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Research Article

Settlement of Funing Natural Clays under 14-Year Embankment Loads: A Case Study

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The settlement behavior of soft natural clays under a 14-year embankment load is investigated by obtaining the high-quality undisturbed samples of soft natural clays beneath a constructed embankment. It is found that the conditional way of determining the settlement by combining the layerwise summation method of calculating settlement, the Osterberg method of determining the embankment load-induced stress in subsoil, and the compression parameters determined from high-quality undisturbed samples overestimates the settlement, compared with the field observations. The key cause is that the Osterberg method overestimates the vertical superimposed stress in this case. The modified method of determining the vertical stress in subsoil suggested by the authors is adopted for improving the accuracy of vertical superimposed stress calculated. The balance among errors in the settlement calculation model, determination of vertical superimposed stress induced by embankment loads, and measurement of compression parameters are also discussed.

1. Introduction

It has been well recognized that the constructions of embankments over the soft natural clay are often beset by settlement problems at various degrees [1–4]. Settlement analysis of soft ground with natural clays under embankment loads inevitably involves three issues: calculation model; determination of vertical superimposed stress in subsoil (σ_s'); and measurements of calculation parameters.

The layerwise summation method is generally adopted as a simple way of calculating embankment load induced settlements with the one-dimensional assumption [3–6]. The soil profile is subdivided into layers, and the mean values of effective stress and soil compression index at each corresponding layer are used for settlement calculations. The Osterberg method is often adopted as a conventional way of determining the vertical superimposed stress (σ_s') in subsoils [1, 7]. The Osterberg method was established based on the assumptions of instantaneous loading from embankment constructions, perfectly flexible loaded area along the base of embankments, and linear elastic theory of soils [8, 9]. On the contrary, the compression parameters are generally obtained from the experimental results of one-dimensional incremental load consolidation tests performed on undisturbed specimens sampled from different soil layers below ground surface [1, 5]. It has been well documented that the laboratory observations of compressibility may be significantly affected by sample disturbance [10–13], consolidation stress paths [14–17], testing methods [18–20], soil microstructure [21, 22], etc.

Note that the probable errors caused by the assumptions in calculating model, the determinations of σ_s', and the observations in compression parameters should be in balance for obtaining the settlements with an accepted accuracy. Several researchers reported that the degrees of accuracy in the calculated results vary from good agreement

[23–25] to poor agreement with the field observations [26–29]. A case study is a powerful way of understanding the key factors responsible for difference between calculated and observed settlements for natural clays induced by embankment loads. Note that case studies have been little reported in China on embankment load-induced settlements over soft natural clays.

The first-phase engineering project of Huaihe River Waterway had been constructed for three years from 1999 to 2002. The planning on the second-phase project of Huaihe River Waterway provides us an opportunity to investigate the settlement behavior of natural clays under a 14-year embankment load. This study obtained undisturbed samples at different depths below ground surface from two holes at the central point of the embankment and out of the embankment, respectively. The physical properties of samples from two holes are compared to identify that the samples out of the embankment can be used to measure compressibility of natural clays before embankment constructions. Then, the settlement under embankment loads is analyzed using the Osterberg method. The calculated settlement is compared with the observation to check the behavior of σ_s' determined by the Osterberg method in this case, and a modified way suggested by Wang et al. [30] is adopted for improving the accuracy of calculations on σ_s'. The values of σ_s' obtained by the Osterberg method and the modified way are compared with the yield stresses by one-dimensional consolidation tests. Finally, the most probable factors of controlling the calculations of settlements are discussed.

2. Project Background and Measured Settlement

Figure 1 shows the location of the embankment investigated which locates at Huiahe River Waterway. The Huaihe River is a flood-prone river in China that originates from Tongbai Mountain, flows through Henan and Anhui Provinces, and pools up into the Hongze Lake in Jiangsu Province. The Huaihe River Waterway was designed to link up Hongze Lake with Yellow Sea for enhancing the flood discharge of Huaihe River and Hongze Lake. The first-phase waterway project was constructed from 1999 to 2002 for increasing the floor control standard of the Hongze Lake from once every 50 years to once every 100 years. The first-phase of the waterway project was about 162.3 km. There are three different geological deposits along the waterway, stiff deposit (K3–K57), soft deposit (K57–K108), and sand deposit (K108–K162) [31].

The soft deposit in the first-phase of the waterway project has a length of about 52 km. Based on the experience and stability analysis, the embankments over soft deposit were stage-constructed without any reinforcement of soft foundation. To monitor the magnitude and the rate of the soft foundation settlement, the surface settlement gauges were installed during the construction. Figure 2 shows the development of surface settlement at the centerline of embankment with elapsed time during and after construction, based on the observations reported by JPEIRI [32]. It can be seen that the surface settlement at the end of construction is 1.35 m. Note some settlement gauges were destroyed or damaged caused by the flooding emergency in July 2003. The reconstruction of the monitor system was conducted in August 2005. The observations were continued to December of 2013. As shown in Figure 2, the maximum post-construction settlement under the embankment load is recorded as 1.7 m in December 2013 [32]. Figure 3 shows the settlement rate based on the settlement-time curve in Figure 2. It can be seen that the settlement rate decreases with the increase in elapsed time after construction. The settlement rate varies from the 2.11 mm/d in August 2002 to 0.78 mm/d in August 2005, 0.50 mm/d in July 2006, 0.41 mm/d in October 2010, and 0.26 mm/d in December 2013.

Nowadays, the State Council of China [33] has approved of the project for further reducing the frequency of major downstream flooding of Hongze Lake to once every 300 years. The second-phase of Huaihe River Waterway project will broaden and deepen the entire channel and strengthen the existing embankments on the basis of first-phase project [31].

