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CONTROLLING THE SETTLEMENT OF THE LOW EMBANKMENT ON SOFT GROUND BY SURCHARGE PRELOADING METHOD

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ABSTRACT: Preloading with surcharge is the most common technique of ground improvement used in highway engineering. At the end of the preloading period, the surcharge load is removed, and most or all of the primary settlement and some of the secondary settlement would have occurred. The soil beneath the embankment becomes over-consolidated, resulting in the elimination or a huge reduction in post-construction settlement. When pre-compression is applied to a relatively free-draining material, consolidation times are short, and therefore primary settlements can be effectively controlled by using a preload equal to the final load over a limited period of time. The subsequent movements associated with unloading/reloading will be small and will occur rapidly. In these cases, the main source of long-term displacements is likely to be secondary compression settlements. In this paper, a laboratory investigation of one-dimensional consolidation of clayey soils was carried out and a formula has been proposed to compute the post-construction settlement of low embankment. The results show that the settlement increases with the increasing amplitude of the cyclic load, and the effectiveness of surcharge preloading depends on the difference between the surcharge and the amplitude of the cyclic load.

Keywords: Soft soil, Low embankment, Consolidation, Surcharge preloading, Secondary settlement

1. INTRODUCTION

Preloading involves the placement of a surface load prior to construction in order to precompress the foundation soil. On some occasions, a preload greater than the structure load is used. This situation is termed surcharging. The excess of the preload over the actual load of structures is termed a surcharge. It is important to determine at what time the surcharge is removed after a short period of time. Preconsolidation does not have an adequate effect on the improvement of soft clay. Watanabe and Imran Khan studied the stabilization of soft subsoils and showed that the preloading method has an effect on reducing the settlement of soft clay deposits subjected to traffic-induced cyclic loading. In this case, the cyclic loads were the railway train loads, and the strain of the deposit without any improvement is large than that which was preloaded [1, 2]. Alonso presented the pre-compression design for secondary settlement reduction. In the study, pre-compression has been selected as the method for achieving the required ground improvement. An instrumented preload test has been carried out to obtain reliable information on pre-compression performance. The results of the study show that the magnitudes of ground deformation are largely dependent on the over-consolidation state of the soil. Laboratory and field data also indicated clearly that overconsolidated soil significantly reduces the secondary compression rate. Therefore applying a preload surcharge larger than the final structure load is quite effective in controlling the magnitude of subsequent secondary settlements [3]. Zhang conducted a study on deformation behavior and post-construction settlement of Shenzhen soft clay and has shown that surcharge can reduce the secondary settlement of a soft foundation by decreasing the coefficient of secondary compression and combined with delaying the occurrence of secondary compression [4]. Zhou and Shoji analyzed the change in settlement of the soft foundation of an expressway under surcharge unloading and reloading. It was observed that the settlement occurred mostly during the period of filling and surcharge preloading, and the rate of settlement decreased during reloading after surcharge unloading, but it was not accomplished quickly after the reloading period [5, 6]. Yang and LI presented a settlement rate method for determining surcharge removal time for an embankment on soft ground. Based on this method, an equation describing the relationship between the settlement rate during surcharge preloading and the post-construction settlement under the conditions of equal loads was derived. From the equation, the settlement rate under the conditions of overloading depended not only on the allowed construction settlement but also on the consolidated state of the soil, the loading rate, and the starting and ending time for all load increments [7]. Li investigated the mechanism of surcharge preloading on the soft ground highway foundations. Longer surcharge preloading duration and larger over-pressure ratios (OPR) corresponded with a lower magnitude of secondary compression deformation of soft clay after unloading. The study also indicated that low OPR could be sufficient for controlling post-construction settlement when the surcharge preloading duration is sufficiently long [8].

This study aims at not only analyzing the long-term effect of dynamic loading on the post-construction settlement of over-consolidated clay but also clarifying the role of the surcharge preloading method in controlling this settlement increment. Detailed means of settlement prediction of surcharge preloaded low embankment on soft ground are also described. Based on the consolidation theory combined with the test results, a formula has been proposed to compute the post-construction settlement of low embankment on soft ground.

2. RESEARCH SIGNIFICANCE

Laboratory investigation of the one-dimensional creep behavior of clayey soils subjected to cyclic loading is discussed in this study. Theoretical background evaluation and examination of results are presented based on oedometer tests performed on soft clay deposits located beneath the embankment of the Shanfen highway in Shantou City in the Guangdong province of China. Detailed means of settlement prediction of surcharge preloaded low embankment on soft ground are also described.

Based on the consolidation theory combined with the test results, a formula has been proposed to compute the post-construction settlement of low embankments under cyclic loading; further, technical measures can be developed to control the settlement of soft ground under the low embankments.

3. EMBANKMENT ON SOFT GROUND

Embankments are required in the construction of roads, motorways, railway networks, hydraulic structures (e.g. dams and retention dikes), harbor installations (e.g. seawalls and breakwaters), and airport runways [9, 10]. Low embankments are the most widely used embankment configuration nowadays due to numerous obvious advantages, such as reduction of land occupation, suitability for the natural landscape, road traffic safety, give facilities for widening and development in the future [11].

