolidation stage. At present, the secondary consolidation stage is considered to be the creep process of soil after the completion of the main consolidation.

However, there are two different hypotheses about whether there is creep in the main consolidation. Leonards [1], Ladd et al. [2], and Mesri [3, 4] adhere to hypothesis A through experimental studies. And they believe that the creep deformation only occurs the duration of primary consolidation. Hypothesis B holds that the creep of soil is because of its own viscosity, and the creep of soil exists not only after the end of the primary consolidation, but throughout the whole process of soil consolidation. It is supported by Bjerrum [5], Sukljie [6], Leroueil et al. [7], and Kabbaj et al. [8]. Yin et al. [9–12] calculated the consolidation pressure under different loading conditions through hypothesis A and pointed out the limitations of hypothesis A. Then, they established an elastic-viscoplastic model (EVP model) for one-dimensional stress-strain behavior of time-dependent soils. Hypothesis A and hypothesis B are both opposite and unified. Based on Yin-Graham one-dimensional EVP model, Hu et al. [13] corrected the creep start time in hypothesis A to $T_0$ according to theoretical analysis, obtained approximate equivalence between hypothesis A and hypothesis B, and unified the two hypotheses. According to hypothesis B, the creep coefficient is not a constant but a variable related to the overconsolidation ratio (OCR). Nowadays, many scholars [14–18] believe that the
creep coefficient decreases with the increase of OCR. Therefore, the overload preloading method can effectively reduce the postconstruction settlement.

Expanded polystyrene (EPS) is one of the useful materials in engineering. The density of EPS material is about 0.2–0.3 kN/m³, which is about 1/100–1/60 of the density of soil. It has the characteristics of ultra-lightweight, compressibility resistance, self-reliance, water resistance, flame retardant, and so on, which make it an excellent filling material. Light embankment began to be used in expressway engineering in 1970s [19–22]. When the early light embankment method was used in soft soil foundation, the soft soil foundation was not strengthened, so the settlement after construction was still large. With the continuous exploration of the engineering field, the filling technology of lightweight embankment is becoming more and more mature, which plays an increasingly excellent role in controlling the postconstruction settlement and improving the stability of embankment. The determination of filling thickness and filling time of light embankment plays a key role in controlling the settlement.

Chen et al. [23] studied the physical properties of the replaced lightweight material FCB and proposed a correction method for the material’s weight. Besides, a calculation program of the relationship between postconstruction settlement and overload ratio was compiled based on a large number of engineering monitoring data, in which an appropriate overload ratio is precisely calculated through the specific allowable settlement after construction to determine the replacement thickness. It is indicated that the determination of the replacement thickness demands the settlement rate during construction and the settlement reference data of the constructed project as relatively comprehensive factors are considered in this method. However, when the section with a large difference in geological conditions or lacking the measured data of the same type of constructed projects, it is hard to determine the replacement thickness in the construction design stage. In addition, as a result of comparative parameters selected in this method, it is difficult to apply and popularize in engineering.

Jiang et al. [24] proposed two methods for calculating replacement thickness in combination with engineering practice—the consolidation degree method and the calculated settlement method. Relationship between the settlement of embankment and the preload load through the degree of consolidation is established in the consolidation degree method. This method requires comparably detailed background information of the project. Additionally, it demands to monitor and estimate the degree of consolidation in layers during the construction period. As for calculated settlement method, relationship between soil stress and residual settlement is utilized to calculate the additional stress and void ratio of each layer of soil according to the standard, thereby calculating the residual settlement. The replacement thickness is considered to meet the engineering requirements when the residual settlement is lower than the allowable value. This method can be used when the degree of consolidation cannot be accurately estimated due to lack of monitoring data.

At present, the research on filling design of lightweight embankment mainly focuses on the design of filling thickness. However, it is important to determine the appropriate time of light embankment but there are only few studies in this area. Based on the EVP model, this paper will use the concept of equivalent time, combined with the actual engineering simplification, deducing the method of determining the time of filling replacement in order to be more convenient and to have a quick calculation in the engineering design.