3. Undisturbed Samples of Soft Deposit and Physical Properties

Based on the information of site investigation reported by JPEIRI [34, 35], a typical subsoil profile can be illustrated in Figure 4(a). The subsoils beneath the embankment can be divided to eight soil layers. The top layer is a thin crust (TC) of approximately 1.0 m thick; below the TC, there are four soft clay layers (labeled as SC-1, SC-2, SC-3, and SC-4) identified by detailed physical tests described later; their average thicknesses are 3.5 m, 6.4 m, 6.0 m, and 7.4 m, respectively. These soft deposits were laid down during the quaternary period [36]. Next are three layers of stiff sand clay (SSC-1, SSC-2, SSC-3) extending down to 37.9 m depth, and their N values of standard penetration test (SPT) are 8, 45, and 50, respectively.

The planning on the second-phase project of Huaihe River Waterway calls for further detail site investigations by performing laboratory tests on high-quality undisturbed samples. Taking this opportunity, we obtained the undisturbed samples of soft natural clays beneath the embankment constructed at K85.5 section of the first-phase engineering project. For comparing the physical and mechanical behaviors of undisturbed samples with and without embankment loads, the undisturbed samples of soft natural clays under the ground surface out of the embankment (50 m distance from the centerline) were also sampled, as shown in Figure 4(b).

A thin-wall free-piston sampler was used to obtain two holes of undisturbed samples at the depths of 3 m, 6 m, 9 m, 12 m, and 15 m below ground surface. The hole under the center of embankment was termed as Em borehole, and the hole out of the embankment is termed as Gr borehole, as shown in Figure 4(b). The number after the borehole represents the depth of the samples. For example, the Em_U9m represents the undisturbed sample obtained from a depth of 9 m at the embankment borehole.

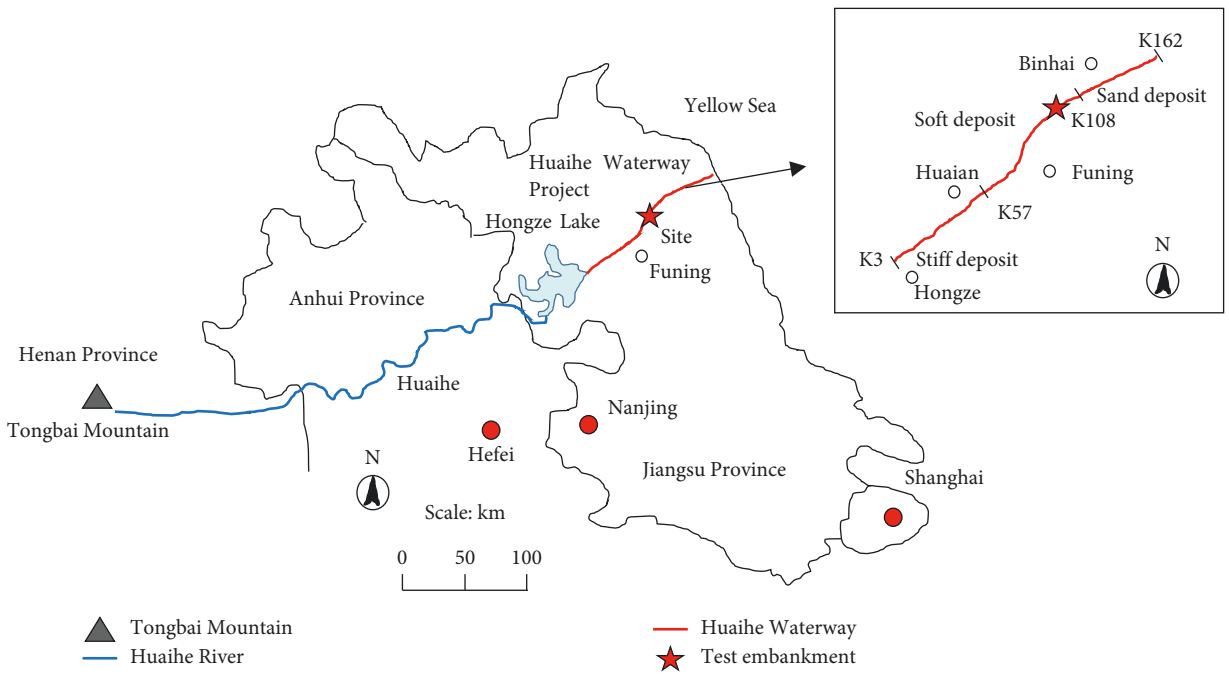

Figure 1: Location of embankment and sampling.

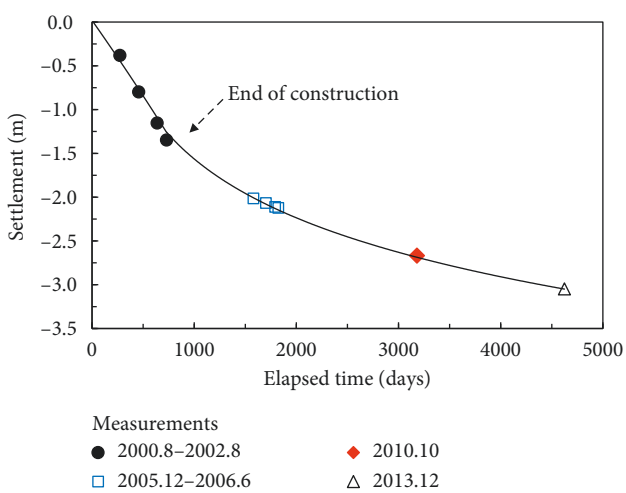

Figure 2: Development of surface settlement with elapsed time during and after construction.