Figure 1 shows a typical embankment configuration with soil layers underneath. The embankment increases the stress in the soil layers and the saturated soft clay soils, being a highly compressible, will consolidate (settle). The estimation of the total and rate of settlement of an embankment with good serviceability is the main design concern of embankments on soft soils. Several numerical methods have been developed to predict embankment behavior on soft soils based on the drainage conditions of the soft soils. All the design methods require laboratory testing and/or field testing to determine the parameters to be used. Each parameter can be determined using different tests, resulting in different values for the consolidation parameters [12].

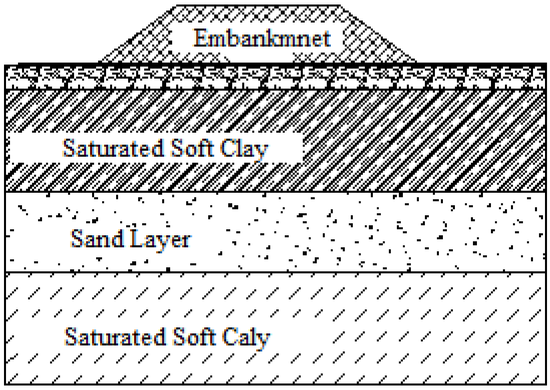


Fig.1 Typical configuration of soil layers under an embankment [12]

The stress on the foundation soil increases when the embankment compresses the foundation soil layers through elastic and consolidation settlement. Due to traffic loads and the self-weight of the embankment, the long-term settlement also occurs within the embankment itself. Thus, settlement takes place both in the foundation used to consider the effect of the overlying fills and traffic [13]. In addition, settlement may occur during the construction of the embankment and also for some time afterward. This can be considered to arise from a) the deformation of soil particles, b) the rearrangement of soil particles, and c) the dissipation of air and water from the inter particles' void spaces. It is usual to classify the resulting settlement as comprising of elastic and consolidation settlement.

When soil is loaded, such as with an embankment, the increase in load is carried by an increase in pore water pressure. This creates a pressure differential or excess in pore pressure between the soil affected by the load and the residual groundwater pressure. The excess pressure dissipates over time as water flows out from under the loaded area. With time excess pore water pressure generated under the application of the load dissipates. Soil particles undergo a certain stable state of compaction after dissipating excess pore water pressure completely [14].

As the primary consolidation continues with time, the rate of settlement can be found from Tezaghi's one-dimensional consolidation theory for saturated soils if the consolidation characteristics of soils are known [15]. The primary consolidation parameters are characterized by the coefficient of consolidation (C_v) and coefficient of volume compressibility (m_v), while the coefficient of secondary compression (C_α) is one of the most important soil parameters required to calculate secondary compression.

There are two main methods of pre-compression: a) Preloading - the placement of load for a period of time to achieve the desired compression and removing it before actual construction and b) Surcharging - the application of loading that is greater than the structure load. The application of pre-compression minimizes immediate and primary settlement and reduces the rate of secondary settlement. After the removal of the surcharge, it is necessary to control any rebound or swelling [16]. To control the rebound, Hu Yang suggests the final load should not be less than one-third of the surcharge load. The time for removal of the surcharge load is determined in such a way so that the degree of consolidation (U) is at least $\sigma_{sc}/(\sigma_{sc}+\sigma_{st})$ or typically 75%, where σ_{sc} and σ_{st} represent the surcharge and structure load, respectively [17].

4. OEDOMETER CONSOLIDATION TESTS

3.1 Test Program

3.1.1 Sample and specimen:

Soil samples were taken from a soft clay deposit located beneath the embankment of the Shanfen highway in Shantou City, Guangdong Province of China. The test was conducted on a group of specimens; that had 5 cm and 120 cm^2 height and cross-sectional area, respectively, as shown in Figure 2.

Fig.2 Photo of the soil specimen

The soil properties were as follows: density ρ = 1.52~1.56 g/cm^3, natural water content w_n = 61.4~66.6%, plasticity index I_P = 22~25%, liquidity index I_L = 1.1~1.5%, initial void ratio e = 1.65~1.83, degree of Saturation S_r = 96.9~98%, compression factor $\alpha_{0.1\text{-}0.2}$ = 1.61~1.84MPa^{-1}, compression modulus E_s = 1.31~1.42MPa, coefficient of consolidation C_{v100} = 0.448x10^{-3}~0.704x$10^{-3}$$cm^2/s$ and C_{v200} = 0.645x10^{-3}~0.667x$10^{-3}$$cm^2/s$. All data were determined from tests according to JTG E40-2007 Test Methods of Soils for Highway Engineering. The grain size distribution is shown in Figure 3, according to Thang [18].

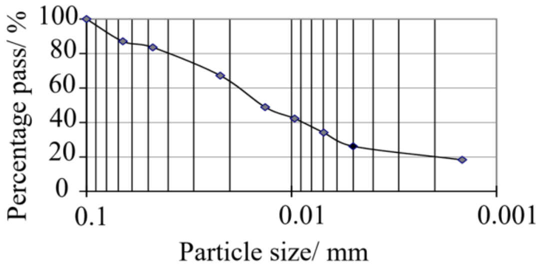


Fig.3 The particle size distribution

3.1.2 Test procedure:

The loading apparatus in this study is described in detail in [18] and consists of three main components: the stress sensors, controller and loading device; the schematic of the apparatus in the test is shown in Figure 4.

