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ORIGINAL PAPER

Finite Element Analysis of Vacuum Consolidation With Modified Compressibility and Permeability Parameters

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Abstract Laboratory tests and case history studies indicate that soil subjected to vacuum preloading may not behave the same as ground treated by traditional surcharge preloading. In detail, soil compression under vacuum pressure is smaller than or equal to that induced by positive pressure with the same magnitude; soil rebound after stopping the vacuum is not as high as after removing the surcharge; and the consolidation rate is usually faster under vacuum pressure than with surcharge preloading. Analysis of vacuum consolidation with existing methods cannot gain all these differences. Thus, in this study, three factors for adjusting compressibility and permeability are proposed based on past laboratory and field results which are used in a finite element analysis of soft soil foundation under vacuum-assisted preloading. This proposed method can be incorporated in existing computer programs associated with classical soil models (e.g., the modified Cam-Clay model and the Soft-Soil model); it is then examined via three distinct simulation scenarios including a laboratory model test and two prototype field cases. The improved accuracy in relation to consolidation by the proposed method is demonstrated and practical ranges for the adjustment factors are discussed.

Introduction

Vacuum preloading via prefabricated vertical drains is an effective ground improvement technique and has been widely used [1]. Developed from the traditional fill surcharge method, vacuum preloading method usually is being compared to the surcharge preloading by both practitioners and researchers [2–4]. Saowapakpiboon et al. [5, 6] carried out two model tests to compare the consolidation of a prefabricated vertical drain (PVD) system under surcharge preloading and vacuum pressure at a macro treatment scale and found that settlement under vacuum pressure and surcharge load is almost the same but the former leads to faster consolidation. Rondonuwu et al. [7] carried out small size tests on PVD systems using a triaxial apparatus with confinement conditions for surcharge preloading and vacuum pressure preloading and found that different confinement conditions result in different consolidation processes. Indraratna et al. [8] compared a vacuum system with surcharge loading to a traditional surcharge fill preloading at the Port of Brisbane, Australia, and showed that vacuum combined preloading increases consolidation faster than surcharge preloading alone.

Although vacuum-assisted preloading is a special process, not many theoretical methods have been proposed to simulate its final settlement and consolidation. Imai [9] considered the stress state and pattern of deformation of a soil element under vacuum pressure and proposed a method to calculate final settlement based on the theory of elasticity. Chai et al. [10] indicated that Imai's method ignores the

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possibility of tension cracks, and also the assumption of elastic theory when determining compression could lead to errors. They proposed a method where three zones within the ground and a tension cracked zone close to the ground surface could be differentiated. Robinson et al. [11] also subdivided soft ground into three zones to simulate vacuum consolidation and carried out laboratory tests on reconstituted samples under different states of stress to illustrate the mechanisms of compression under vacuum pressure. Wu et al. [12] extended Chai's method to calculate the settlement process during vacuum preloading and after releasing the vacuum. However, this simplified method has various approximations inherited from classical radial drainage theory; apart from which, the micro-structural behaviour of clays under isotropic stress (e.g. in vacuum consolidation) and anisotropic stress conditions (e.g. in surchage preloading) are different [13–15]. Such a difference in its microstructural behaviour may lead to a different value of soil permeability under vacuum pressure or surchage preloading. There is no current method whereby this difference in the numerical simulation of vacuum consolidation can be captured.

Numerical tools such as the finite element method (FEM) are widely used to simulate the consolidation of a ground with vertical drains [4, 16]. In FEM, classical constitutive models (e.g., Cam-clay type model and Soft Soil model) whose parameters can be obtained from well-established laboratory tests (e.g., oedometer test and triaxial test) are widely used. However, the special stress state of vacuum preloading means the consolidation of a soil element may differ from that observed in traditional laboratory tests carried out under compression. To address these gaps in knowledge, a novel method consisting of three adjustment factors is proposed here to capture the correct compression and permeability of soil subject to vacuum pressure, and the finite element program is used to validate this proposed method on the basis of simulations of a laboratory model test and two prototype field cases.

Theory and Background

Stress State of Ground Under Vacuum Pressure

Vacuum consolidation contains the use of atmospheric pressure which produces a load onto ground to be consolidated. The installation of vertical drains facilitates the suction pressure propagating into the deep soil layers which imparts an increased hydraulic gradient towards the drain which in turn creates a particular stress state in the ground.

In the depth direction, three zones can be introduced after applying a vacuum pressure [10, 11], as shown in Fig. 1. Zone I ($0 \leq z < z_c$) is the tension crack zone which is

close to the ground surface, and where the soil deforms vertically and laterally (inward compression). The confining stress from the surrounding soil is negligible, which results in an ideal isotropic effective stress state in this zone. Based on the earth pressure theory, the depth of the tension zone (z_c) can be determined [10], i.e.,

$$z_c = \frac{2c'}{\gamma t} \sqrt{k_a}, \quad for \ z_c \leq z_W \tag{1a}$$

$$z_c = \frac{1}{(\gamma_t - \gamma_w)} \left(\frac{2c'}{\sqrt{k_a}} - \gamma_w z_w \right) \quad for \ z_c z_w \tag{1b}$$

where γ_t is total unit weight of soil, γ_w is the unit weight of pore water, c' and ϕ' are the effective stress cohesion and friction angle of the soil, respectively, $k_a = \tan^2(45 - \phi'/2)$ is the active earth pressure coefficient, and z_w is the depth of the groundwater level below the ground surface.