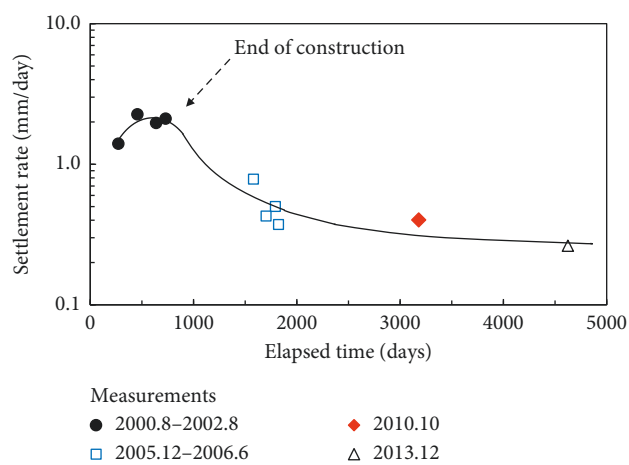

Figure 3: Development of settlement rate with elapsed time during and after construction.

For evaluating the sample quality, the method suggested by Lacasse et al. [10] was adopted with measuring volumetric strain (ε_{v0}) at the effective overburden stress (σ'_{v0}). The ε_{v0} was calculated as follows: $\varepsilon_{v0} = (e_0 - e_{v0})/(1 + e_0) \times 100\%$. The term e_0 and e_{v0} represent the initial void ratio and the void ratio under σ'_{v0}, respectively. The values of σ'_{v0} were calculated by $\gamma' H$, where γ' is the effective unit weight of soil and H is thickness of the soil layer. As suggested by Lacasse et al. [10], the sample quality can be classified as good or fair corresponding to the values of ε_{v0} within 0–2% or 2–4%, respectively. Figure 5 shows the sample qualities of the undisturbed samples from Gr borehole and Em borehole, and the most values of ε_{v0} varied from 1.4% to 4%. It can be concluded that the qualities of investigated samples are good or fair. Note that embankment loads were not considered in calculating σ'_{v0} for undisturbed samples from the Em borehole. Embankment loads will induce vertical superimposed stress in subsoils and decrease the values of e_0. On the contrary, the embankment loads will cause destructuration of natural soft deposits [13], consequently resulting in the decrease in the values of e_{v0}. More studies are required to investigate the balance among the variation in e_0 and e_{v0}, the destructuration, and the vertical superimposed stress for evaluating sample quality of natural clays subjected to embankment loads.

Table 1 shows some physical properties of the samples investigated. The liquid limits (w_L) were measured using the Casagrande method according to Head [18]. The plastic limit tests were also conducted in accordance with rolling thread suggested by Head [18]. It can be seen that the investigated clays have a wide spectrum of w_L, ranging from 50% to 81%.

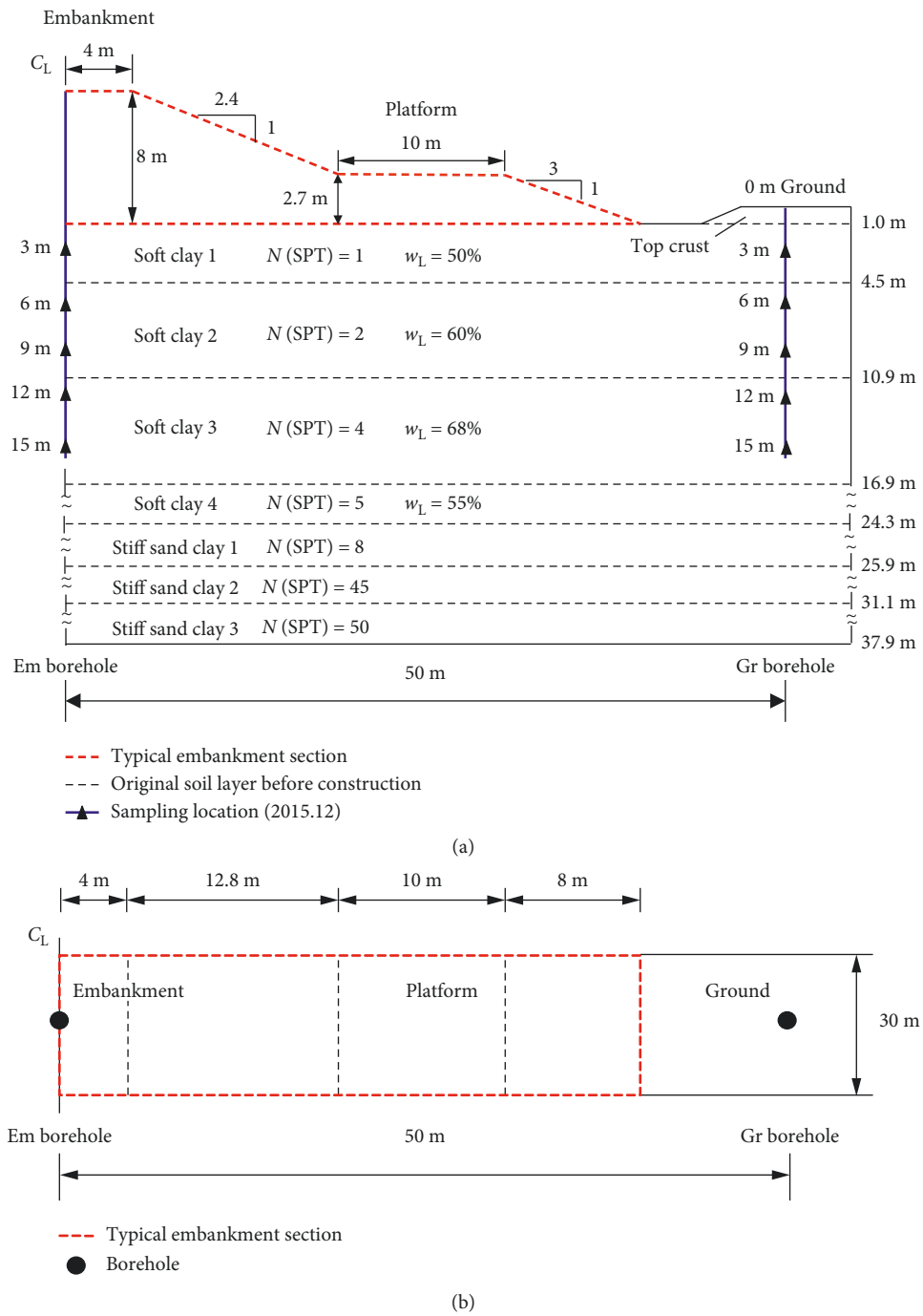

Figure 4: Test site N88K: (a) cross section of embankment and soil profile before construction; (b) borehole location.

