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Settlement prediction method for staged preloading on soft clay ground

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Abstract: To accurately predict the settlement of soft soil foundations under graded surcharge preloading, the Asaoka method was improved. Considering the change in the soil consolidation coefficient with load, the slope of the settlement prediction line was modified, and a prediction calculation method for the prediction line intercept was used. The improved Asaoka method considered the influence of consolidation stress on the consolidation coefficients and nonlinearity of soil consolidation. The reliability of the improved method was then verified through indoor consolidation tests and field observation. The results demonstrate that the consolidation coefficient C_v is not a constant, and it satisfies the relationship with the consolidation pressure P: $C_v = a\ln(b\ln P)$, which has an error of less than 5%. The settlement prediction lines at various stress levels are not parallel, and the improved method exhibites a lower error rate than the original Asaoka method. The improved Asaoka method offers higher prediction accuracy than the traditional method and can reliably predict the settlement of soft soil foundations during graded surcharge preloading.

Key words: Asaoka method; settlement prediction; staged preloading; consolidation coefficient

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Soft clays have high water content, poor permeability, high compressibility, and low strength[1−3]. When engineering construction such as highways, railroads, docks, and yards are built on this soft ground, ground failure caused by the low bearing capacity[4] and large settlements due to the high compressibility[5−6] easily occur. To maintain engineering safety and prevent disaster, ground improvement methods, such as the replacement method, dynamic consolidation, reinforcement method, and drainage consolidation method, are usually employed[7−8]. Among them, the preloading method with prefabricated vertical drainage (PVD) is a simple, economical, and effective method for soft ground improvement[9]. The pore water in the soil is discharged along the PVDs under the overburden load, leading to ground consolidation and settlement. However, to ensure stability during preloading, the overburden load is applied stage by stage according to the ground strength[10]. Thus, an accurate prediction method of the ground settlement is necessary for construction.

Currently, the settlement prediction mainly includes the curve fitting method based on field observation and the theoretical calculation method based on the Terzaghi consolidation theory[11−12]. For the theoretical calculation method, Terzaghi's theory assumes that the consolidation coefficient is constant[13]. However, the consolidation coefficient influenced by various factors such as soil structure, permeability, and stress history[14−16] can change significantly, resulting in a large settlement deviation between the theoretical calculation and in situ observation. Examples of the curve fitting methods include the hyperbolic method, exponential method, three-point method, etc.[17−19]. These methods use the observed data after the i-th loading stage to deduct the fitting parameters and predict the settlement tendency at the $(i+1)$-th loading stage. Therefore, the accuracy of prediction depends on the curve fitting efficiency and the observed settlement. If the settlement under a loading stage is small, the reliability of the back-analyzed parameters results in error accumulation or transmission in the settlement prediction of the next stage[20]. Among these methods, the Asaoka method is considered more reliable for predicting the final settlement based on shorter-term observation data. Liu et al.[21] compared settlement prediction models and found that the Asaoka method has a smaller error. It is widely used in overseas projects, as recommended by the Hong Kong *Port Works Design Manual*[22]. With the Asaoka method, a comprehensive dynamic design process for treating soft ground can be established. This process involves projecting the final settlement, consolidation degree, consolidation coefficient, and compression index under preloading and predicting the preloading time and unloading timing. Such detailed design facilitates the accumulation of engineering experience.

Asaoka[23] proposed a settlement prediction method that has been used extensively in engineering to guide preloading. The method suggests that for loading, the fitted linear line for the i-th loading stage in the S_j-S_{j-1} coordinates

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is upward shifted for the $(i+1)$-th loading stage. With this hypothesis, the settlement tendency at the $(i+1)$-th loading stage can be predicted when the intercept of the prediction line is known. However, the method does not provide the steps on how to determine the offset distance from the fitting line at the i-th stage to the $(i+1)$-th stage or consider the change of the consolidation coefficient, which affects the accuracy of prediction.