The tension crack zone overlies an active condition zone (Zone II, $z_c \leq z < z_l$) where the confining stress consists of vacuum pressure and earth pressure from the surrounding soil. The determination of depth z_l, which denotes a threshold depth where lateral displacement occurs, is presented in the next section. Due to the inward lateral displacement induced by vacuum pressure, the earth pressure in Zone II is less than the earth pressure at-rest. The lateral inward compression at Zone II decreases with depth and the confining stress changes from an active state (K_a condition) to at-rest state (K_0 condition). Below the active condition zone, i.e., $z \geq z_l$, is the at-rest zone (Zone III) where inward movement can be negligible and any stress in the soil may remain in a K_0 condition.

Stress State of Soil Elements Under Vacuum Pressure

Due to the inward movement induced by vacuum pressure, the confining condition of the soil elements and stress state are different from that under surchage loading. In fact, according to existing researches [2, 11], the stress state of a soil element under surchage loading is mainly governed by the outward horizontal compression, which corresponds to $\varepsilon_h \leq 0$, and where ε_h is the horizontal strain of the soil element. For a vacuum pressure condition however, the value of ε_h is positive. The stress state for surchage loading is close to the failure line (K_f line), but it is far from the failure line for vacuum preloading, even under pressure changes of the same magnitude. The final vertical effective stress is the same under the same magnitude of positive or negative pressure, i.e., $\sigma'_{1,sur} = \sigma'_{1,vac}$, but the final horizontal effective stresses ($\sigma'_{3,sur}$ or $\sigma'_{3,vac}$) are quite different. Note that $\sigma'_{1,sur}$ ($\sigma'_{3,sur}$) and $\sigma'_{1,vac}$ ($\sigma'_{3,vac}$) are the maximum (minimum) effective stress under surchage load and vacuum
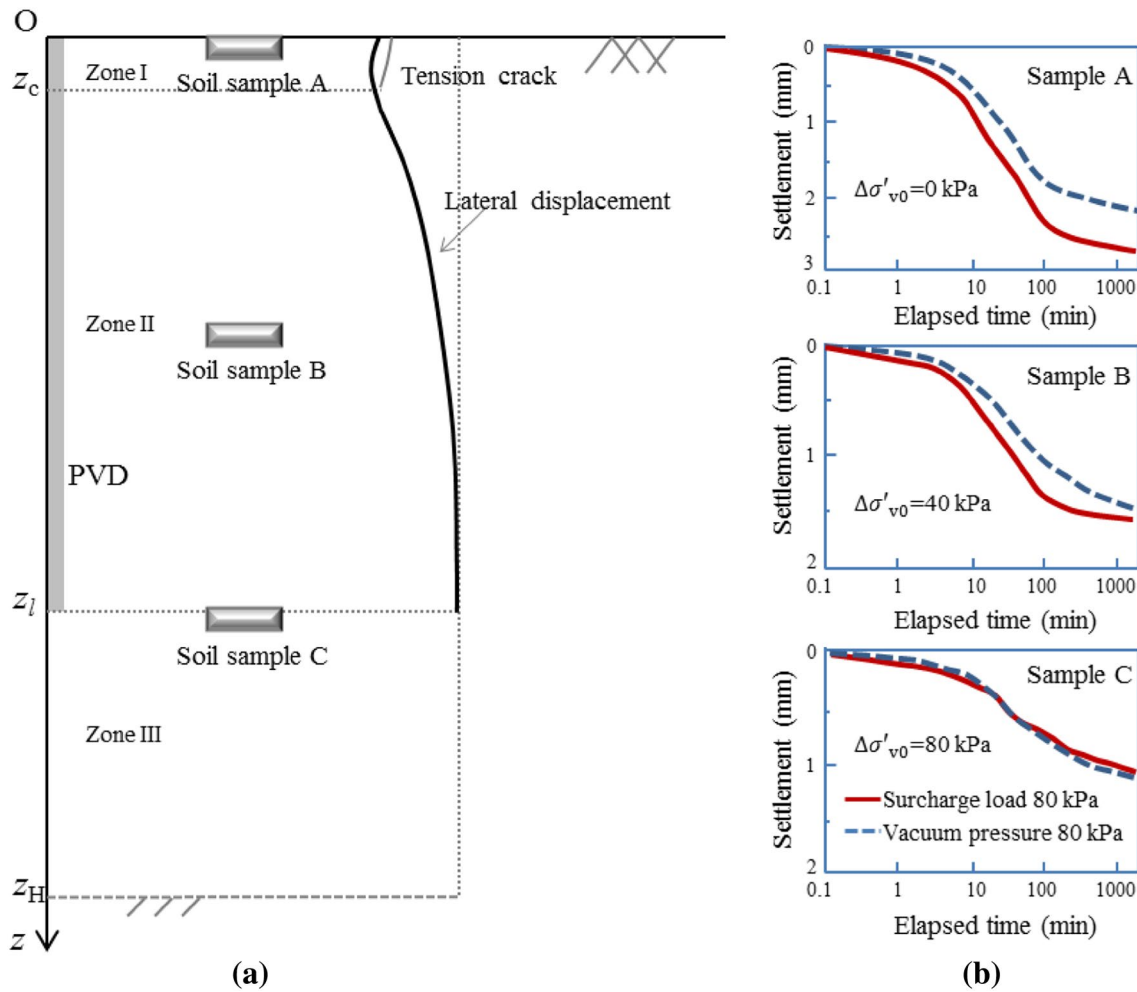
Fig. 1 Soil profile and corresponding soil elements under vacuum preloading. **a** Three zones for a ground under vacuum preloading; **b** compression of soil elements at different depths [10]

pressure, respectively. Robison et al. [11] presented some equations for determining the final stress state. It can be deduced from these theoretical analyses that the stress state and consolidation of a soil element under vacuum pressure are different from that which results from 1D consolidation. Thus, the behaviour of soil obtained from common laboratory tests under compression differs from those under vacuum pressure in the field.