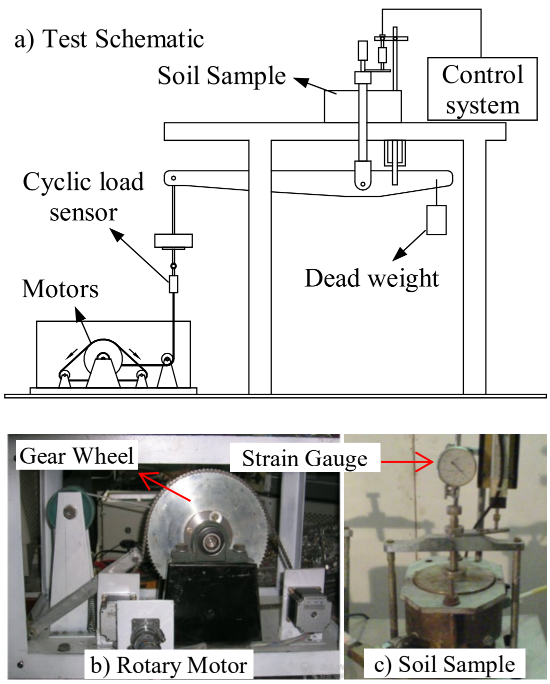


Fig.4 Pictures of Apparatus Used in the Test [18]

The main controller consists of the processor board, two stepper electric motor drives, a main control panel, a ring-shaped transformer, a rectifier and power supply components. The loading device consists of two stepper electric motors, pinions, a chain, a wire buckle, wire wheels and a pressure sensor. This device overcomes the defects and shortcomings of the dynamic triaxial apparatus.

The main features are as follows:

(a) Various forms of cyclic load: apparatus can be used to apply triangular wave, rectangular wave, square wave, and sine wave loads. In addition, customized load forms can be defined.

(b) Cyclic loading frequency: This refers to the frequency of the vibration period or irregular, periodic vibration. This test apparatus can apply cyclic load within the range of 0.001Hz-1Hz, which covers covering the common frequency range.

(c) Amplitude and vibration time: Amplitude refers to the magnitude of the dynamic load, and vibration time refers to cyclic loading or the acting duration of vibration. This test apparatus can apply loads with an amplitude range of 0-1MPa.

The samples were subjected to one dimension consolidation test in which static loads, P, or dynamic cyclic loads of amplitude, σ_{dmax}, were applied following surcharge preloading. The loading procedure for cyclic load tests is as follows. In the first loading step, the specimens were consolidated under a sustained load of 25 kPa for 2 hours; in the second loading step, the specimens were consolidated under sustained loads of 50 and 60 kPa for 24 hours; in the third loading step, the consolidation pressure was reduced to 40 kPa, and then was kept under that load for 24 hours; in the final loading step, the specimens were consolidated under the additional cyclic loads of 5, 10 and 20 kPa for a week. In the cyclic consolidation tests, the loading, which varied with different frequencies, was simulated to follow the half-sine wave. The loading scheme of group specimens is summarized in Table 1.

Table 1 Summary of the test scheme

Test type	P (kPa)	ΔP (kPa)	OPR	σ_{dmax} (kPa)	f (Hz)
Static	40	0	1.0	0	0
	40	10	1.25	0	0
	40	20	1.50	0	0
Dynamic	40	0	1.0	5	0.025
	40			10	0.025
	40			20	0.025
	40	10	1.25	5	0.025
	40			10	0.025
	40			20	0.025
	40	20	1.5	5	0.025
	40			10	0.025
	40			20	0.025

In this study, the cyclic loading in Figure 5 is simulated based on [18], shows typical half sinusoidal curves of cyclic loading with T, the period of one cycle and Figure 6 below shows a schematic of consolidation pressure-time relationship for sustained loading, unloading and cyclic loading.