2. Creep Coefficient of Overconsolidated Soil

2.1. Definition of Creep Coefficient. Terzaghi first proposed the one-dimensional consolidation theory of saturated soil. According to the effective stress principle, the excess pore pressure gradually dissipates with the discharge of pore water, and the additional stress is finally transformed into effective stress. Under this assumption, the consolidation settlement of soil can be calculated. However, it is found that there is a large gap between the calculated results of consolidation settlement and the actual settlement in many projects.

At the beginning of the last century, some scholars [25–29] found that the deformation of soil continued to grow with the development of time after the completion of consolidation in tests, which was defined as secondary consolidation. Then, the consolidation of soil was divided into primary consolidation stage and secondary consolidation stage, and the secondary consolidation takes place after the completion of primary consolidation.

After long-term settlement observation, Buisman [30] concluded that the settlement of soft clay showed a linear relationship with the logarithm of time, and proposed a formula to calculate the coefficient. As is shown in Figure 1 [31], it is believed that the \( e - \lg t \) curve is close to a straight line after primary consolidation.

\[
C_a = \frac{e_0 - e_1}{\lg t_1 - \lg t_0},
\]

where \( t_0 \) is the time at the end of primary consolidation, \( t_1 \) is any time after the end of primary consolidation, and \( C_a \) is the secondary consolidation coefficient.
However, Equation (1) is not suitable for the overconsolidated soil, and according to the definition of secondary consolidation, the overconsolidated soil has finished primary consolidation before test. Crawford [32] drew a set of $\varepsilon - \log p$ curves based on one-dimensional creep test results of normally consolidated soil under different load durations, as shown in Figure 2. Line a corresponds to the completion time of primary consolidation of soil, line b corresponds to 1 day after the completion of the primary consolidation, and line c corresponds to 7 days after the completion of the primary consolidation. It can be seen from the figure that lines a, b, and c are roughly parallel curve clusters. Based on Crawford test curve, Bjerrum simplified the curve by parallel lines obtaining a series of lines. Based on Bjerrum’s creep diagram, Yin-Graham proposed 1-D EVP model, and the total strains can be written as follows:

$$
\varepsilon_z = \varepsilon_{zo} + \frac{\lambda}{V} \left( \frac{\sigma'_z}{\sigma_{zo}} \right) + \frac{\psi}{V} \ln \left( \frac{t + t_o}{t_o} \right), \tag{2}
$$

where $\varepsilon_{zo}$ is the initial strain of soil, $\varepsilon_z$ is the total strain, $\sigma'_z$ is the initial stress which is under the initial condition, and $V$ is the specific volume. Here, we call $C_{ac}$ “creep coefficient” instead of the “secondary consolidation coefficient”, because Equation (2) considers creep occurs during and after “primary consolidation.” $t_e$ is called the equivalent time [32]. $t_o$ is a parameter which is usually taken as unit time or the boundary time of the primary and secondary consolidation [33].

According to Equation (2), the relationship between the effective stress and the void ratio $\varepsilon$ can be expressed by the equivalent time as follows:

$$
\varepsilon = \varepsilon_0 - C_c \log \left( \frac{\sigma'_z}{\sigma_{zo}} \right) - C_{ac} \log \left( \frac{t + t_o}{t_o} \right),
$$

$$
C_{ac0} \log \left( \frac{t + t_o}{t_o} \right) = (C_c - C_e) \log \left( \frac{\sigma'_z}{\sigma_{zo}} \right),
$$

where $C_{ac0}$ is the normal consolidation soil creep coefficient and $C_e$ is the expansion coefficient, from which

$$
t_e = t_o \left( \frac{\sigma'_z}{\sigma_{zo}} \right) \left( \frac{(C_c - C_e)}{C_{ac0}} \right) - t_o.
$$