Classical Constitutive Models and its Parameters for Soft Soil

The Modified Cam-clay model and the Soft Soil model are two constitutive models widely used to capture the deformation of soil elements when simulating problems relating to consolidation [17]. The compression index λ (or C_c) and the swelling index κ (or C_s) are the main parameters used

to depict the compression and rebound behaviour, respectively. λ and C_c are slopes of the virgin compression curve in e-lnp' and e-log p' plots, respectively; κ and C_s are slopes of unloading-reloading line in e-lnp' and e-log p' plots, respectively; e is the void ratio; and p' is the mean effective stress.

Moreover, as the void ratio e decreases during consolidation, the hydraulic conductivity k will also decrease so the following equation can be used to capture this phenomenon [18].

$$k = k_0 10^{(e-e_0)/C_k} \tag{2}$$

where k_0 is the initial value of hydraulic conductivity, e_0 is the initial void ratio, and C_k is the permeability index. Tavenas et al. [19] proposed an empirical relationship for C_k based on e for soft clay, i.e., $C_k = 0.5e_0$.

The above empirical relationships and parameters have been developed for positive loading conditions, but as discussed earlier, the consolidation of soil under a vacuum pressure differs from soil under surcharge preloading, so in the following sections an improved method is proposed by which the soil parameters are modified to better capture their nonlinear behaviour during vacuum consolidation.

Proposed Method

Modified Compression Index

Shi et al. [20] reports a ground improvement case history with vacuum preloading in a land reclamation project at Guangzhou Port, China where the calculated settlement with normal compression parameters are 10–43% greater than the settlement observed, and the simulated consolidation rate is smaller than the observation. Chai and Carter [4] presents a series of oedometer test results from either vacuum pressure or surcharge loading, or a combination with different initial effective vertical stresses, as shown in Fig. 1b. The tests indicate that settlement under a vacuum is less than or equal to settlement under a corresponding surcharge load, and it is sensitive to increasing the $\sigma_{vac}/\sigma'_{v0}$ ratio (σ_{vac} is the increase of vacuum pressure; σ'_{v0} is the initial effective vertical stress). For $\sigma_{vac} = \sigma'_{v0}$, the settlement induced by vacuum and surcharge are almost the same.

Considering these conditions, an adjustment parameter α_c, is used to capture the reduced soil compression under vacuum pressure. The modified compression index λ^* or C_c^* is therefore

$$\lambda^* = \alpha_c \lambda \text{ or } C_c^* = \alpha_c C_c \tag{3}$$

The parameter $\alpha_c \leq 1$ is a function of the vacuum pressure increment ($\Delta\sigma_{vac}$) and the initial vertical effective stress (σ'_{v0}). Chai et al. [10] also proposed a similar adjustment parameter for the compression index, so when the increase of vacuum pressure is equal to or smaller than the initial effective vertical stress, then $\alpha_c = 1$:

$$\alpha_c = 1.0 \text{ if } \Delta\sigma_{vac} \leq \sigma'_{v0} \tag{4}$$

The value of α_c varies with the depth z, and a simplified expression for calculating α_c can be proposed where:

$$\alpha_c = \begin{cases} \alpha_{c,min} & z \leq z_c \\ \alpha_{c,min} + \left(1 - \alpha_{c,min}\right)\frac{z-z_c}{z_l-z_c} & z_c < z < z_l \\ 1 & z \geq z_l \end{cases} \tag{5}$$

Depth z_l (see Fig. 1) can be determined by satisfying the following equation:

$$\sigma'_{v0}(z = z_l) = \Delta\sigma_{vac} \tag{6}$$

Note that for a vacuum preloading condition, Eq. (6) can be simplified as $z_l = L$, where L is the length of a PVD with no surcharge. Figure 2a shows the variation of α_c with depth.
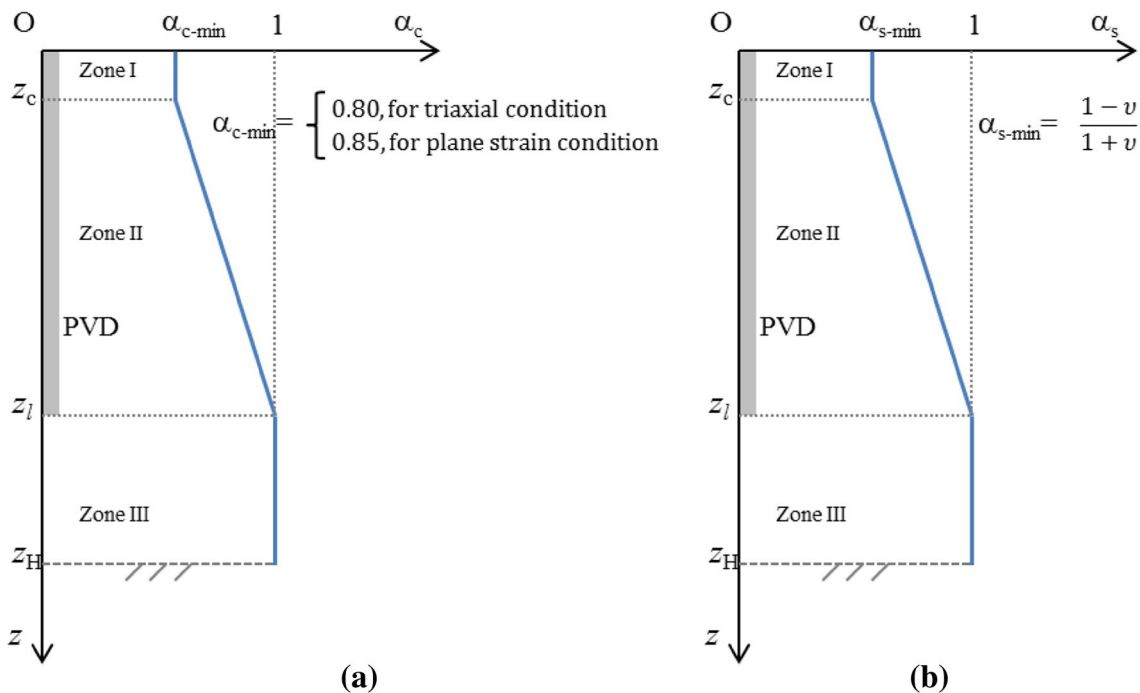

Fig. 2 Variation of α_c and α_s in depth of soil profile: **a** distribution of α_c; **b** distribution of α_s

Note that under K_0 condition (in Zone III), there is no lateral displacement and α_c reaches a maximum value of 1.0. Under an active K_a condition (in Zone II), the stress state deviates from the K_0 condition and $\alpha_c < 1.0$; and if the deviation increases, α_c decreases. In the tension crack zone (i.e. Zone I), the stress state deviates furthest from the K_0 condition and therefore α_c reaches a minimum value of $\alpha_{c,min}$. Note that the value $\alpha_{c,min}$ can be determined by a series of compression tests or triaxial tests at different stress states (i.e., in a K_a condition and a K_0 condition), though it can be also recommended to set $\alpha_{c,min} = 0.8$–0.85 according to the existing test results [10, 11].

Modified Swelling Index

Peng et al. [21] reports the application of vacuum pressure combined with surcharge preloading to improve a soft soil foundation in the Guangzhou-Zhuhai highway project. During the unloading phase (the vacuum is switched off) the calculated settlement decreased while the measured settlement continued to increase. Peng et al. [21] also refer to a field test in the Xiaoshan-Quzhou highway project where the actual rebound phenomenon is not as obvious as anticipated when the vacuum pumps are switched off. Shi et al. [20] carried out laboratory tests with a combination of vacuum and surcharge load (both are 40 kPa) and found a very small (around 0.1 mm) soil rebound after the vacuum was released. To capture the behaviour of soil subject to rebound after switching off the vacuum, an adjustment factor α_s is introduced here, so the modified swelling index will be

$$\kappa^* = \alpha_s \kappa \ \ or \ \ C_s^* = \alpha_s C_s \tag{7}$$

The distribution of α_s with depth, as shown in Fig. 2b, can be assumed to be similar to the one for α_c, which can be expressed as

$$\alpha_s = \begin{cases} \alpha_{s,min} & z \leq z_c \\ \alpha_{s,min} + \left(1 - \alpha_{s,min}\right)\dfrac{z - z_c}{z_l - z_c} & z_c < z < z_l \\ 1 & z \geq z_l \end{cases} \tag{8}$$

Note that $\alpha_s = 1.0$ if $\Delta\sigma_{vac} \leq \sigma'_{v0}$. Value $\alpha_{s,min}$ can be determined by the comparison of rebound amount of soil element under the stress conditions for vacuum consolidation and laboratory compression test. It has been pointed out previously that the deformation of a soil element near the ground surface subjected to vacuum pressure is closer to deformation under isotropic consolidation. Here ε_{iso} denotes the rebound strain under isotropic consolidation and ε_{1D} denotes the change in vertical strain that occurs during 1D-consolidation, so the value $\alpha_{s,min}$ can be determined by the following expression:

$$\alpha_{s,min} \frac{\varepsilon_{iso}}{\varepsilon_{1D}} \tag{9}$$

During unloading, soil rebound can be assumed as elastic, so based on elastic theory the vertical strain ε_{1D} can be calculated as:

$$\varepsilon_{1D} = \frac{\Delta\sigma_z}{E}\left(1 - \frac{2v^2}{1 - v}\right) \tag{10}$$

where $\Delta\sigma_z$ is the increment of vertical stress, E is the elastic modulus of the soil, and v is Poisson's ratio. Under isotropic conditions the changes of stress from all directions are the same, i.e., $\Delta\sigma_x = \Delta\sigma_y = \Delta\sigma_z$, so the vertical strain can be calculated by:

$$\varepsilon_{iso} = \frac{\Delta\sigma_z}{E}(1 - 2v) \tag{11}$$

By combining Eqs. (9)–(11) we have:

$$\alpha_{s,min} = \frac{1 - v}{1 + v} \tag{12}$$

It can be easily deduced from Eq. (12) that $\alpha_{s,min} = 0.33$–1.0 since $v = -0.5$, e.g. for $v = 0.33$, $\alpha_{s,min} = 0.5$. This means the vertical rebound of a soil element for an isotropic loading condition is half that for the K_0 condition (without lateral deformation) under the same change of vertical stress. We can also infer that using the swelling index obtained from a conventional one dimensional compression test will certainly overestimate the amount of rebound when simulating vacuum consolidation.