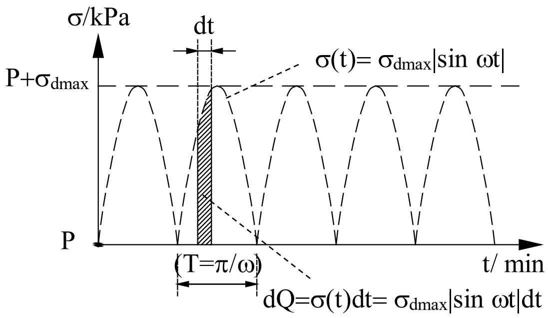


Fig.5 Cyclic loading in typical half-sinusoidal form

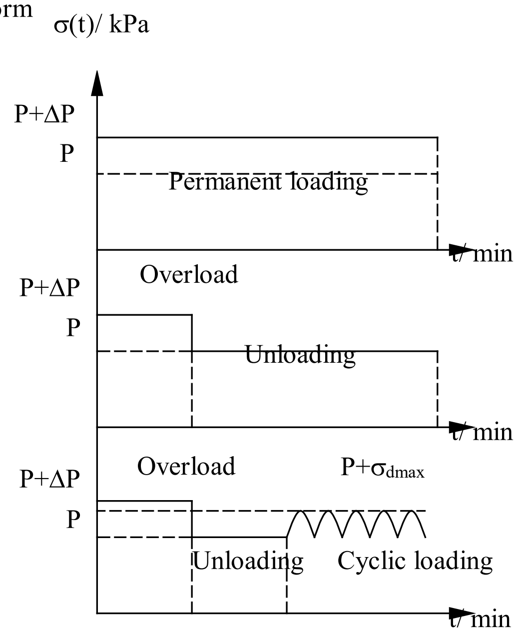


Fig.6 Loading sequence

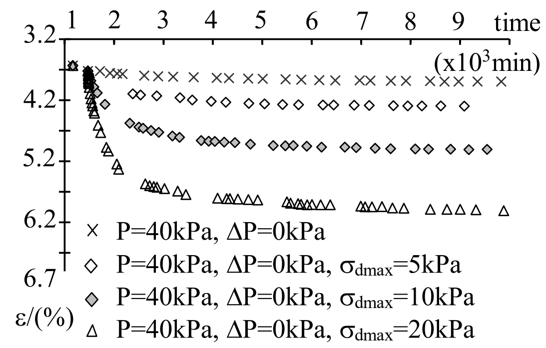
5. RESULTS AND ANALYSIS

5.1 Strain History

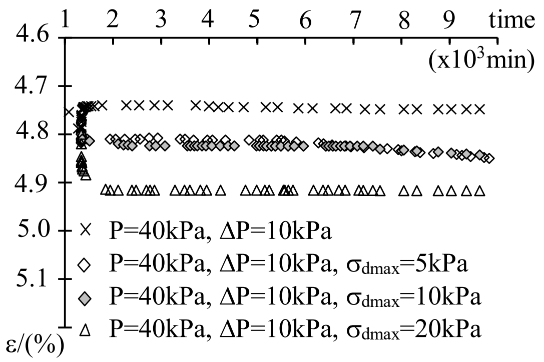
Figure 7 displays the strain-time results for the group test scheme shown in Table 1. Figure 7a shows the results for cases of OPR = 1, where change in strain ($\Delta\varepsilon$) of 0.44%, 1.01% and 2.03% were obtained for σ_{dmax} values of 5kPa, 10kPa and 20kPa, respectively, over the time period of 7 days.

Over the same time period, $\Delta\varepsilon$ values of 0.10%, 0.11% and 0.17%, respectively, were obtained for σ_{dmax} values of 5kPa, 10kPa and 20kPa for the case of OPR value of 1.25 shown in Figure 7b.

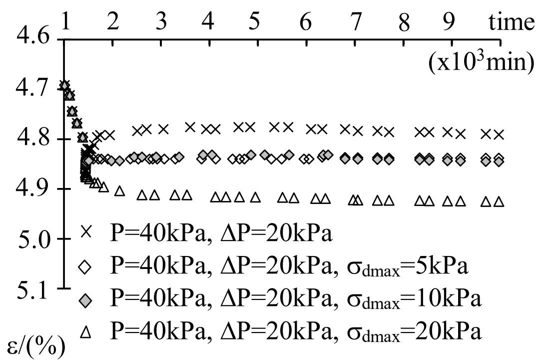
The case OPR = 1.5 shown in Figure 7c recorded the minimum strain variation values of 0.02%, 0.03%, and 0.07% for σ_{dmax} values of 5kPa, 10kPa and 20kPa, respectively.



a) OPR=1.0



b) OPR=1.25



c) OPR=1.5

Fig.7 Strain histories for group test scheme

These strain difference (Δε) values for different values of σ_{dmax} and ΔP are summarized in Table 2 and illustrated in Figure 8.

Table 2 Variability of strain over 7 days

ΔP (kPa)	OPR	Strain variation values Δε (%)		
		σ_{dmax} of 5 (kPa)	σ_{dmax} of 10 (kPa)	σ_{dmax} of 20 (kPa)
0	1.0	0.44	1.01	2.03
10	1.25	0.10	0.11	0.17
20	1.50	0.02	0.03	0.07

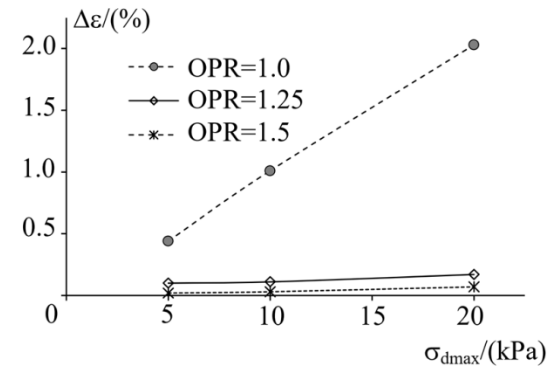


Fig.8 Variations of strain (Δε) with σ_{dmax} for group test scheme

Figure 8 shows that the deformation of the soil samples increases with increasing σdmax and the degree of variation Δε when the OPR is 1.0 increases drastically in comparison with OPR of 1.25 and 1.50. So, the soil deformation will be reduced when the soil is pre-consolidation by surcharge preloading. Therefore, the surcharge preloading method plays a role in reducing the post-construction settlement of embankment under cyclic loading. In addition, considering the values of Δε in the correlation between the magnitude of σ_{dmax} and ΔP shows that Δε is quite small in the cases of ΔP ≥ σ_{dmax}. Thus, the effect of the surcharge preloading method is efficient for ΔP ≥ σ_{dmax}, and the larger the difference between ΔP and σ_{dmax}, the more effective the method is in controlling the post-construction settlement.