For staged preloading on soft clay ground, the consolidation coefficient changes with the overburden load and consolidation degree. The fitted line at the $(i+1)$-th loading stage in S_j-S_{j-1} coordinates is no longer parallel to that at the i-th stage. Accordingly, to investigate the variation in the consolidation coefficients under staged preloading and determine the slope and offset of the fitted line of the further stage, a literature review and laboratory tests were initially conducted. An improved Asaoka method suitable for staged preloading was proposed, and reliability was then verified by the oedometer test and in situ observation.

1 Asaoka Method

Asaoka[23] introduced the differential equation $\dot{\varepsilon} = C_v \varepsilon_{zz}$ from Mikasa's one-dimensional consolidation theory, and soft ground settlement can be expressed as

$$S_t = \int_0^H \varepsilon(t, z)\,\mathrm{d}z \tag{1}$$

where S_t is the settlement at time t; ε is the vertical strain; C_v is the vertical consolidation coefficient; z is the soil height; H is the drainage distance. An integral simplification of Eq. (1) was reorganized as follows[23]:

$$S + c_1\dot{S} + c_2\ddot{S} + \cdots + c_n \overset{n}{\ddot{S}} = C \tag{2}$$

where S is the total settlement (including immediate, primary, and secondary consolidation settlement). The parameters $(c_1, c_2, \ldots, c_n, C)$, depending on the consolidation and boundary condition, can be back-analyzed with the existing settlement series to predict the future settlement tendency. Eq. (2) can be simplified as the following differential expression[23]:

$$S_j = \beta_0 + \sum_{s=1}^{n} \beta_s S_{j-s} \tag{3}$$

where S_j is the settlement at the time t_j with a constant time interval $\Delta t = t_j - t_{j-1}$; S_{j-1} is the settlement at time t_{j-1}; β_0 and β_s are parameters. For calculation convenience, Eqs. (2) and (3) can be simplified with the first-order expression as follows[23]:

$$\left.\begin{aligned} &S + c_1\dot{S} = C \\ &S_j = \beta_0 + \beta_1 S_{j-1} \end{aligned}\right\} \tag{4}$$

Under the S_j-S_{j-1} frame, β_0 and β_1 are the intercept and slope of the fitting line, respectively. When t tends to ∞, S_j should be equal to S_{j-1} and S_f (final settlement). The method has realized a graphical solution based on[24]

$$\ln\beta_1 = \begin{cases} -\dfrac{6\,C_v}{H^2}\Delta t & \text{Double drainage} \\ -\dfrac{2\,C_v}{H^2}\Delta t & \text{Upward drainage} \end{cases} \tag{5}$$

$$S_f = \frac{\beta_0}{1-\beta_1} \tag{6}$$

The Asaoka method is performed as follows:

① Select the settlement series $(S_1, S_2, \ldots, S_j)$ from the settlement curve.

② Plot the data series (S_{j-1}, S_j) in the frame of S_{j-1} and S_j and a linear line with a slope of 1.0.

③ Obtain the best-fit linear line with the data series. When it intersects with the straight line with a slope of 1.0, the final settlement at an overburden preloading can be achieved.

Fig. 1 shows the application diagram of the Asaoka method in the S_j-S_{j-1} frame, where L_i, S_{fi}, β_{1i}, and β_{0i} indicate the prediction line, final settlement, slope, and intercept at the i-th preloading stage, respectively.

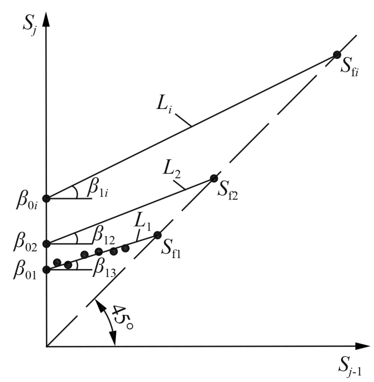

Fig. 1 Sketch of the Asaoka method

2 Improved Asaoka Method

2.1 β_1 mobilization

The relationship between the slope β_1 of the fitting line and the coefficient C_v is established in Eq. (5). The constant slope β_1 was usually assumed during the preloading stage. However, many studies have shown that the C_v is strongly affected by the stress history and consolidation degree[25–26]. To confirm this tendency, a staged oedometer test on Zhuhai marine clay was referred to, whose physical parameters are presented in Table 1. The consolidation coefficient was calculated using Eq. (7)[27].