Modified Permeability Index

Gangaputhiran et al. [15] carried out laboratory tests to compare the properties of soil after surcharge or vacuum preloading and found the horizontal permeability of soil samples to be generally larger when subject to vacuum pressure rather than surcharge loading. Note that horizontal flow mainly controls the consolidation of vertically drained soft ground. Kianfar et al. [22] carried out consolidation tests using a Rowe cell and monitored the dissipation of pore pressure at different locations; they found the dissipation of excess pore pressure under vacuum pressure to be faster than under a surcharge alone. Saowapakpiboon et al. [5] compared the consolidation of two ground improvement sites with and without vacuum preloading and found the final settlements were almost the same for the two sites, but the time for vacuum preloading to reach a 90% degree of consolidation was one-third shorter than for surcharge preloading. They therefore concluded that horizontal permeability was greater with vacuum preloading than with surcharge preloading. It can be inferred from their

studies that the reduction in hydraulic conductivity during consolidation can be smaller for vacuum consolidation.

To capture this behaviour, another adjustment factor α_k is introduced into this paper and the modified permeability index C_k^* is expressed as:

$$C_k^* = \alpha_k C_k \tag{13}$$

The consolidation tests for soil elements [15, 22] and model tests for PVD system [5] indicate that $\alpha_k \geq 1.0$, where α_k is assumed to be constant along the depth within the PVD penetration depth $(0 \sim L)$, and below that where $\alpha_k = 1$. For a normally consolidated soil, according to Eq. (2), it can be obtained that

$$e = e_0 + C_k \log\left(k_{sur}/k_0\right) = e_0 + C_k^* \log\left(k_{vac}/k_0\right) \tag{14}$$

where k_{sur} and k_{vac} are permeability coefficients of soil under surcharge preloading and vacuum preloading, respectively. It can be deduced from Eq. (14) that

$$\alpha_k = C_k^*/C_k \frac{\log(k_{sur}/k_0)}{\log(k_{vac}/k_0)} \tag{15}$$

Thus, value α_k can be determined by the comparison of soil permeability after surcharge and vacuum preloading [15]. Here it is recommended to use $\alpha_k = 1.0$–1.3 and set $\alpha_k = 1.25$ in the simulation of vacuum consolidation in lack of any other data.

Table 1 summarises the main aspects of vacuum consolidation in comparison to surcharge preloading. The newly introduced factors for adjusting the consolidation parameters can easily be applied in commercial finite element programs (e.g., PLAXIS) to simulate vacuum-assisted consolidation. All the adjustments needed to capture these

aspects are suggested in this paper and will be examined thoroughly in the proceeding sections.

Simulation of a Laboratory Model Test

Model Set Up and Parameters

Saowapakpiboon et al. [6] use a large-scale consolidometer (0.45 m in diameter and 0.70 m high) to compare the consolidation of a PVD system with and without vacuum pressure. Figure 4 shows the model condition in this simulation with PLAXIS 2D, which consists of a vertical drain, smear zone, and undisturbed zone. Further details are given in Table 2, where the equivalent diameter of PVD is $d_w = (b + \delta)/2$ (b and δ are the width and thickness of PVD, respectively), the diameter of smear zone d_s is equal to twice the equivalent mandrel diameter [6].

The Soft Soil model is used in the simulation and the corresponding parameters are also listed in Table 2. Here, the relationships $C_c = 2.3\lambda$, $C_s = C_c/10$, and $C_k = 0.5e_0$ are used [17]. POP is the pre-overburden pressure used to establish the initial stress states for both tests. The compression properties of the drain and smear zone are assumed to be the same as the soil, i.e., only the permeability of the three zones is different. The characteristic permeability of the smear zone, which can be denoted as the ratio of horizontal permeability of soil in an undisturbed area to the smeared zone soil (k_h/k_s), has been studied earlier [23, 24]. In this simulation the ratio k_h/k_s and permeability of the drain (k_w) are taken from Saowapakpiboon et al. [6]. It is assumed that the initial permeability is isotropic, i.e. $k_{h0} = k_{v0}$.

Table 1 Differences in consolidation between vacuum pressure and surcharge preloading

Phenomenon	References	Parameters modification
Soil compression under vacuum pressure is smaller than or equal to that induced by positive pressure with the same magnitude	Saowapakpiboon et al. [5] Chai et al. [10] Robison et al. [11] Wu et al. [12]	Introducing a factor α_c (≤ 1) to decrease the compression index of soil element
Soil rebound after stopping the vacuum is not as high as after removing the surcharge	Wu et al. [12] Shi et al. [20] Peng et al. [21]	Introducing a factor α_s (≤ 1) to decrease the swelling index of soil element
The consolidation rate is usually faster under vacuum pressure than with surcharge preloading	Saowapakpiboon et al. [5] Rondonuwu et al. [7] Indraratna et al. [8] Chai et al. [14] Gangaputhiran et al. [15]	Introducing a factor α_k (≥ 1) to increase the permeability index of soil element