Figure 6 shows the plasticity chart, indicating that all samples lie above the A-line defined by the Unified Soil Classification System [37]. The Funing soft clays can be classified as high plasticity clay (CH).

From Figure 6, it can be also seen that the investigated clays can be divided into four groups with the values of w_L: (1) $w_L = 50\%–51\%$; (2) $w_L = 58\%–60\%$; (3) $w_L = 68\%–70\%$; and (4) $w_L = 80\%–81\%$. It is interesting to note that the w_L value of Gr_U3m is approximately equal to that of Em_U6m. That is, the sample of Em_U6m with embankment loads is identical to that of Gr_U3m without embankment loads, with considering the total settlement of 3.05 m during and after the construction of embankment. Meanwhile, the value of w_L for Gr_U6m is almost exactly the same as that of the Em_U9m. Furthermore, the w_L values of Gr_U12/15m are in good agreement with the values for Em_U12/15m. Detail information on embankment load-induced changes in subsoil layers can be seen in the later section.

On the contrary, Figure 7 shows the particle size distributions curves for all tested soil samples, which were measured by hydrometer analysis as described by Head [18]. It is interesting to note that there are four type of patterns in

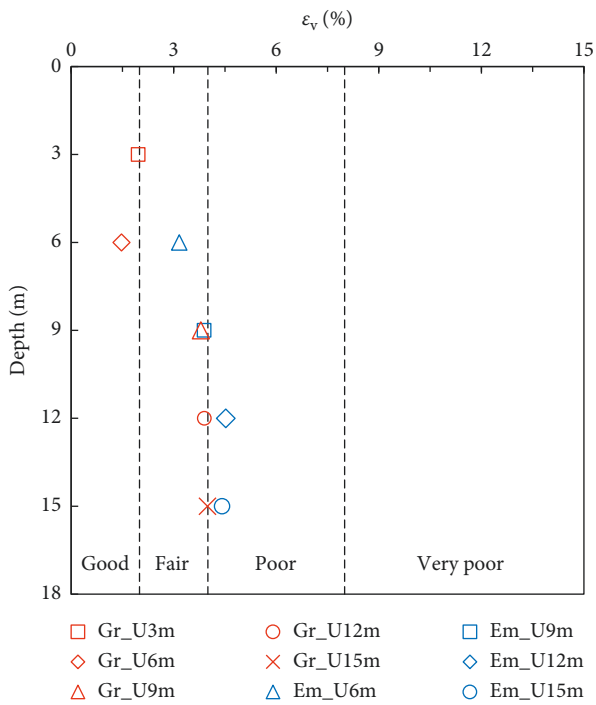

Figure 5: Sample quality classification.

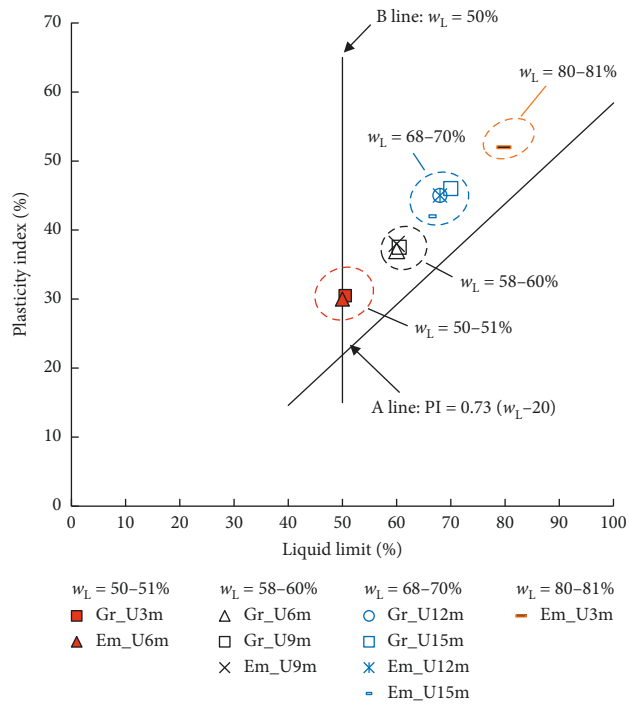

Figure 6: Plasticity chart of investigated soils.

Table 1: The oedometer test for undisturbed samples.

Sample	G_s (g/cm^3)	w_n (%)	w_L (%)	w_p (%)	w_n/w_L	Consolidation stress (kPa)
Gr_U3	2.72	49.8	50	20	0.99	
Gr _U6	2.72	59.8	60	23	0.99	
Gr _U9	2.70	59.7	60	23	0.99	
Gr _U12	2.70	56.7	68	23	0.83	6.25-12.5-25-50-
Gr _U15	2.70	57.5	71	24	0.81	100-200-400-800-
Em_U6	2.70	41.0	50	20	0.82	1600
Em_U9	2.69	46.9	60	22	0.78	
Em_U12	2.70	47.9	68	23	0.70	
Em_U15	2.69	48.9	66	24	0.74	

the plot, corresponding to $w_L = 50\%–51\%$; $w_L = 58\%–60\%$; $w_L = 68\%–70\%$; and $w_L = 80\%–81\%$, respectively. It can be seen that the grade curve of Gr_U3m is almost identical to the Em_U6m. For the Gr_U6m, its grade distribution is consistent with the Em_U9m. The grade distribution curves of Gr_U12/15m are almost the same as those of the sample of Em_U12/15m. Hence, the Em_U6m, Em_U9m, and Em_U12/15m can be considered belonging to the SC-1 layer (1.0 m–4.5 m), SC-2 layer (4.5 m–10.4 m), and SC-3 layer (10.4 m–16.9 m) in preconstruction stage, respectively. Note that the samples of Em_U3m are the embankment fill materials [31].