Table 1 Physical properties of the soil specimen

Natural unit weight/(kN·m^{-3})	Moisture mass fraction/%	Plastic limit/%	Liquid limit/%	Plasticity index	Liquid index	Clay mass fraction/%	Void ratio e_0
16.95	37.4	22.44	42.40	19.59	0.77	70.7	1.19

$$C_v = 0.848\frac{H^2}{t_{90}} \tag{7}$$

In this study, an inverse analysis of the consolidation test results was conducted using Terzaghi's consolidation theory solution. This study also provided an analytical solution for Terzaghi's consolidation equation[13]:

$$U = 1 - \frac{8}{\pi^2}\sum_{m=0}^{\infty}\frac{1}{(2m+1)^2}\exp\left[-(2m+1)^2\frac{\pi^2}{4}\frac{C_v}{H^2}t\right] \tag{8}$$

where U is the degree of consolidation. According to

Ref. [28], taking the first term of the infinite series satisfies the accuracy requirement:

$$U = 1 - \frac{8}{\pi^2}\exp\left(-\frac{\pi^2}{4}\frac{C_v}{H^2}t\right) \tag{9}$$

$$C_v = \frac{4H^2}{\pi^2 t}\ln\left[\frac{8}{\pi^2(1-U)}\right] \tag{10}$$

Referring to the inverse calculation of C_v and combining the results of Table 2 and Fig. 2, and the relationship between C_v and effective vertical stress P can be expressed as follows:

$$C_v = a\ln(b\ln P) \tag{11}$$

where a and b are the fitting parameters related to C_v and P.

Table 2 Eq. (11) fitting results

Fitting equation	a	b	Standard error a	Standard error b	R^2	Data source
Jiangmen soft clay	−1.02	0.11	0.08	0.01	0.97	Ref. [29]
Soft clay-Tien Giang	−1.38	0.15	0.26	0.02	0.84	Ref. [15]
Soft clay with organic matter-Kien Giang	−0.61	0.15	0.09	0.01	0.90	Ref. [15]
Soft sandy clay-Ha Tinh	−0.79	0.04	0.06	0.01	0.97	Ref. [15]
Ireland clay	−0.60	0.20	0.15	0.02	0.78	Ref. [15]
Singapore marine clay	−14.28	0.15	1.90	0.01	0.92	Ref. [30]
Kunming peat soil	−2.64	0.17	0.38	0.01	0.92	Ref. [31]
Shenzhen clay	−5.77	0.17	2.17	0.02	0.60	Ref. [32]
Ballina clay	−2.65	0.18	0.67	0.01	0.69	Ref. [33]
Gemas clay	−18.32	0.09	2.37	0.01	0.93	Ref. [34]
Kluang clay	−11.01	0.11	1.41	0.01	0.92	Ref. [34]
Burswood clay	−3.25	0.15	0.33	0.01	0.95	Ref. [35]
Pusan clay	−10.74	0.17	2.35	0.01	0.83	Ref. [36]
Ningbo soft clay	−1.55	0.13	0.19	0.01	0.94	Ref. [37]

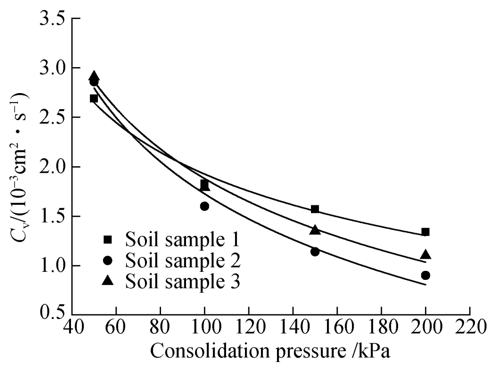

Fig. 2 P-dependent C_v for soft marine clay in Zhuhai

Fig. 2 shows that C_v decreases with P, and its relationship can satisfy Eq. (11) with a higher variance factor (R^2) of approximately 0.98. Fig. 3 is the error analysis between the tested and calculated results by Eq. (11), where the relative errors range from 2.65% to 3.54%. To further confirm the applicability of Eq. (11), literature data[15, 23–32] were adopted. Table 2 presents a summary of the fitted parameters, where most have good confidence besides Shenzhen and Ballina clay with R^2 of 0.60 and 0.68, respectively. The error analysis in Fig. 4 also confirmed this conclusion.