Let

$$
\text{OCR} = \frac{\sigma'_z}{\sigma_{zo}},
$$

$$
\delta = \frac{C_c - C_e}{C_{ac0}},
$$

where $t_e$ can be described as follows:

$$
t_e = t_o (\text{OCR})^\delta - t_o. \tag{6}
$$

According to Equation (6), $t_e$ is not a constant, but it is a variable related to OCR. So, the creep coefficient of overconsolidated soils can be written as follows:

$$
C_{ac} = \frac{\varepsilon_1 - \varepsilon_2}{\log \left( \frac{\sigma_1 + t_o}{\sigma_1 + t_o} \right)} = \frac{\varepsilon_1 - \varepsilon_2}{\log \left( \frac{(t_1 + t_o)(t_1 + t_o)}{(t_2 + t_o)(t_2 + t_o)} \right)} \tag{7}
$$

$$
= \frac{\varepsilon_1 - \varepsilon_2}{\log \left( 1 + \frac{\Delta t(t_1 + t_o)}{t_1 + t_o} \right)},
$$

where $t_1$ and $t_2$ are the equivalent time, $\varepsilon_1$ and $\varepsilon_2$ are the void ratios at equivalent time $t_1$ and $t_2$, respectively, and $\Delta t$ is the loading duration from $t_1$ to $t_2$.

We can learn from Equation (6) and Equation (7) that the creep coefficient is not a constant. It varies with the change of OCR and real time.

2.2. One-Dimensional Consolidation Test

2.2.1. Experiment Scheme and Description. This paper fully considers the actual engineering situation and selects the overconsolidation ratio of 1.0, 1.1, 1.2, 1.4, and 1.8 to study the relationship between creep coefficient and overconsolidation ratio. In order to study the relationship between creep coefficient and loading time, the last stage of loading lasted for 7d. In this test, the soil depth is 8 m, and the effective stress of the soil sample in the natural state is about 60 kPa. In order to ensure the acquisition of overconsolidated soil and considering the actual engineering preload, the maximum value of overloading is set as 150 kPa. The spring back of soil in internal test is not instantaneous and needs a certain stabilization time. In the preliminary test, it
is found that the deformation of soil is very small at the early stage after unloading and reloading. This is because the soil rebound after unloading is not instantaneous, the soil will produce negative pore pressure during unloading, and its effective stress cannot be reduced to the value of reloading instantaneously. The deformation at the initial stage of reloading and the spring back deformation cancel each other, so almost no deformation occurs at the early stage.

In this paper, through preliminary tests, it is found that the spring back stability time of soil samples in Huzhou area is $1 \sim 2 \text{d}$ under the condition of reloading. Therefore, in the design test of this paper, three levels of load were applied after unloading and then loading. In order to control variables, the loading ratio was ensured to be 1 during the loading process, and the duration of each level of load was 1 d. Therefore, the equivalent time $t_e$ of soil can be approximated to 1 d. The specific scheme for one-dimensional compression tests is shown in Table 1.