5.2 Formula Derivation and Analysis

Based on the test results, a formula has been derived to estimate the post-construction settlement of low embankment under traffic loading after surcharge removal. Due to the discontinuous nature of traffic loading, the proposed formula also considers the influence of the frequency of vehicular passage. In this proposed model, the clay is assumed to be subjected to surcharge loading for a period of time which is sufficient for primary consolidation to be completed. Thus, the strain developed under permanent and cyclic loading consists only of secondary compression and recompression strains. In addition, the time at which the surcharge is removed is assumed as the starting time of cyclic load application.

Therefore, the total vertical strain, ε, of a clay layer under cyclic loading after removal of surcharge comprises two components; the secondary consolidation strain, ε_P, and the recompression strain, ε_σ as given by Eq. (1)

$$\varepsilon = \varepsilon_P + \varepsilon_\sigma \quad (1)$$

LI investigated the secondary compression characteristics and settlement calculation of soft clay under overload unloading and reloading [19]. It was found that the relationship between the coefficient of secondary consolidation ($C_{a(\varepsilon)}$) and over-consolidation ratio (OCR) can be simulated by the simplified hyperbolic model. Based on the model $C_{a(\varepsilon)}$ is defined in Eq. (2)

$$C_{a(\varepsilon)} = \frac{1}{\alpha + \beta.OCR} \tag{2}$$

The coefficient of secondary consolidation is defined by Eq.(3)

$$C_{a(\varepsilon)} = \frac{\varepsilon}{\log t_2 - \log t_1} \tag{3}$$

Combining Eq. (2) and (3) gives the secondary consolidation strain, ε_P; Eq. (4)

$$\varepsilon_P = \frac{1}{\alpha + \beta.OCR}\log\left(\frac{t_2}{t_1}\right) \tag{4}$$

Where, t_1 and t_2 are the starting and finishing time of secondary consolidation under P; α and β are parameters depending on the coefficient of compression of the soft soil and are obtained from results of a series of 1-D consolidation tests which are carried out for soft remolded clay under overload-unloading and reloading ([20]). In this case, α= 3.447, β= 0.872 for values of OCR ≤ 4.0.

The recompression strain (ε_σ) under cyclic load can be calculated by using the strain caused by surcharge load, ΔP. Supposing that the difference in strain between P+ΔP and P is equal to the strain caused by surcharge load ΔP, namely $\varepsilon_{\Delta P}$, then it can be defined by Eq. (5)

$$\varepsilon_{\Delta P} = \varepsilon_{P+\Delta P} - \varepsilon_P \tag{5}$$

Where, $\varepsilon_{P+\Delta P}$, and ε_P are the strains corresponding to P+ΔP and P, respectively, and ε_P is calculated from Eq.(4). Computing $\varepsilon_{P+\Delta P}$ from Eq. (3) and combining with Eq. (4) and substituting into Eq. (5) results in Eq. (6)

$$\varepsilon_{\Delta P} = \varepsilon_{P+\Delta P} - \varepsilon_P = C_{\alpha(N)}\log\left(\frac{t_a + t_2}{t_a}\right) - \frac{1}{\alpha + \beta.OCR}\log\left(\frac{t_2}{t_1}\right) \tag{6}$$

Where, $C_{\alpha(N)}$ is the coefficient of secondary consolidation under a normally consolidated state, ta is taken as the time duration in which primary consolidation is completed under P+ΔP. Realistically, it is difficult to determine the exact completion time of primary consolidation of the soil during and after construction. Thus, in the theoretical calculation, the starting and finishing time of the secondary consolidation need to be corrected by some factors.

LI investigated the mechanism of surcharge preloading on a soft ground highway foundation and observed that the time for primary consolidation under permanent load during reloading would be smaller than that under loading [21], (i.e., $t_a > t_1$), and this is given in Eq. (7)

$$\log\left(\frac{t_a + t_2}{t_a}\right) < \log\left(\frac{t_1 + t_2}{t_1}\right) \tag{7}$$

Thus, t_1 can substitute for ta in Eq. (6), resulting in Eq. (8)

$$\varepsilon_{\Delta P} = \varepsilon_{P+\Delta P} - \varepsilon_P = C_{\alpha(N)}\log\left(\frac{t_a + t_2}{t_a}\right) - \frac{1}{\alpha + \beta.OCR}\log\left(\frac{t_2}{t_1}\right) \tag{8}$$

The difference in time between the removal of the surcharge and the application of cyclic loading is ignored in Eq. (2.31) since the time for unloading is the same as the time of reloading.

Based on the energy concept, the relationship between the strains ε_σ and $\varepsilon_{\Delta P}$ that are generated by cyclic load and surcharge load, respectively, is linear and is expressed by Eq. (9)

$$\varepsilon_\sigma = \varepsilon_{\Delta P}.\zeta \tag{9}$$

Where, ζ is the ratio of the area under the cyclic loading curve to the area under the surcharge loading curve for the same time period. Here, the surcharge load is ΔP, and the cyclic load varies according to the half sinusoidal form with a maximum amplitude of σ_{dmax} and frequency of ω. Thus, ζ is defined by Eq. (10)

$$\zeta = \frac{Q}{\Delta Pt} = \frac{\frac{t}{\pi/\omega}\int_0^{\pi/\omega}\left|\sigma_{d\max}\sin\omega t\right|dt}{\Delta Pt} = \frac{2\sigma_{d\max}}{\Delta P\pi} \tag{10}$$