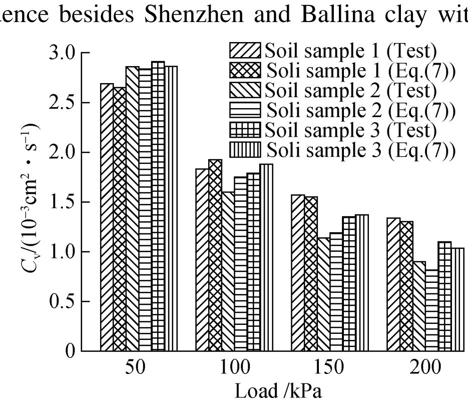

Fig. 3 C_v's error analysis by Eq. (11) for soft marine clay in Zhuhai

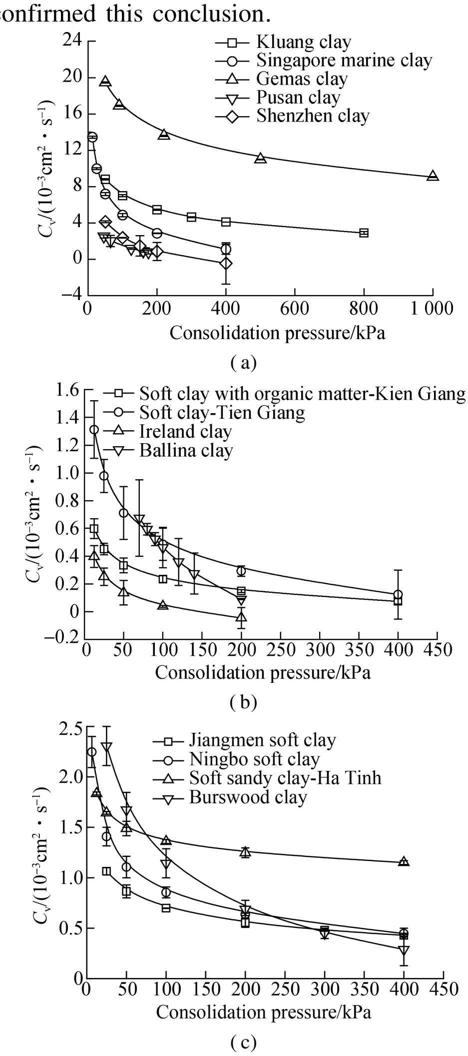

Fig. 4 C_v's error analysis by Eq. (11) of the other clays. (a) Soft clay 1; (b) Soft clay 2; (c) Soft clay 3

Eqs. (12) and (13) give the relationship between β_{1i} and C_{vi} of the upward drainage as an example[24]:

$$\ln \beta_{1i} = -\frac{2C_{vi}}{H^2}\Delta t \tag{12}$$

$$C_{vi} = -\frac{H^2}{2\Delta t}\ln \beta_{1i} \tag{13}$$

When Eqs. (11) and (13) are combined, Eq. (14) can be concluded as follows:

$$C_{vi} = -\frac{H^2}{2\Delta t}\ln \beta_{1i} = a\ln(b\ln P) \tag{14}$$

When $i = 1$, $\beta_{11} = 0.8$, so

$$\beta_{1i} = \begin{cases} 0.8 & i = 1 \\ b_i \ln P_i + 0.8 & i \geqslant 2 \end{cases} \tag{15}$$

Eq. (15) indicates that β_{1i} increases with the overburden load P_i.