Table 2 Parameters and its value in the simulation of laboratory model test

d_w (mm)	d_s (mm)	d_e (mm)	$H = L$ (m)	γ_t (kN/m³)	e_0 (/)	C_c (/)	C_s (/)	C_k (/)	ν (/)	POP (kPa)	k_w (m/s)	$k_{h0} = k_{v0}$ (m/s)	k_h/k_s (/)
26.8	87.1	450	0.7	14.7	2.29	1.31	0.13	1.15	0.3	50	5.62×10^{-2}	2.0×10^{-9}	3
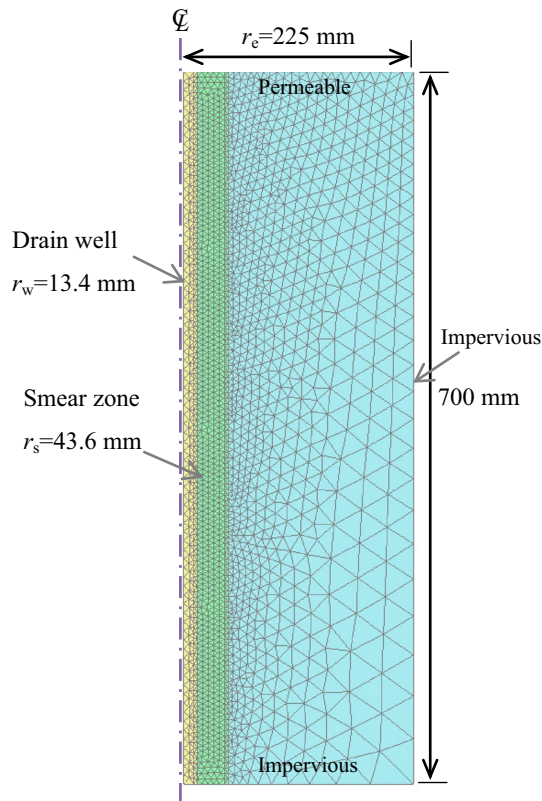
Fig. 3 Schematic diagram of mesh and boundary conditions for a model test with PLAXIS

Fig. 4 Comparison of measured and simulated result for vacuum consolidation model test. **a** settlement; **b** excess pore water pressure

The mesh discretization is also shown in Fig. 3, where a two-dimensional axisymmetric model consisting of 4011 triangular 6-node elements and 32,403 nodes is used; the left hand side of the mesh denotes symmetry boundary (no horizontal displacement); the top surface is permeable and free to deform; the base is impermeable and fully constrained, the peripheral boundaries are impervious to water flow and the horizontal component of displacement is fixed. A uniform downward pressure of 100 kPa is applied instantaneously onto the top surface for surcharge alone model. For the combined preloading condition, 50 kPa compressive pressure (on the top surface) and 50 kPa vacuum (by setting a constant water head $h = -5$ m along the drain in depth) are applied instantaneously.

The input data for simulating vacuum preloading is the same as those for surcharge preloading, and the three newly introduced adjustment factors are applied as described below:

- According to the model test description $\Delta\sigma_{vac} = 50$ kPa, and $\sigma'_{v0} = 50$ kPa; thus $\Delta\sigma_{vac} = \sigma'_{v0}$, and hence $\alpha_c = 1$ or $C_s^* = C_c$.
- Since $\Delta\sigma_{vac} = \sigma'_{v0}$, then $\alpha_s = 1$ or $C_s^* = C_s$.
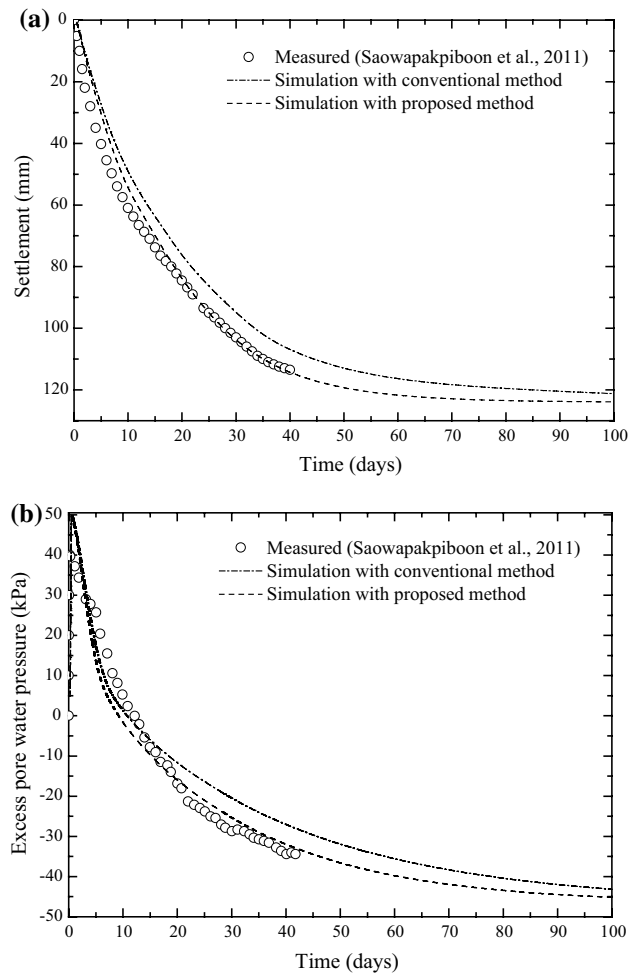
- $\alpha_k = 1.30$ is based on a back analysis which implies that $C_k^* = 1.3C_k$.