### Table 1: One-dimensional consolidation experiment scheme.

<table>
<thead>
<tr>
<th>Number</th>
<th>Depth</th>
<th>OCR</th>
<th>Load path and duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-1</td>
<td>8 m</td>
<td>1.0</td>
<td>37.5 kPa (1 d) $\rightarrow$ 75 kPa (1 d) $\rightarrow$ 150 kPa (7 d)</td>
</tr>
<tr>
<td>A-2</td>
<td>8 m</td>
<td>1.1</td>
<td>37.5 kPa (1 d) $\rightarrow$ 75 kPa (1 d) $\rightarrow$ 150 kPa (1 d) $\rightarrow$ 60.6 kPa (1 d) $\rightarrow$ 90.9 kPa (1 d) $\rightarrow$ 136.4 kPa (7 d)</td>
</tr>
<tr>
<td>A-3</td>
<td>8 m</td>
<td>1.2</td>
<td>37.5 kPa (1 d) $\rightarrow$ 75 kPa (1 d) $\rightarrow$ 150 kPa (1 d) $\rightarrow$ 55.5 kPa (1 d) $\rightarrow$ 83.33 kPa (1 d) $\rightarrow$ 125 kPa (7 d)</td>
</tr>
<tr>
<td>A-4</td>
<td>8 m</td>
<td>1.4</td>
<td>37.5 kPa (1 d) $\rightarrow$ 75 kPa (1 d) $\rightarrow$ 150 kPa (1 d) $\rightarrow$ 47.5 kPa (1 d) $\rightarrow$ 71.32009 kPa (1 d) $\rightarrow$ 107 kPa (7 d)</td>
</tr>
<tr>
<td>A-5</td>
<td>8 m</td>
<td>1.8</td>
<td>37.5 kPa (1 d) $\rightarrow$ 75 kPa (1 d) $\rightarrow$ 150 kPa (1 d) $\rightarrow$ 37.0 kPa (1 d) $\rightarrow$ 55.5 kPa (1 d) $\rightarrow$ 83.3 kPa (7 d)</td>
</tr>
</tbody>
</table>

2.2.2. Test Results. Figure 3 shows the $e - \lg t$ curve of sample A-1. As described before, the sample A-1 is normally a consolidation soil. According to Figure 4, the boundary between the primary consolidation and secondary consolidation is about 100 min, which means $t_e = 100 \text{ min}$. According to Equation (1),

$$C_a = \frac{e_0 - e_1}{\lg t_1 - \lg t_0} = \frac{1.469 - 1.437}{\lg 1440 - \lg 100} = 0.0271.$$  \hspace{1cm} (8)

Figure 5 shows the $e - \lg t$ curves of sample A2-A5 under the final load. According to Equation (7), calculate the creep coefficient of these samples.

Taking sample A-2 as an example, $t_e = 1440 \text{ min}$, $t_o = 100 \text{ min}$, $\Delta t = 9980 \text{ min}$, $e_i = 1.245$, and $e_2 = 1.228$. So, the creep coefficient is 0.0197, which is a 7-day creep coefficient.

$$C_{ac} = \frac{e_1 - e_2}{\lg (1 + (\Delta t)/(t_1 + t_o))} = \frac{1.245 - 1.228}{\lg (1 + (9980/(1440 + 100)))} \approx 0.0197.$$  \hspace{1cm} (9)

According to the above calculation procedure, creep coefficients of sample A-2 to A-5 are calculated as Table 2.

As is shown in Figure 6, the relationship between a 7-day creep coefficient and OCR can be fitted by Equation (10), whose correlation coefficient $R^2$ is 0.991.

$$C_{ac} = -0.0031 + 0.2446e^{-2.1184\text{OCR}}.$$  \hspace{1cm} (10)

For silt soil, when OCR is 1.1, the creep coefficient decreases by about 30%; when OCR is 1.2, the creep
When OCR is 1.0, the creep coefficient decreases by about 40%; when OCR is 1.4, the creep coefficient decreases by about 60%; when OCR is larger than 2, the creep coefficient decreases by more than 90%.

In order to study the time effect of creep coefficient, the creep coefficients under 2d, 3d, 4d, 5d, 6d, and 7d of each sample with different OCR are calculated as shown in Figure 7.

As is shown in Figure 7, the creep coefficient has time effect obviously. The creep coefficient decreases gradually over time before the third day and increases after the third day. On the sixth day, the creep coefficient tends to be stable. According to the test results, it is suggested that the 7-day creep coefficient should be used to calculate the postconstruction settlement in practical engineering.