2.2 Intercept offset β_0

In the S_{j-1}-S_j coordinate system of the Asaoka method, Liu and Jing[24] defined β_0 as the initial instantaneous settlement S_0 ($S_0 = \beta_0$) of the ground. By combining this with Eq. (16), they could back-calculate the undrained modulus of elasticity of the embankment under the first level of loading. The instantaneous settlement induced by the load increment of the next level was then computed as the fitting distance between the straight line and the fitting line under the first level of loading. This translation distance, along with the slope, was used to identify the corresponding straight line for the next level of loading, ultimately leading to the derivation of the final settlement. The behavior of the next-stage construction can be predicted with the β_0 and β_1 from the last-stage construction; the more the previous stages with settlement measurement, the higher the accuracy of the next-stage prediction.

$$E_u = \frac{(1 - v^2)qBI}{S_0} \tag{16}$$

where E_u is the undrained modulus; q is the overburden preloading; B is the dimension of the load area; I is a shape; v is Poisson's ratio. For the staged preloading, β_{0i} can no longer be simplified as the initial immediate settlement S_0; however, it has two components: the immediate settlement S_{di} under the i-th stage and the consolidation settlement S_{ci-1} at the $(i-1)$-th stage. β_{0i} is the intercept offset at the i-th stage.

$$\beta_{0i} = S_{di} + S_{ci-1} \tag{17}$$

$$S_{ci-1} = U_{ti-1}(S_{fi-1} - S_{di}) \tag{18}$$

where S_{fi-1} is the final settlement at the $(i-1)$-th stage, and U_{ti-1} is the average consolidation degree at the $(i-1)$-th stage, which can be calculated by

$$U_t = 1 - \frac{8}{\pi^2}\sum_{m=1}^{\infty}\frac{1}{m^2}\exp\left(-\frac{m^2\pi^2}{4}T_v\right) \tag{19}$$

For engineering practices, Eq. (19) can be simplified to Eq. (20) to satisfy the precision, as shown below:

$$U_t = 1 - \frac{8}{\pi^2}\exp\left(-\frac{\pi^2}{4}T_v\right) \tag{20}$$

If Eq. (14) is substituted into Eq. (20), U_{ti-1} can be calculated as follows:

$$U_{ti-1} = \begin{cases} 1 - \dfrac{8}{\pi^2}(\beta_{1i-1})^{\frac{n}{4}} & \text{Double drainage} \\ 1 - \dfrac{8}{\pi^2}(\beta_{1i-1})^{n} & \text{Upward drainage} \end{cases} \tag{21}$$

where

$$k = \frac{t - t_{i-1}}{\Delta t}$$

By substituting Eq. (21) into Eq. (18), the intercept β_{0i} at the i-th stage can be obtained.

3 Reliability Verification of the Improved Asaoka Method

To verify the reliability of the proposed method, laboratory oedometer data and field observation were used.

3.1 Oedometer tests

The oedometer test was performed using a Zhuhai marine soft soil, where an in situ specimen was sampled at approximately 8.0 m below the ground using a thin-walled extractor. It has a natural weight γ_w of 17.7 kN/m^3, moisture mass fraction of 54.64%, compression modulus of 1.65 MPa, and consolidation coefficient of 0.77×10^{-3} cm^2/s. The loading procedure (0→25→50→100→200 kPa) was performed. Each stage was loaded continuously for 24 h, and the time-dependent settlement is shown in Fig. 5. A series of settlements (S_1, S_2, …, S_j) were selected with a time interval Δt equal to 1 h. The data series is plotted in the S_{j-1} and S_j analysis frame, and β_0 and β_1 were acquired.

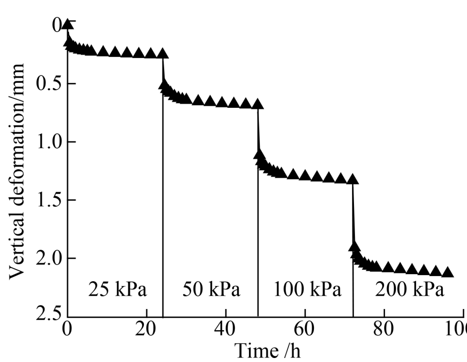

Fig. 5 Time-dependent settlement under staged preloading

Table 3 presents the predicted error by the modified Asaoka method when compared with the observed values. At the first stage (P = 25 kPa), β_1 is equal to 0.661, and b is 0.04. With Eq. (15), β_i at P of 50, 100, and 200 kPa can be forecasted, and the results suggest that the improved method works well. On the contrary, for the constant β_1 in the traditional Asaoka method, the prediction error becomes larger and larger.