Substituting ζ from Eq. (10) into Eq. (9) and combining with Eq. (8) results in Eq. (11)

$$\varepsilon_\sigma = \frac{2\sigma_{d\max}}{\Delta P\pi}\left\{C_{\alpha(N)}\log\left(1 + \frac{t_2}{t_1}\right) - \frac{1}{\alpha + \beta.OCR}\log\left(\frac{t_2}{t_1}\right)\right\} \tag{11}$$

From Eq. (1), Eq. (4) and Eq. (10), the total vertical strain of a clay layer is

$$\varepsilon = \frac{1}{\alpha + \beta.OCR}\log\left(\frac{t_2}{t_1}\right) + \frac{2\sigma_{d\max}}{\Delta P\pi}\left\{C_{\alpha(N)}\log\left(1 + \frac{t_2}{t_1}\right) - \frac{1}{\alpha + \beta.OCR}\log\left(\frac{t_2}{t_1}\right)\right\} \tag{12}$$

If the soil layer under the embankment is divided into n layers, with hi being the average thickness of the ith layer, then the post-construction settlement of the embankment under dynamic traffic load can be given by Eq. (13)

$$S = \sum_{i=1}^{n}\varepsilon_i^* h_i = \sum_{i=1}^{n}\left[\frac{h_i}{\alpha + \beta.OCR}\log\left(\frac{t_2}{t_1}\right) + \frac{2\sigma_{d\max}h_i}{\Delta P\pi}\left\{C_{\alpha(N)}\log\left(1 + \frac{t_2}{t_1}\right) - \frac{1}{\alpha + \beta.OCR}\log\left(\frac{t_2}{t_1}\right)\right\}\right] \tag{13}$$

5.3 Verification of Calculation Assumptions with Test Results

Figure 9 shows a nearly linear relation between variation of strain, ε_σ and $\varepsilon_{\Delta P}$, which are obtained from test results for the group test scheme. The ζ ratio is equal to the slope of the straight lines in Fig. 10.

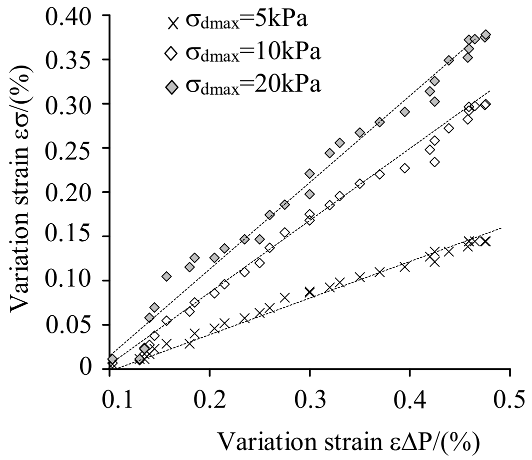


Fig.9 Relation between ε_σ and $\varepsilon_{\Delta P}$ generated by cyclic load and surcharge load, respectively, for group test scheme

Figure 10 also indicates that values ζ calculated from Eq. (10) are consistent with that from test data. Thus, the theoretical assumptions are reasonable.

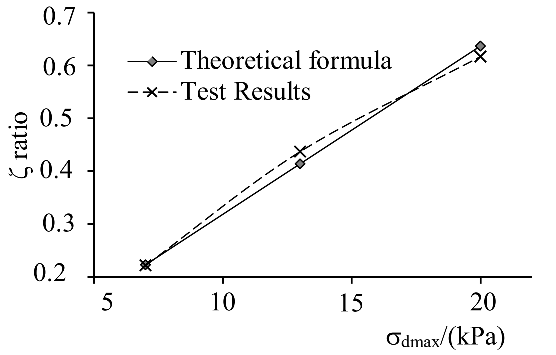


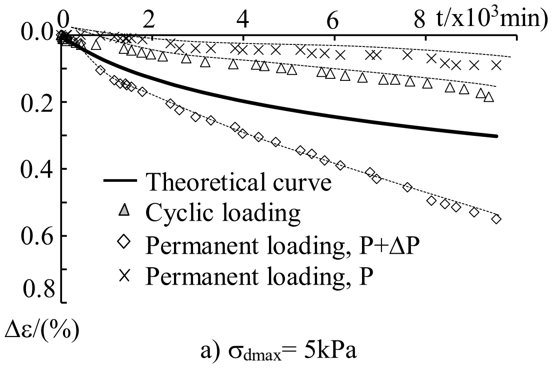
Fig.10 Comparison of ζ ratio from theoretical formula and test results

Figure 11 (a-c) shows the comparison of strain histories of measured and theoretical (i.e., determined from Eq. (8)) data under sustained loading, unloading and cyclic loading. In Eq. (8), $C_{\alpha(N)}$ ratio is determined from group test data and t_1, the time at which the surcharge is removed, is 1440min (1 day). The origin corresponds to the starting time of the application of cyclic loading. The theoretical curves lie between the strain curves for P and P+ΔP. And this indicates that the theoretical calculations are fairly consistent with the test results.