3. Determination of the Preloading Time of Layered Soils Based on EVP Model

Hu et al. [13] proposed the concept of absolute equivalent time and deduced the unloading time of overload preloading method using EVP model. On this basis, we derive the calculation method of preloading time of layered soil.
3.1. Basic Assumptions. Considering the application of the EVP model in the actual engineering, when the fluid-solid coupling is not considered and the soil is in a normal consolidation state (as shown at point A in Figure 8), the initial state of the soil is on the normal consolidation line. At this point, the loading is carried out step by step. When the first-level load is applied, the stress-strain state of soil will move down along the normal consolidation line and the main consolidation is completed at point B. Then, the soil begins to deform along the creep curve and reaches point C. Due to the short preload time of this level load, after the application of the second-level load, the soil will move down along the rebound and recompression curve and reach point D at the normal consolidation line. Then, the cycle repeats. When the last level of load is applied, the soil will move along the normal consolidation line to point E. At this point, the main consolidation is completed and then the soil starts to move along the creep curve. When unloading, the stress-strain state of the soil is at point F. The equivalent time from point F to point E is the equivalent time of the soil when unloading.

After unloading and pavement structural layer laying, the equivalent time of unloading process and the time of pavement structural layer laying process cannot be simply added up as the equivalent time at the time of completion, which should be reasonably assumed according to the actual situation of the project. During the process of unloading and pavement structural layer laying, the soil will rebound and recompress, and will move along the rebound and recompression curve to point a. After the pavement is laid, the equivalent time is the equivalent time of the soil when unloading.

After unloading and pavement structural layer laying, the equivalent time of unloading process and the time of pavement structural layer laying process cannot be simply added up as the equivalent time at the time of completion, which should be reasonably assumed according to the actual situation of the project. During the process of unloading and pavement structural layer laying, the soil will rebound and recompress, and will move along the rebound and recompression curve to point a. After the pavement is laid, the equivalent time is the equivalent time of the soil when unloading.

3.2. Calculate the Preloading Duration. In practical engineering design, the layered summation method is often used to calculate the settlement. In order to combine with the practical engineering, the effective stress of each layer is very small and the increase of equivalent time generally does not exceed 5 d. Therefore, it can be considered that the equivalent time of the soil does not change substantially at this stage. The equivalent time calculated in this way will be slightly smaller than the actual situation, the creep coefficient will be a little larger, and the calculation results of post-construction settlement are also more conservative. So, the equivalent time of the soil when unloading can be used to calculate the postconstruction settlement according to Equation (11).

\[
s = \sum_{i=1}^{n} \frac{H_i}{1 + e_{i0}} C_{\text{act}} \log \frac{t_0 + t_{ei} + T}{t_0 + t_{ei}},
\]

where \(H_i\) is the thickness of the \(i\)-th layer of soil; \(C_{\text{act}}\) is the creep coefficient of the \(i\)-th layer of soil, which can be calculated and determined according to the laboratory test method in Section 3 of this paper; \(t_{ei}\) is the equivalent time of the \(i\)-th layer of soil when unloading; \(T\) is the design service time of pavement.

In actual engineering, the total settlement is more concerned, so the completion time of the primary consolidation can be determined according to the total settlement of all soil layers. For existing engineering projects, the completion time of primary consolidation can be determined by the construction monitoring data. And for new engineering projects, it can be calculated according to the consolidation theory in design stage. The average equivalent time of all soil layers can be taken as the equivalent time of each soil layer, and the average equivalent time is the difference between the unloading time and the completion time of the primary consolidation determined according to the total settlement.
calculated according to the practical engineering. The average soil stress in a layer is expressed by the stress at the midpoint of the soil layer. As for the top soil layer, which is the first soil layer, the thickness is set as $H_1$, and the average effective stress is expressed by the following equation:

$$
s_1 = \frac{1}{2} \gamma_1 H_1. \quad (12)$$

As is shown in Figure 4, as for the $i$-th layer ($i \geq 2$), the average effective stress is expressed by the following equation:

$$
s_i = \sum_{j=1}^{i} \gamma_{i-1} H_{i-1} + \frac{1}{2} \gamma_i H_i. \quad (13)$$