Table 3 Error analysis of β_0 and β_1 by the improved and traditional methods with the oedometer test ($\Delta t = 1$ h)

β_i	Load/kPa	Observation fitting	Improved method	r_1/%	Traditional method	r_2/%
β_1	25	0.66	0.67	1.55	0.66	0
	50	0.64	0.64	0.08	0.66	2.80
	100	0.61	0.62	1.12	0.66	8.54
	200	0.59	0.59	0.18	0.66	12.61
β_0	25	0.16	0.16	0	0.16	0
	50	0.30	0.30	−0.23	0.32	6.42
	100	0.57	0.51	−10.49	0.64	11.89
	200	1.06	0.79	−26.03	1.28	20.61

Note: r_1 is the error between the observed value and the improved value; r_2 is the error between the observed value and the traditional value.

Liu and Jing[24] did not consider the nonlinearity of the soil modulus and proposed that β_0, equal to the initial immediate settlement S_d, increased linearly with the overburden load. However, Table 3 indicates that β_{0i} increases gradually with the staged overburden load. Fig. 6 presents a comparison of the final settlement, indicating that the predicted settlement by the improved Asaoka method with a changeable β_0 is much closer to the observation.

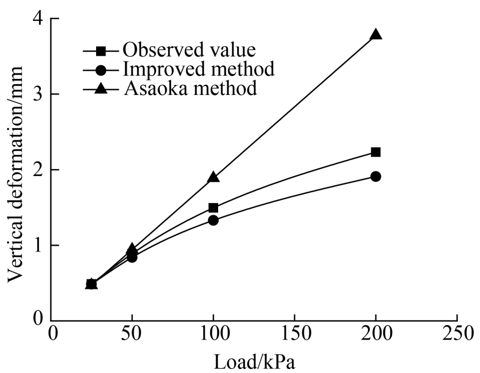

Fig. 6 Comparison between the predicted settlement and laboratory observation

3.2 Field settlement monitoring

3.2.1 Case 1

To further verify the reliability of the modified Asaoka method, field monitoring data obtained by Liu and Jing[24] were used for the analysis. The Xulian Expressway crosses a marine sedimentary plain with a layer of marine soft clay with a thickness of approximately 5 to 13 m. The bottom width of the embankment is 40 m, and its height is 3 to 7 m. To control the post-construction settlement, the two-stage preloading was adopted. The construction(backfilling) curve is shown in Fig. 7, where the monitoring data at two sections (K21 + 130 and K21 + 583) were also supplied. The improved and traditional Asaoka methods were adopted to predict the embankment settlement. The results are shown in Fig. 8 and Table 4.

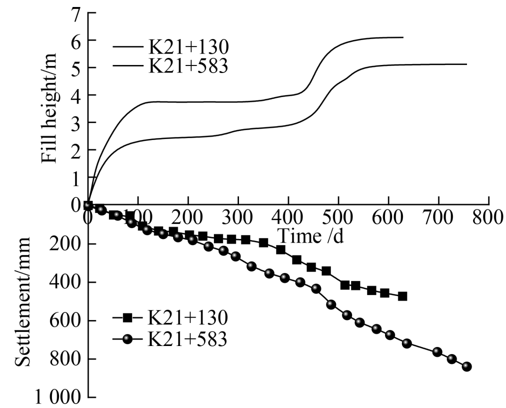

Fig. 7 Time-dependent fill height and settlement (case 1)