Simulation Results

A simulated surcharge only test to calibrate the material parameters has been carried out (results not shown here), followed by a simulation of the vacuum-assisted preloading test with and without the proposed method; the results for vacuum consolidation are shown in Fig. 4a. Note the obvious difference between the measured value and the FEM results when the proposed method is not applied in the simulation. The proposed method with adjustment factors however gives much better results compared to the measured data. All the simulated settlements are smaller than the measured data when preloading begins, but they are more consistent towards the end of consolidation.

The measured and simulated dissipation of pore water pressure with time at the mid-height of the specimen are

Table 3 Soil layers and PVD properties for Port Brisbane

Sections	Dredged materials	Upper Holocene clay	Lower Holocene clay	Pleistocene layer	L (m)	B_e (m)	B_s (m)	d_w (m)	k_w (m/s)
WD2	0.0–3.5	3.5–7.5	7.5–24.5	24.5–30	24.5	1.3	0.1	0.034	1.74×10^{-4}
VC2	0.0–3.5	3.5–6.0	6.0–21.0	21.0–30	21.0	1.2	0.1	0.034	1.74×10^{-4}

Table 4 Material parameters and soil layers at the Port of Brisbane

Soil layers	γ_t (kN/m³)	e_0 (/)	C_c (/)	C_s (/)	C_k (/)	ν (/)	POP (kPa)	OCR (/)	k_h/k_v (/)	k_h/k_s (/)	k_{h0} (m/s)
Dredged materials	14.0	2.62	0.85	0.085	1.31	0.42	20	/	1	1	2.0×10^{-9}
Upper Holocene clay	16.0	2.17	0.75	0.075	1.09	0.35	/	1.0	2	2	1.3×10^{-8}
Lower Holocene clay	16.0	1.96	0.68	0.068	0.98	0.35	/	1.0	2	2	5.1×10^{-9}
Pleistocene	17.5	0.75	0.20	0.02	0.38	0.30	/	4.0	2	/	1.0×10^{-8}

compared, as shown in Fig. 4b. It can be found that the measured data shows a faster rate of dissipation than the simulations; the consolidation rate attributed to the proposed method is closer to the measurements than conventional analysis.

Simulation of Vacuum-Assisted Ground Improvement at Port Brisbane

Site Conditions and Simulation Parameters

Indraratna et al. [8] reported a ground improvement project with vacuum-assisted preloading in conjunction with prefabricated vertical drains at Brisbane, Australia. The trial treating area was sub-divided into seven areas with five non-vacuum (WD series) and two vacuum combined areas with membrane-type vacuum consolidation system (VC series). The non-vacuum area WD2 and the vacuum area VC2 were close together and had approximately the same depth of soft clay and wick drains; the total loading for these two areas was similar.

The main geological formations for the trial areas are normally consolidated Holocene deposits overlying over-consolidated Pleistocene deposits, which in turn overlie the basalt bedrock [25]. Table 3 shows the average thickness of the soil layers in these two areas. The groundwater level is roughly between the work platform and dredged mud, so $z_w = 0$ is assumed in the simulation. Vertical drains have been installed to penetrate through the Holocene clay and Table 3 shows their characteristics. The Soft Soil model is used to depict the soil behaviour and the material parameters are presented in Table 4. A 20 kPa POP is assumed for the dredged materials after considering the load applied by the construction machine, and the remaining parameters are taken from Indraratna et al. [8]. The boundary conditions and
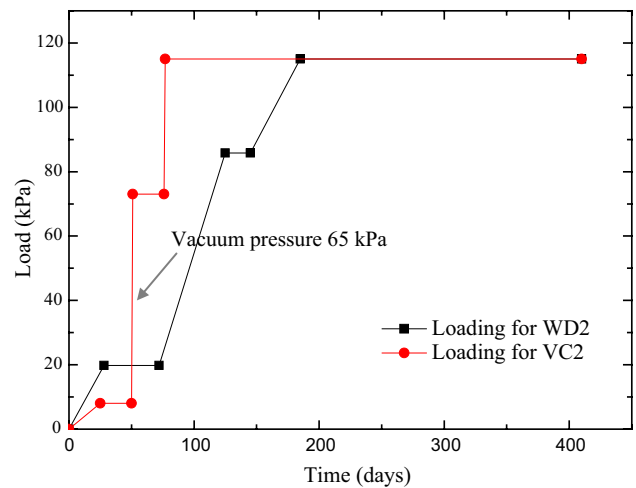

Fig. 5 Loading sequences of two areas in the Port of Brisbane

mesh discretization are basically the same as the previous simulation for model test. Here a total of 2299 triangular 6-node elements and 19,255 nodes are used in the FEM mesh discretization. Figure 5 shows the loading sequences for the two areas. The total fill height for WD2 is 7 m (equivalent to a surcharge load of about 115 kPa). In VC2, a vacuum pressure is applied after 40 days preloading, so the total increase of applied load is 115 kPa, consisting of 65 kPa of vacuum and 50 kPa of fill loading.

The adjustment parameters are selected via the following procedure:

- z_c: A tension crack will not take place in the ground because a fill surcharge has been added and thus z_c is set as 0.
- z_L: Calculated based on Eq. (6).
- α_c: Determined by Eq. (5) and $\alpha_{c,min} = 0.8$.
- α_s: Calculated by Eq. (8) and Eq. (12).

- α_k is set as 1.25.
- Modified parameters C_c^*, C_s^*, and C_k^* are calculated by Eqs. (3), (7), and (13), respectively.