It is assumed that when the preloading is completed, the equivalent time of the soil layer is $t_{emi}$, and the void ratio $e_{mi}$ can be obtained according to the following equations:

$$
e_{mi} = e_{0i} - C_{si} \log \left( \frac{s_i + \alpha p_0}{s_i} \right) - C_{aui} \log \left( \frac{t_{emi} + t_0}{t_{emi} + t_0} \right), \quad (14)$$

$$
t_{exi} = t_0 \left( \frac{s_i}{s_i + \alpha p_0} \right)^{\delta_i} - t_0, \quad (15)$$

where $\alpha$ is the additional stress coefficient, $t_{exi}$ is the initial equivalent time of preloading of the $i$-th soil layer, and $p_0$ is the preloading load.

During unloading time, the soil will rebound. And according to the one-dimensional EVP model, the soil moves along the rebound curve. When unloading is completed, the corresponding void ratio can be expressed by
the following equation:

\[ e_{ai} = e_{mi} + C_{ai} \log \left( \frac{\sigma_i + \alpha p_m}{\sigma_i + \alpha p_f} \right) - C_{aci} \log \left( \frac{t_{emi} + t_0}{t_{emi} + t_0} \right), \]

\[ = e_{0i} - C_{ai} \log \left( \frac{\sigma_i + \alpha p_m}{\sigma_i + \alpha p_f} \right) - C_{aci} \log \left( \frac{t_{emi} + t_0}{t_{emi} + t_0} \right), \]

where \( e_{ai} \) is the void ratio of the \( i \)-th soil layer after unloading and \( p_m \) is the filling load after unloading.

The lightweight material is replaced, and the pavement structure layer is paved after unloading. In this process, the soil is under reloading stage. And the soil is still under the overconsolidation state because the reloading load is smaller than the preloading load. Then, according to the one-dimensional EVP model, the soil will undergo elastic deformation along the rebound curve, and the corresponding void ratio can be expressed as follows:

\[ e_{ci} = e_{ai} + C_{ai} \log \left( \frac{\sigma_i + \alpha p_m}{\sigma_i + \alpha p_f} \right) - C_{aci} \log \left( \frac{t_{emi} + t_0}{t_{emi} + t_0} \right), \]

\[ = e_{0i} - C_{ai} \log \left( \frac{\sigma_i + \alpha p_m}{\sigma_i + \alpha p_f} \right) - C_{aci} \log \left( \frac{t_{emi} + t_0}{t_{emi} + t_0} \right), \]

where \( e_{ci} \) is the void ratio of the \( i \)-th soil layer after pavement construction.

The service period of expressway \( T \) is generally 15 years, the vehicle load on the expressway is assumed to be \( p_{car} \), and it is equivalent to the reloading process. Then, the void ratio of \( i \)-th soil layer after 15 years is expressed by the following equation:

\[ e_{fi} = e_{ci} - C_{ai} \log \left( \frac{\sigma_i + \alpha p_f + \alpha p_{car}}{\sigma_i + \alpha p_f} \right) - C_{aci} \log \left( \frac{t_{emi} + T}{t_{emi} + t_0} \right), \]

\[ = e_{0i} - C_{ai} \log \left( \frac{\sigma_i + \alpha p_f + \alpha p_{car}}{\sigma_i + \alpha p_f} \right) - C_{aci} \log \left( \frac{t_{emi} + t_0 + T}{t_{emi} + t_0} \right), \]

where \( e_{fi} \) is the void ratio of the \( i \)-th soil layer after 15 years and \( p_f \) is the final load after the pavement being paved.