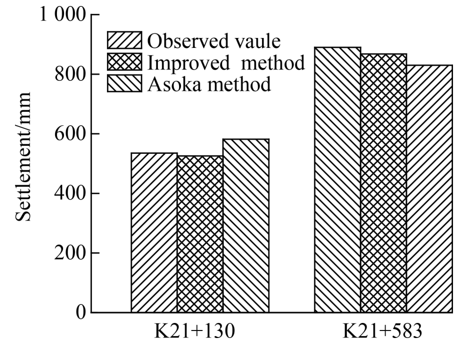

Fig. 8 Comparison between the predicted settlement and in situ observation (case 1)

Table 4 Error analysis of β_{0i} and β_{1i} by the improved and traditional methods (case 1)

Section	β_i	Height/m	Observation fitting	Improved method	r_1/%	Traditional method	r_2/%
K21 +130	β_{1i}	3.740	0.890	0.890	0	0.890	0
		6.080	0.884	0.880	−0.452	0.890	0.679
	β_{0i}	3.740	37.815	38.000	0.489	38.000	0.489
		6.080	63.500	61.500	−0.966	64.000	3.060
K21 +583	β_{1i}	2.500	0.900	0.900	3.388	0.900	3.388
		5.160	0.923	0.919	−0.433	0.900	−2.492
	β_{0i}	2.500	36.150	37.000	2.353	37.000	2.353
		5.160	68.530	70.300	2.583	83.000	21.115

Note: The time interval of field observation settlement is 1 h ($\Delta t = 1$ h).

Table 4 confirms that β_1 varies with increasing fill height. When a constant β_1 was used, errors of 0.679% and 2.452% were noted from the observed values. After the first stage of filling, the β_1 calculated by the improved and traditional Asaoka methods are updated to adapt the observed values, and β_1 calculated using the Asaoka method has a smaller error than that obtained by Eq. (11). β_0 also increased with the fill height. As shown in Table 4, the β_0 calculated by the improved Asaoka method before and after the first stage has a small error relative to the observed value. The above analysis shows that both β_0 and β_1 affect the accuracy of settlement prediction. Fig. 8 shows the comparison between the predicted final settlement and the observed values by the improved and traditional Asoka methods, indicating errors of 1.78% and 8.69% relative to the observed value at the K21 + 130 section and errors of 2.43% and

6.72% at the K21 + 583 section.

3.2.2 Case 2

The reliability of the improved Asaoka method was verified by Wang's case[38]. The embankment had a width of 26.0 m at the top surface, a slope ratio of 1 : 1.5, and a height of 5.5 m. For the ground, the gray-yellow silty clay was deposited at a depth of 0-6.5 m, which has a unit weight of 19.8 kN/m^3 and a compression coefficient of 0.21 MPa^{-1}. Under the top soil layer, the gray-brown silty clay deposited from 6.5 to 22.8 m has a unit weight of 18.9 kN/m^3 and a compression coefficient of 0.16 MPa^{-1}. Ground treatment was carried out by staged preloading. It was backfilled by five stages, with the materials having a unit weight of 19.0 kN/m^3 and a compression coefficient of 0.36 MPa^{-1}. Fig. 9 shows the time-dependent fill height and ground settlement curves.

Table 5 indicates that β_1 increases with the fill height by direct fitting and calculated by the modified method. However, the results obtained by the traditional method remain constant, and the error with the observed value became increasingly large with increasing height, indicating the higher accuracy and applicability of the improved method for the β_1 calculation. β_0 both increases with increasing height, and both methods have similar errors with the observed values. Fig. 10 shows the observed

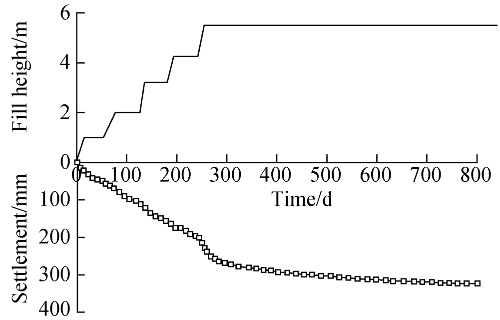

Fig. 9 Time-dependent fill height and settlement (case 2)

and predicted final settlements of each stage, which overall shows a good agreement between the observed and predicted settlements. A large error between the observed and predicted settlements at the stages 2, 3, and 4 by the improved method had a large error, possibly because of the short-term preloading period and continuous consolidation development. However, after the stage 5, which lasted for approximately 600 d, the final observed and predicted settlements by the improved method were very consistent. Totally, Fig. 10 also indicates that the settlement predicted by the traditional method is smaller than that by the improved method because the traditional method only considers the immediate settlement of the next stage when calculating β_0, where the consolidation settlement was not involved.