The above physical properties of the Atterberg limits and particle size distributions indicate that the foundations can be considered as horizontally homogeneous before embankment was constructed. That is, the samples out of the embankment can be used to measure compressibility of natural clays for calculating embankment load induced settlement.

4. Assessment on Embankment Settlements

It is recognized that the natural sedimentary clays are generally subjected to soil structure effects developed during the depositional and the postdepositional processes [1, 38–43], resulting in the vertical yield stress (σ_p') often being larger than the effective overburden pressure. The soil structure results relatively the small deformation of natural clays under effective vertical stress up to the consolidation yield stress. The compressibility of clays increases dramatically when the applied load exceeds the yield stress [39, 44–47]. The settlement of natural sedimentary clays under σ_v' can be calculated using the following equations:

$$S = \frac{H}{1+e_0} C_r \log_{10} \frac{\sigma_{v0}' + \sigma_s'}{\sigma_{v0}'}, \quad \text{when } \sigma_{v0}' \le \sigma_v' \le \sigma_p',$$

$$S = \frac{H}{1+e_0} \left(C_r \log_{10} \frac{\sigma_p'}{\sigma_{v0}'} + C_c \log_{10} \frac{\sigma_{v0}' + \sigma_s'}{\sigma_p'} \right), \quad \text{when } \sigma_v' > \sigma_p', \tag{1}$$

where the effective vertical stress (σ_v') in subsoil can be calculated as $\sigma_{v0}' + \sigma_s'$. σ_s' represents vertical superimposed stress and C_r represents the compression index at the preyield zone when $\sigma_{v0}' \le \sigma_v' \le \sigma_p'$ and is expressed as $C_r = \Delta e/\log(\sigma_v'/\sigma_{v0}')$. C_c is the compression index at the postyield state when $\sigma_v' > \sigma_p'$ and is obtained by $C_c = \Delta e/\log(\sigma_v'/\sigma_p')$.

The compression parameters (C_r, C_c, σ_p') were determined by one-dimensional incremental load consolidation tests on undisturbed natural clay obtained from ground borehole. All the specimens had a diameter of 61.8 mm and an initial height of 40 mm. The loading steps ranging from

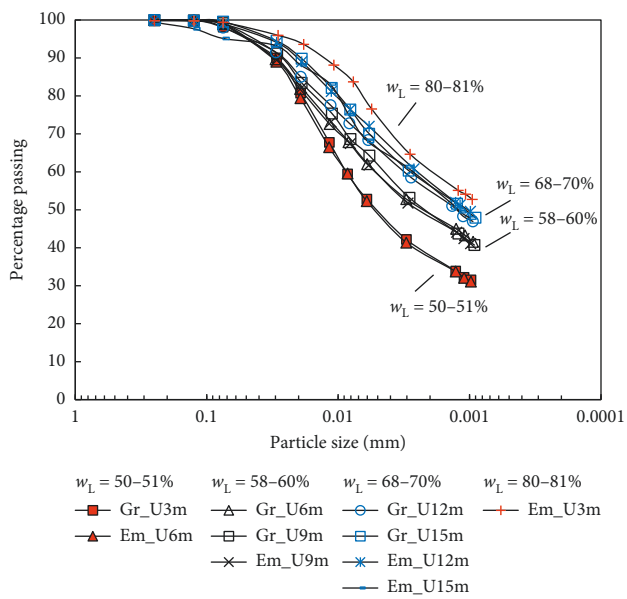

Figure 7: Grading curve of investigated soils.

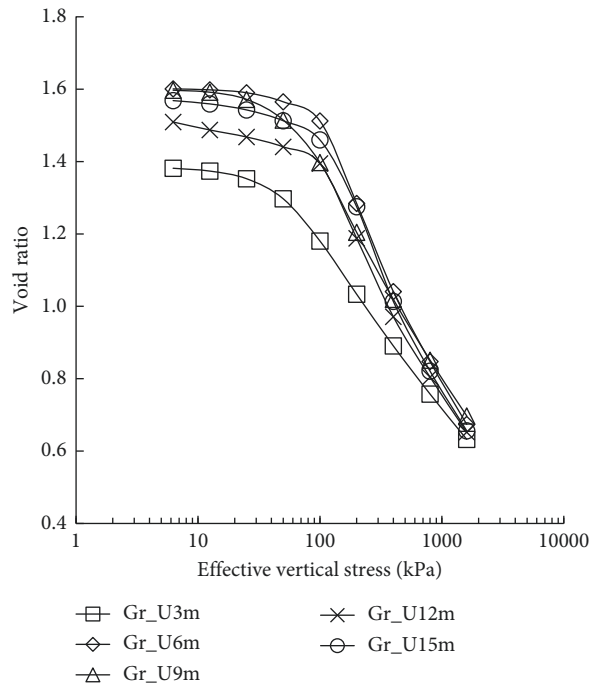

Figure 8: Compression curves of natural clay.