Then, the postconstruction settlement of the \( i \)-th soil layer within 15 years can be calculated as follows:

\[ S_i = \frac{e_{ci} - e_{fi}}{1 + e_{0i}}, \]

\[ = \frac{H_i}{1 + e_{0i}} C_{ai} \log \left( 1 + \frac{T}{t_{emi} + t_0} \right) + \frac{H_i}{1 + e_{0i}} C_{ai} \log \left( \frac{\sigma_i + \alpha p_f + \alpha p_{car}}{\sigma_i} \right). \]

In practical engineering such as expressway, without considering the cumulative effect of cyclic load, the settlement caused by vehicle load is recoverable elastic deformation, which is not considered in general sections. As this study mainly considers the control of bridgehead section, the effect can be ignored. Then, the settlement of the soil layer within 15 years can be calculated as follows:

\[ S = \sum_{i=1}^{n} \frac{H_i}{1 + e_i} C_{ai} \log \left( 1 + \frac{T}{t_{emi} + t_0} \right). \]

The total postconstruction settlement of the section can be obtained by summing the settlement calculation of the soil layer. The total settlement should be smaller than the allowable settlement of the section \( |S| \), which is expressed by the following equation:

\[ S = \sum_{i=1}^{n} \frac{H_i}{1 + e_i} C_{ai} \log \left( 1 + \frac{T}{t_{emi} + t_0} \right) \leq |S|. \]

The equivalent time is also different due to the different OCR of each soil layer. \( t_{emi} \) is the equivalent time of each soil layer at the end of the preloading period, and it is also the actual time of creep after the primary consolidation. Therefore, the maximum \( t_{emi} \) is obtained as the equivalent time of all soil layers after the primary consolidation, which is expressed as follows:

\[ t_{emi} = \max (t_{emi}) = \max (t_{emi1}, t_{emi2}, \ldots, t_{emln}). \]
Let \( \mu_i = \frac{C_{\alpha i}H_i}{1 + e_i} \). \hspace{1cm} (23)

According to Equation (20), \( t_{em} \) is calculated as follows:

\[
t_{em} \geq \frac{T}{10\left\{\left[\frac{1}{H_i}\sum_{i=1}^{n}\mu_i\right] - 1\right\}} - t_0. \hspace{1cm} (24)
\]

In this study, the calculated equivalent time is regarded as the load duration after the end of the primary consolidation time. Therefore, the unloading time is determined by calculating the required preloading time after the primary consolidation. The replacement starts after the rebound is stable. So, the replacement time is equal to the duration of stabilization add \( t_{em} \).

4. Project Study

4.1. The Background of Engineering Project. Huzhou Avenue is located in the plain area with flat terrain and dense ponds. The surface is soft silty clay, the upper distribution of silt, gray, and fluid plastic, local phase into soft plastic clay. The lower part consists of clayey pebbles and fully weathered argillaceous siltstone.

As is shown in Figure 9, the bridge approach is about 140 m long. Section I (S1 and S2) adopts composite foundation method (reinforced cushion+cement mixing pile). Section II (from S3 to S5) adopts this method (drainage consolidation+light embankment). The soft clay beneath is about 13 m deep. The soil layer is depicted in Figure 10. It is obtained by one-dimensional consolidation experiment that the compression index \( C_c \) is 0.43 and the swelling index \( C_s \) is 0.05.

4.2. Calculate the Preloading Duration. As is shown in Figure 10, hard crust exhibits low compressibility, so the postconstruction settlement of this layer can be ignored. As for S5, the designed embankment height \( H \) is 4.4 m. The density of filling soil \( \gamma \) is 20 kN/m\(^3\). So, the preloading load is calculated as follows:

\[
p_f = \gamma H = 20 \times 4.4 = 88 \text{kPa}. \hspace{1cm} (25)
\]

The thickness of pavement structure is 0.9 m. And its density \( \gamma_s = 23 \text{kN/m}^3 \). When the thickness of EPS \( \Delta h \) is 0.5 m, the final load \( p_f \) and OCR are calculated as follows:

\[
p_f = \gamma h + \gamma_f \Delta h + \gamma_{sh}h_i = 20 \times 3 + 0.2 \times 0.5 + 23 \times 0.9 = 80.47 \text{kPa}. \hspace{1cm} (26)
\]