Table 5 Error analysis of β_0 and β_1 by the improved and traditional Asaoka methods (case 2)

β_i	Backfilling stage	Height/m	Observation fitting	Improved method	r_1/%	Traditional method	r_2/%
β_{1i}	1	1.000	0.838	0.838	0	0.838	0
	2	2.000	0.843	0.847	0.545	0.838	-0.510
	3	3.215	0.848	0.853	0.596	0.838	-1.179
	4	4.257	0.872	0.857	-1.651	0.838	-3.798
	5	5.057	0.877	0.860	-1.842	0.838	-4.358
β_{0i}	1	1.000	9.688	9.688	0	9.688	0
	2	2.000	23.238	22.609	-2.707	19.377	-16.617
	3	3.215	28.979	36.738	26.773	31.143	7.468
	4	4.257	30.256	41.661	37.696	41.239	36.301
	5	5.057	40.998	45.882	11.913	53.354	30.139

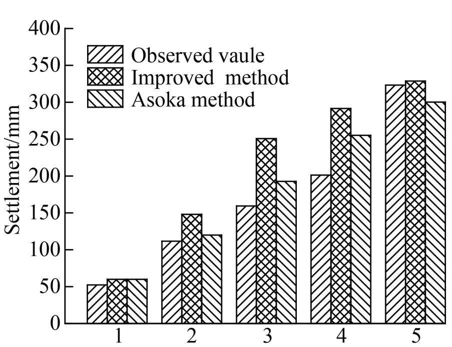

Fig. 10 Comparison between the predicted settlement and in situ observation (case 2)

4 Conclusions

1) The consolidation coefficient is not a constant; it satisfies the relationship with the consolidation pressure: $C_v = a\ln(b\ln P)$, which is a good fit with a correlation coefficient of 0.98 and an error of less than 5%.

2) The improved Asaoka method integrates the variation of the consolidation coefficient with consolidation stress, considering the nonlinearity of soil consolidation. In this method, β_1 is no longer a constant, whereas the fitted straight lines for each level of loading in the S_j-S_{j-1} coordinate system are no longer parallel. The calculated β_1 using the improved Asaoka method provides a closer approximation to the actual situation.

3) In multi-stage loading, the intercept β_{0i} in the S_j-S_{j-1} coordinate system is equal to the sum of the instantaneous settlement at the current load level and the consolidation settlement at the previous load level.

4) The accuracy of the improved Asaoka method was validated through one-dimensional consolidation tests and in situ settlement monitoring data. The error of the im-

proved method was lower than that of the Asaoka method, offering a more precise and practical approach for predicting the settlement of soft ground using stage surcharge preloading.

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软土地基分级堆载预压沉降预测方法

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摘要:为了准确预测分级堆载预压条件下软土地基的沉降,对 Asaoka 法进行了改进.考虑土体固结系数随荷载的变化,对沉降预测线的斜率进行了修正,并给出了预测线截距的预测计算方法.改进的 Asaoka 方法考虑了固结应力对固结系数的影响和土体固结的非线性.通过室内固结试验和现场监测,验证了改进方法的可靠性.结果表明,固结系数 C_v 不是常数,它满足与固结压力 P 的关系 $C_v = a\ln(b\ln P)$,拟合良好,误差小于5%.不同应力水平下的沉降预测线并不平行,与原始的 Asaoka 方法相比,改进方法误差率较低.改进的 Asaoka 方法比传统方法具有更高的预测精度,能够可靠地预测分级堆载预压过程中软土地基的沉降.

关键词: Asaoka 法; 沉降预测; 分级堆载; 固结系数

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