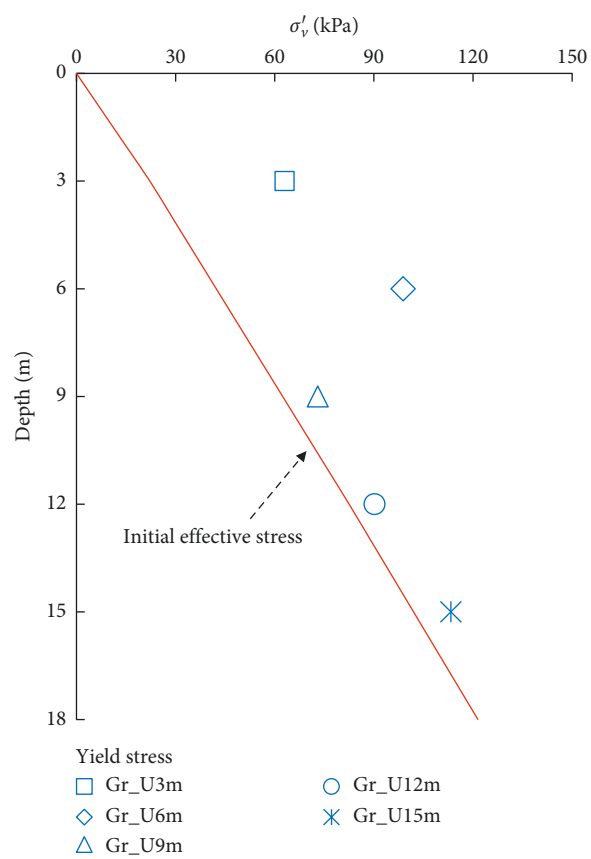

Figure 9: Comparison between the yield stress and initial effective stress.

6.25 kPa to 1600 kPa by doubling the load for each increment and the duration of every loading increment was about 3 days, following Zeng et al. [48]. Detailed test program is listed in Table 1.

It can be seen from Figure 8 that the compression curves of undisturbed samples show a typical inverse 'S' shape as a result of the effects of soil structure. The relationship between σ'_p and σ'_{v0} is shown in Figure 9. The consolidation yield stress (σ'_p) for all the investigated specimens was determined by the Casagrande method. σ'_p values were all larger than σ'_{v0} values due to the soil structure during depositional and postdepositional processes [40]. The values of C_r, C_c, and σ'_s are shown in Table 2. Note the values of σ'_s in Table 2 were calculated by the Osterberg method.

Figure 10 shows the comparisons between the calculated settlements and field measurements. It can be seen that the calculated settlement is approximately 14% larger than the field after construction measurement of 1.7 m. Note that the Osterberg method of determining σ'_s is based on the assumption of instantaneous loading, perfectly flexible loaded area along the base, and soil elasticity. These assumptions are far from real condition of embankment [9, 49]. The assumptions in the Osterberg method results in oversimplifying actual embankment condition, leading to the significant miscalculations of embankment load induced settlements, as reported by Wang et al. [30].

5. Vertical Superimposed Stress in Subsoil

In the calculation of embankment load induced settlements, quantitatively evaluating the contact stress along the base (σ'_b) and vertical superimposed stress in subsoil (σ'_s) is great importance [8]. Wang et al. [30] proposed a modified way of determining σ'_s for overcoming the assumptions of instantaneous loading, perfectly flexible loaded area along the base, and soil elasticity in traditional approach using the geometric parameters of embankment and the friction angle of fill material (ϕ). Moreover, a reduction coefficient of 0.85 is suggested for considering the effect of elastoplastic nature

Table 2: The calculated results.

Soil	Depth (m)	H (m)	e_0	C_c	C_r	σ_{pi} (kPa)	σ_{v0i} (kPa)	σ_{si} (kPa)	S (m)
SC-1	1.0–4.5	3.5	1.39	0.47	0.07	63	20	121	0.29
SC-2	4.5–6.9	2.4	1.60	0.72	0.04	99	39	115	0.14
	6.9–10.9	4.0	1.61	0.60	0.09	73	49	110	0.35
SC-3	10.9–13.9	3.0	1.54	0.64	0.08	90	82	102	0.24
	13.9–16.9	3.0	1.58	0.74	0.11	113	102	95	0.21
SC-4	16.9–24.3	7.4	1.54*	0.44*	0.2*	156	156	85	0.28
SSC-1	24.3–25.9	1.6	0.85*	0.12*	0.07*	167	167	78	0.02
SSC-2	25.9–31.1	5.2	0.53*	0.04*	0.03*	189	189	77	0.02
SSC-3	31.1–37.9	6.8	0.65*	0.1*	0.09*	225	225	71	0.05
								Total	1.59

*Obtained from the site survey (Jiangsu Province Engineering Investigation and Research Institute, 2010, 2013).

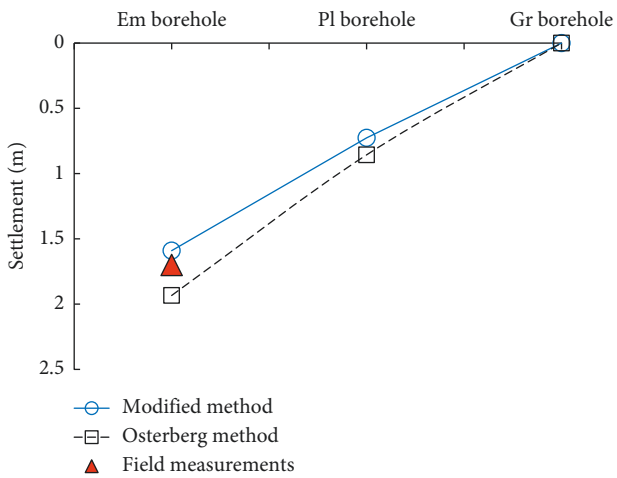

Figure 10: Comparisons of settlements between the calculated and the measured values.

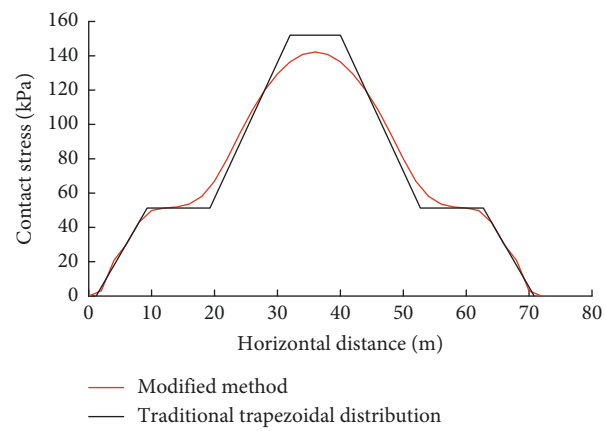

Figure 11: Contact stress along the base.

of subsoil. Note the geometric parameters in Figure 4 and $\phi = 30^\circ$ [31] were adopted for analysis in this study.