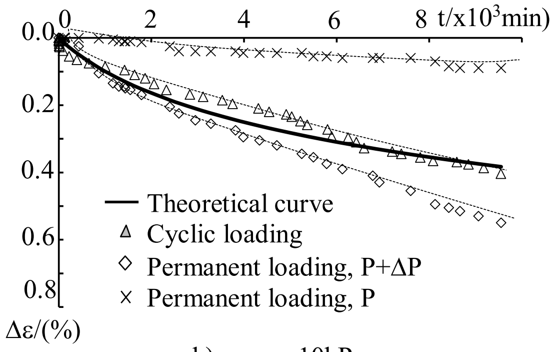
6. APPLICATION OF THE PROPOSED FORMULA

6.1 Project Overview

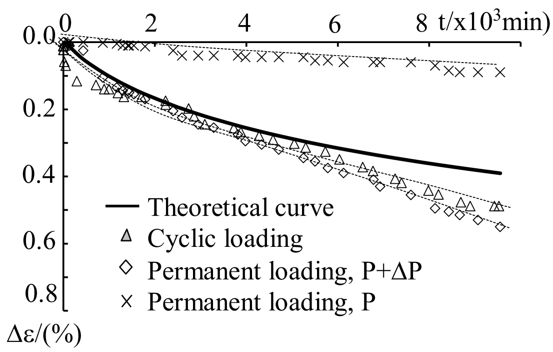
The proposed formula is applicable to evaluating the long-term settlement of surcharge preloaded low embankment on soft ground subjected to cyclic loading. Total settlement data from the K34+850-K38+000 section of the Jie Pu highway (in Guangdong province) has been used to validate the proposed formula. It should be noted, however, that the total settlement comprises the consolidation settlement due to embankment load and the settlement induced by traffic loading. Thus, the settlement prediction was made by the summation of settlements under both embankment and traffic loading. The extent of settlement was calculated using a one-dimensional consolidation analysis, taking into account the effects of cyclic loading time.



a) σ_{dmax}= 5kPa



b) σ_{dmax}= 10kPa



c) σ_{dmax}= 20kPa

Fig.11 Comparison of strain histories of measured and calculated data

Construction of the Jie Pu highway, which is situated on a soft soil deposit, started in 2011 and opened to traffic in 2013. The road was constructed on a typical low embankment road on a lowland plain in Guangdong province of China. Field measurements were performed along the K36+150 section to investigate post-construction settlement. The properties of the soil at section K36+150 are presented in Table 3, which shows that the soil at the site comprises, from top to bottom, loose fill, silty sand, silty mud, sandy clay, muddy clay and coarse sand.

Table 3 shows that the alluvial deposit consists of two layers: silty mud and peaty clay layer, with marine silty sand, and a sandy clay layer, which are sandwiched at a depth of 2.7 m to 5.6m and 8.7 m to 17.0m, respectively. A coarse sand layer exists below a depth of 17.0m. All of these layers may act as drainage layers. The overburden stresses on the ground are normally consolidation states compared with consolidation yield stresses along the depth direction. Figure 12 shows the geological profile of section K36+150 and the ground treatment.

Table 3 Index properties of soil at section K36+150

Soil layer	h	w	γ	e	α	I_p
Silty sand	2.7	27.4	19.4	0.70	0.3	13
Silty mud	2.9	72.5	14.7	1.88	2.1	16
Sandy clay	3.1	35.0	19.1	0.65	0.3	12
Muddy clay	8.3	50.7	13.3	1.32	0.74	-
Coarse sand	29.3	21.3	18.6	0.75	0.37	10

With *h* (m) as the thickness of the soil layer, *w* (%) is moisture content; γ (kN/m$^{3)}$ is unit weight; *e* is void ratio; α (MPa^{-1}) is compression factor and I_p is plasticity index.

The 26m wide four-lane road pavement was designed for two-way traffic at a speed of 120km/h. In order to reduce and control the post-construction settlement of the embankment, the surcharge preloading method combined with the sandy wick was used. The surcharge load had a height of 0.5 to 2.5 m, while the sandy wicks had a diameter and length of 0.07 m and 16 ~ 18m, respectively, and were arranged in a triangular pattern with 1.2m spacing. The subgrade was placed by using a sand cushion layer with a thickness of 0.5m.

6.2 Settlement Due to Embankment and Traffic Loading

In Table 3, there are two weak soil layers with quite large thicknesses beneath the embankment-the 2nd of silty mud and the 4th layer of muddy clay. The settlement of the ground under the impact of traffic loads and the self-weight of the embankment will be concentrated mainly in these layers. So in this section, the calculation considers the settlement of the 2nd and 4th layer only-the settlement of the other layers will not be dealt with. Furthermore, since the 4th layer lies well below the embankment at a depth of 8.7m from the surface, it should hardly be affected by the traffic load.

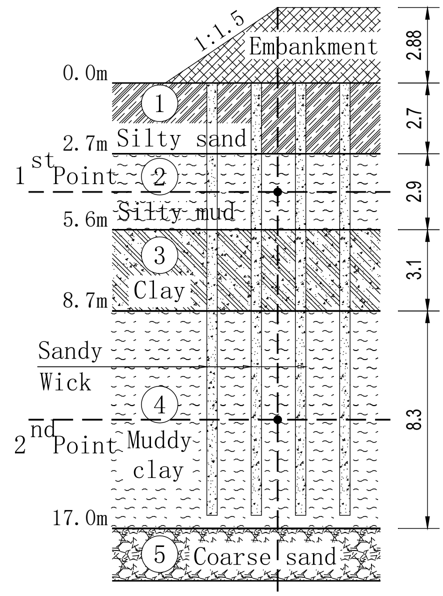


Fig.12 K36+150 section of the Jie Pu highway

Thus, the total post-construction settlement of the low embankment was obtained from the summation of the predicted settlements due to both embankment and traffic loading for the 2nd layer, with the settlement of the 4th layer computed only embankment loading and self-weight of soil above this layer. The settlement was calculated using one-dimensional consolidation analysis, with the parameters determined from the data shown in Table 4, in which the coefficients of consolidation were C_α = 8.53x10^{-4}mm^2/s in the 2nd layer and 6.41x10^{-4}mm^2/s in the 4th layer.