For the top soil layer, the middle point depth \( z \) is 1 m. The additional stress coefficient is \( \alpha = 1.0 \).

\[
\text{OCR} = \frac{\gamma z + \alpha p_f}{\gamma z + \alpha p_f} = \frac{16 \times 1 + 1 \times 88}{16 \times 1 + 1 \times 80.47} = 1.077,
\]

\[
C_{ac} = -0.0031 + 0.2446e^{-2.1184\text{OCR}} = 0.0219,
\]

\[
\mu_1 = \frac{C_{\alpha 1}H_1}{1 + e_1} = \frac{0.0219 \times 2}{1 + 1.469} = 0.0170,
\]

\[
\delta = \frac{C_c - C_s}{C_{\alpha \text{avg}}} = \frac{0.431 - 0.052}{0.027} = 14.03.
\]

The postconstruction settlement of other soil layers is calculated according to this procedure, and the results are shown in Table 3.

The service period \( T \) is usually 15 years, and the allowable settlement value of this section after 15 years of construction is 239 mm. The postconstruction settlement includes the postconstruction settlement of soft clay, filling soil, and EPS. After cyclic compression, the postconstruction

---

**Figure 9:** Schematic diagram of bridge approach.

**Figure 10:** Calculation diagram of soil layer.
settlement of the replacement soil is small. The lightweight EPS has strong compressibility. So, the postconstruction settlement of these two parts can be ignored. According to Equation (24), the equivalent time at the end of preloading time are calculated as follows:

\[ t_{em} \geq \frac{T}{10\left\{ S_t \left( \sum_{i=1}^{n} \mu_i \right) \right\}} - t_0 = \frac{15 \times 365}{10^{0.239/0.1135}} - 1 \approx 43 \text{d.} \]

For S5, when the degree of consolidation reaches 100% during the construction, it shall be preloaded at least 43 d for unloading stability and then replaced with light soil.

The allowable settlement values of S3 and S4 are 135 mm and 193 mm, respectively. According to this method, the equivalent time at the end of preloading time are calculated as follows.

For S3, when the degree of consolidation reaches 100% during the construction, it shall be preloaded at least 144 d for unloading stability and then replaced with light soil.

\[ t_{em} \geq \frac{T}{10\left\{ S_t \left( \sum_{i=1}^{n} \mu_i \right) \right\}} - t_0 = \frac{15 \times 365}{10^{0.135/0.0849}} - 1 \approx 144 \text{d.} \]

For S4, when the degree of consolidation reaches 100% during the construction, it shall be preloaded at least 70 d for unloading stability and then replaced with light soil.

\[ t_{em} \geq \frac{T}{10\left\{ S_t \left( \sum_{i=1}^{n} \mu_i \right) \right\}} - t_0 = \frac{15 \times 365}{10^{0.189/0.0993}} - 1 \approx 70 \text{d.} \]

Even for the section with relatively large overloading, due to its small allowable value of postconstruction settlement, a longer preloading time is still needed to control postconstruction settlement after the completion of primary consolidation.

5. Conclusions

According to practical engineering, a practical design method to determine the replacement time of light embankment is proposed. The main conclusions are drawn as follows:

(a) The creep coefficient of overconsolidated soil decreases with the increase of overconsolidation ratio, and the relationship between creep coefficient and overconsolidation ratio can be fitted by exponential function

(b) The creep coefficient of overconsolidated soil has obvious time effect, which decreases first and then increases with time going. In practical engineering application, the creep coefficient which is stable after 7 days should be selected for calculation

(c) The method is based on two assumptions: (1) For normally consolidated soil, the elastoplastic deformation at any point of soil under external load can be equivalent to the sum of instantaneous elastic deformation and creep based on equivalent time. (2) The increase of equivalent time is ignored during unloading, filling, and paving process. The method can be used for design according to different filling thickness

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this paper.

Acknowledgments

This study was supported by the Science and Technology Plan of Zhejiang Provincial Department of Transportation (2015J10).

References