The typical comparisons of predicted contact stresses along base between the modified method and traditional method are shown in Figure 11. It can be seen that the contact stresses predicted by modified method are close to a bell-shaped distribution that yields lower stresses at the central portion of embankment and higher stresses near outer edge by comparison with the traditional method. Figure 12 depicts the typical comparison of vertical superimposed stresses at embankment boreholes. It can be observed that the modified method yields lower stress than the traditional Osterberg method, up to approximately 24 kPa at the centerline of the embankment. These discrepancies are attributed to the effects of the contact stress along the base and elastoplastic behavior of subsoil on load transfer [30].

The calculated settlements with the modified method of determining σ_s' by Wang et al. [30] are presented in Figure 10. It can be seen that the predicted settlements using the modified method of determining σ_s' are consistent with the field measurements. Figure 13 presents the varied depths of the soil stratification beneath embankment determined by the calculated settlements with the modified method in Table 2 and recorded construction settlements. It is encouraged to find that the varied depths of soil stratification are consistent with the results of liquid limit and particle size distributions tests.

Note the values of σ_s' obtained by the Osterberg method and the modified way are compared with the yield stress determined by one-dimensional incremental load consolidation tests on undisturbed samples of natural clay obtained from embankment borehole, as shown in Table 2. Figure 14 depicts that the predicted effective stress using the modified method yields slightly higher than the yield stresses. This result is attributed to the uncompleted primary consolidation in the soft foundation. The calculated results with the Osterberg method are also shown in the same figure, again indicating that the Osterberg method may significantly overestimate the values of σ_s'.

6. Key Factors on Settlement Calculations

For the classical layerwise summation method, the embankment behaves like a sample tested in an oedometer condition, which neglects lateral displacements. The effect of accuracy of the vertical superimposed stress (σ_s') on settlements is evaluated as shown in Figure 14, but the predicted settlements with the modified method still slightly smaller than the field measurements, as shown in Figure 10. This discrepancy would be induced by the neglect of lateral displacements, the effect of strain rate, the sample disturbance, and the stress paths on compression behavior [1].

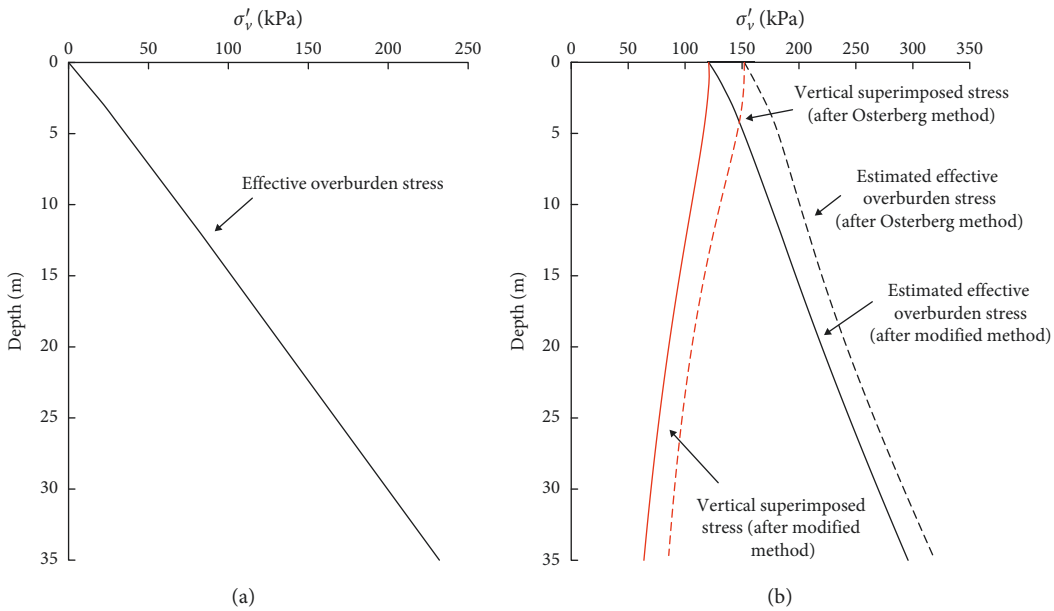

Figure 12: The typical effective stress distribution with depth for two boreholes: (a) ground; (b) embankment.