Table 4 Parameters for calculation of the settlement of embankment at K36+150

Layer	h/m	P/kPa	ΔP_1/kPa	ΔP_2/kPa	OCR	α/10^2	β/10^2	C_α/ 10^{-3} mm^2/s
2nd	2.9	44.2	87.8	56.4	1.31	3.48	1.36	8.53
4th	8.3	92.9	77.1	48.4	1.20	2.09	1.88	6.41

With h is the thickness of the soil layer; P is the average stress due to the self-weight of the soil layer; ΔP1 and ΔP2 are the average stress due to surcharge preloading before and after unloading, respectively; OCR is the over consolidation ratio, defined as (P+ΔP)/P; α, β are the secondary consolidation parameters, obtained from results of a series of 1-D consolidation tests (Li et al. 2016 [16]); C_α = coefficients of consolidation.

Table 4 shows the comparison of the calculated and observed settlement for the K36+150 section, S_i, the calculated settlement, is calculated according to Eq.(13). In Eq.(13), the ratio of acting time of traffic load to the total time was assumed equal to 1; t_1, the starting time of the secondary consolidation, is taken as the time for removal of surcharge (i.e., 50 days); t_2, the finishing time of secondary consolidation, is taken as the time settlement was measured after the road was opened to traffic. It should be noted that measurements were taken 50 days after construction, during which time the calculated settlement, according to the proposed formula, assumes that settlement under traffic loading occurs under conditions of drained cyclic loading, in which the excess pore pressure generation was dissipated completely.

Figure 13 shows the comparison of the calculated and observed settlement history of the embankment at the K36+150 section. The two generally compare favorably, apart from the deviations in the curves in the initial stages, which could be attributed to the fact that the assumptions of the formula could not exactly follow the field process.

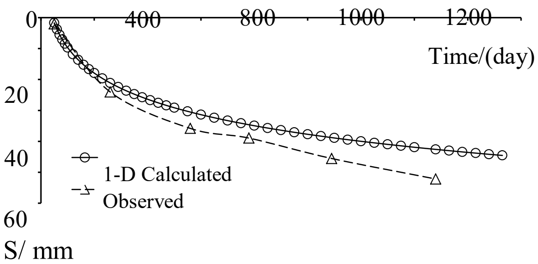


Fig.13 Calculated and observed settlement histories

However, the calculated settlement is smaller than the observed values and the difference (ΔS) increased with elapsed time. ΔS values of 2.95, 5.31, 4.35, 6.79 and 9.61 mm were obtained for elapsed time values (Δt) of 260, 560, 780, 1090 and 1480 days, respectively. The difference between calculated and measured values can be explained as the effect of a dynamic factor of traffic loading might not be touched fully in the proposed formula and the ratio of acting time of traffic load to the total time was larger than 1.0, as assumed. In addition, the amplitude of the dynamic cyclic load was bigger than the value of the surcharge, which led to the large secondary consolidation deformation of soil after the removal of the surcharge.

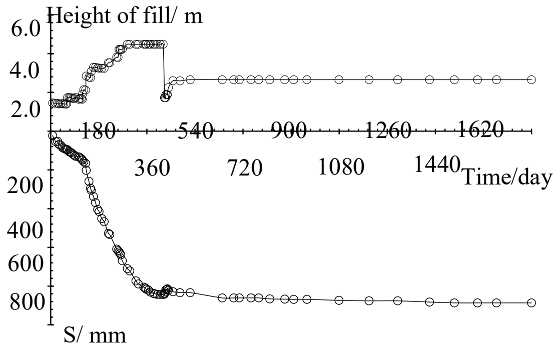


Fig.14 Field settlement histories at K36+150 section

Figure 14 shows that the variation in post-construction settlement with elapsed time depends on the process of surcharge loading and removal. The settlement developed rapidly in the early stages with increasing embankment height during preloading. When the height of the embankment is reduced during surcharge removal, the settlement is also reduced and its variation becomes almost constant at the final embankment height.

7. CONCLUSIONS

One-dimensional consolidation tests on saturated soil subjected to cyclic loading have been used to predict the total post-construction settlement of low embankment. The following conclusions were obtained from this study:

1) The post-construction settlement increases with the increasing amplitude of the cyclic load, and the effectiveness of surcharge preloading depends on the difference between the surcharge and the amplitude of the cyclic load. Comparatively larger surcharge results in smaller post-construction settlement.

2) The theoretical curves for dynamic loading, which were plotted from a combination of the coefficients determined from proposed formulas and test data, are fairly consistent with the test results.

3) The post-construction settlement of a low embankment was observed and compared using the proposed formula. The results show that the calculated post-construction settlement is highly consistent with the observed data; a longer duration of traffic cyclic loading results in larger post-construction settlement, and the rate of settlement increment reduced with the increasing elapsed time. The analysis demonstrates that the settlement induced by traffic loading is approximately 18.4% of the total settlement during a period of four years.

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