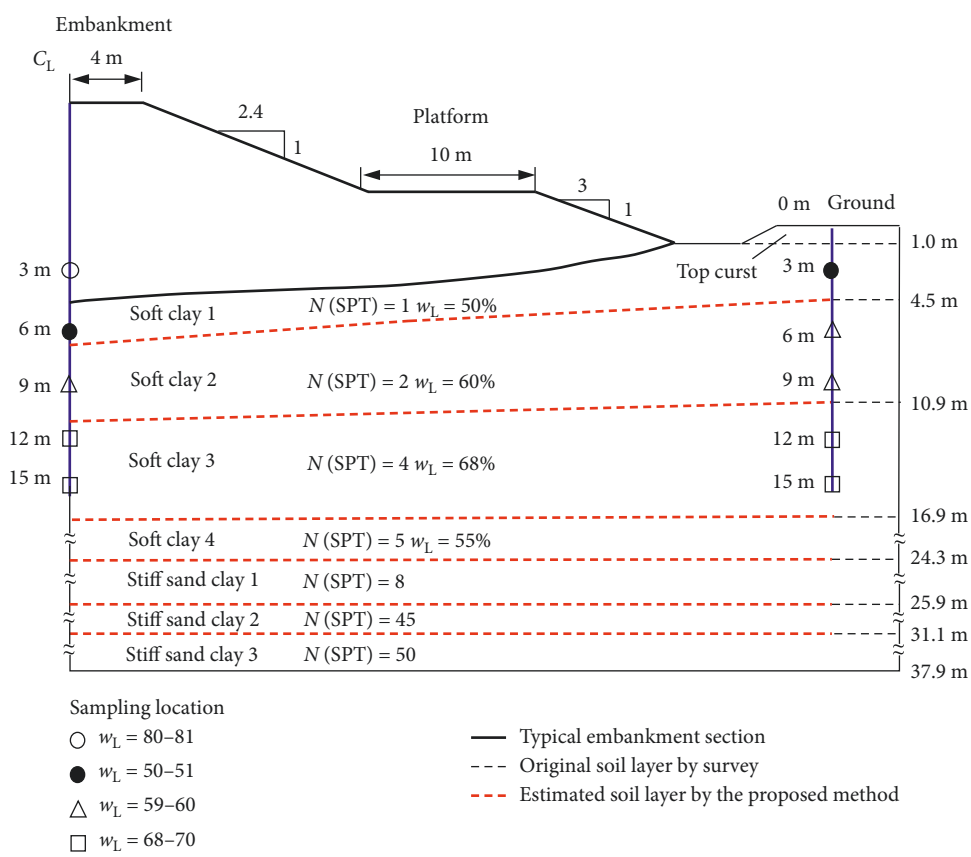

Figure 13: Soil stratification by the proposed method.

In practice, the field consolidation strains are generally higher than the values obtained from conventional consolidation tests, and the rate effects are associated with the $C_c/(1+e_0)$ [50]. Leroueil et al. [1] reported that the effect of strain rate is small for the clay with low $C_c/(1+e_0)$ values between the 0.2–0.25. The values of $C_c/(1+e_0)$ for investigated natural Funing soft clay in Table 2 (vary from 0.19 to 0.28) reveals a minor effect of strain rate in this study.

On the contrary, the natural Funing clay has a yield stress ratio greater than 1.0 (Figure 9) that attributed to the

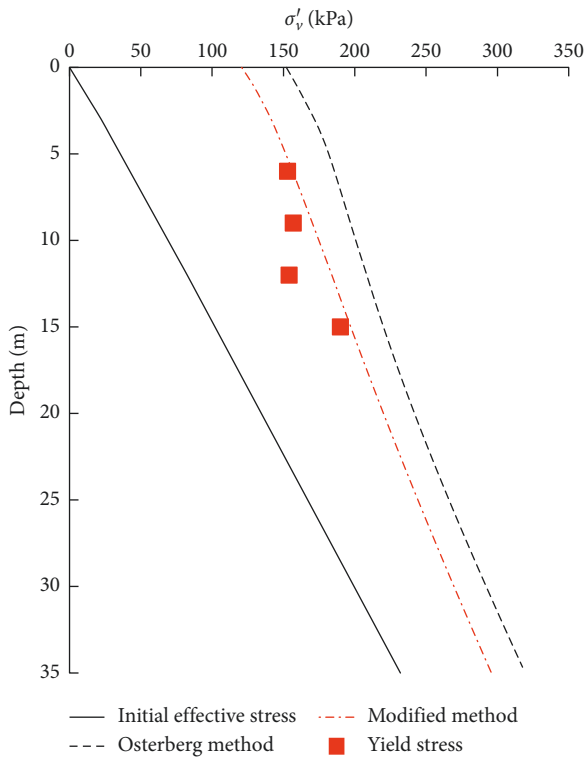

Figure 14: Comparisons between the calculated effective stress and the yield stress.

development of the resistance of soil structure during the depositional and the postdepositional processes. The soil structure of natural clay can be easily disturbed during sampling and handing. The sample disturbance reduces the consolidation yield stress and the compressibility of soil in the postyield state that results in the underestimation of the settlements [1, 13].

The following two factors may be responsible for the discrepancy on settlements using traditional method: the neglect of lateral displacements and the effect of consolidation stress path on mechanical behaviors of natural clays. In practice, the neglect of lateral displacements resulted in an underestimation of the settlements induced by the embankment load and the compression parameters calibrated from oedometer tests result in overestimation of settlements [1, 16, 20]. As shown in Figure 10, the former factor has a much significant influence on the calculation of settlement than the latter factor in this work.

7. Conclusions

This study performs a case study on the settlement behavior of soft natural clays induced by a 14-year embankment load. The predicted settlements obtained by different methods are compared with field observations. The main conclusions are summarized as follows:

(1) The settlement predicted by combining layerwise summation method of settlement calculation model, the modified method of determining vertical superimposed stress suggested by Wang et al. [30], and compression parameters measured from undisturbed samples obtained by a thin-wall free-piston sampler is in agreement with the field observation

(2) The difference in predicted settlement between traditional method and the modified approach can be attributed to the overestimated vertical superimposed stress in subsoil determined by the Osterberg method

(3) The agreement in settlement between predicted value and field observation is a result from the balance among the errors in calculation model, vertical superimposed stress, and compression parameters

Notations

C_c: Compression index at postyield state of natural clay
C_r: Compression index at preyield state of natural clay
e_0: Initial void ratio
e_1: Void ratio after consolidation
G_s: Density of soil particles
H: Thickness of the soil layer
S: Settlement
w_n: Natural water content
w_L: Liquid limit
w_p: Plastic limit
σ_b: Contact stress along the base
σ'_p: Yield stress
σ'_s: Vertical superimposed stress caused by embankment load
σ'_{v0}: Effective overburden stress
$\Delta\varepsilon_v$: Volumetric strain at the effective overburden stress
γ': Unit weight of soil.

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 they have no conflicts of interest.

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