Simulation Results

Figure 6a shows a comparison of the measured surface settlement and the results of the simulation. The solid line
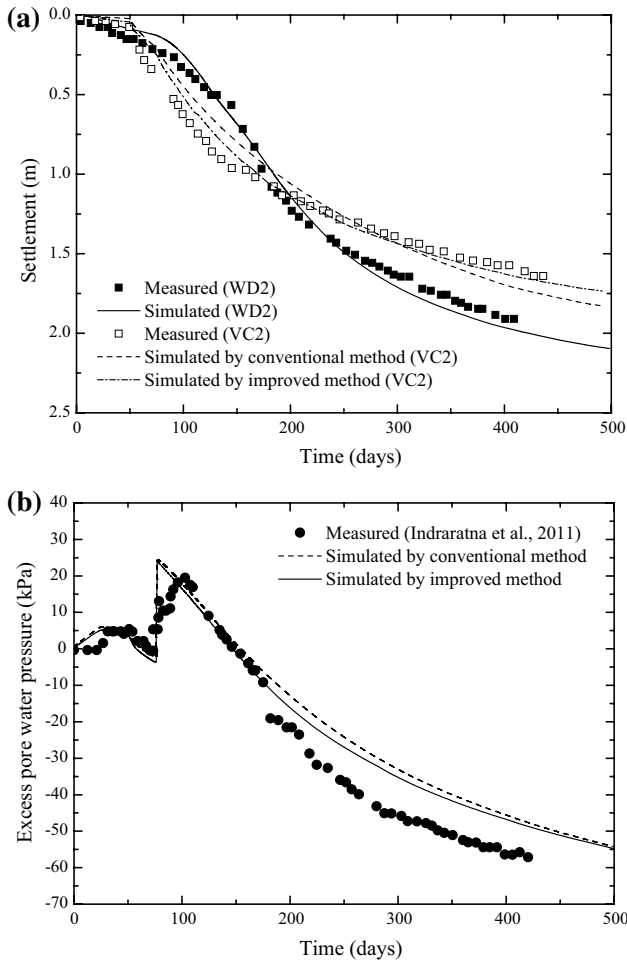
denotes the simulation results for area WD2 (surcharge preloading) and show that they fit the measured data very well. The dash lines denote the simulated results for area VC2 with and without the proposed method; note that the proposed method gives better results for consolidation process than the simulation without the proposed method.

Figure 6b presents the development of excess pore water pressure at depth $z = 14.1$ m. The FEM results related to the dissipation of excess pore water pressure are also plotted in the figure for comparison. It can be seen in this figure that the simulation results are generally less than the measured excess pore water pressure. The improved method however shows a faster dissipation rate of pore water pressure than the conventional method, leading to slightly closer prediction to the measured data.

Simulation of Ground Improvement for a Road Project in Guangzhou

Model Set Up and Material Parameters

Li et al. [26] report a case concerning the application of vacuum preloading in an expressway project in Guangzhou, China. Table 5 lists the soil layers and their material properties. The water table is reported to be at the top surface of muddy clay. A plane strain model (PLAXIS 2D) is used to simulate the road embankment because of its length compared to the width of the model. Since this system is actually symmetrical, only half the embankment is simulated and the model is extended laterally for 84 m to minimise the effect of lateral boundaries, as shown in Fig. 7. The drains are modelled by the 'Drain' element of PLAXIS 2D; their vacuum is set with a constant water head $h = -8$ m (simulating a negative pressure of 80 kPa), i.e. the well resistance is omitted due to the high discharge capacity of the drains. Table 6 shows the properties of the drain elements. The smear zone is assumed to be twice the equivalent diameter of the mandrel, and the embankment fill is assumed to be Mohr-Coulomb material with the following properties: $\gamma_t = 18.2$ kN/m^3, $E = 20$ MPa; $\nu = 0.33$; $c' = 10$ kPa; $\varphi' = 30$.

Fig. 6 Comparison of measured and simulated results for the Port of Brisbane project. **a** Settlement variation with time; **b** pore water pressure variation with time

Table 5 Soil layers and material parameters for Guangzhou Project

| Soil layers	Depth (m)	γ_t (kN/m^3)	e_0 (/)	λ (/)	κ (/)	c' (kPa)	φ' (°)	C_k (/)	k_{ha} (m/s)	k_{hp} (m/s)	k_{v0} (m/s)	OCR (/)	POP (kPa)
Muddy clay	0–4.5	16.4	1.65	0.22	0.04	4.0	22.6	0.83	2.47×10^{-9}	1.59×10^{-10}	2.42×10^{-9}	1.0	30
soft silty clay	4.5–10.5	17.0	1.32	0.12	0.02	3.8	33.2	0.66	5.31×10^{-7}	3.43×10^{-8}	2.36×10^{-7}	1.2	0.0
soft clay	10.5–18.0	16.5	1.52	0.15	0.03	5.1	24.8	0.76	6.81×10^{-9}	4.40×10^{-10}	2.41×10^{-9}	1.0	0.0
Silty clay	18.0–39.5	17.8	1.25	0.14	0.02	10	26.0	0.63	9.00×10^{-8}	9.00×10^{-8}	4.50×10^{-8}	1.0	0.0
Hard silty clay	39.5–48	17.5	1.08	0.12	0.02	20	30.0	0.54	1.00×10^{-7}	1.00×10^{-7}	1.00×10^{-7}	1.0	0.0
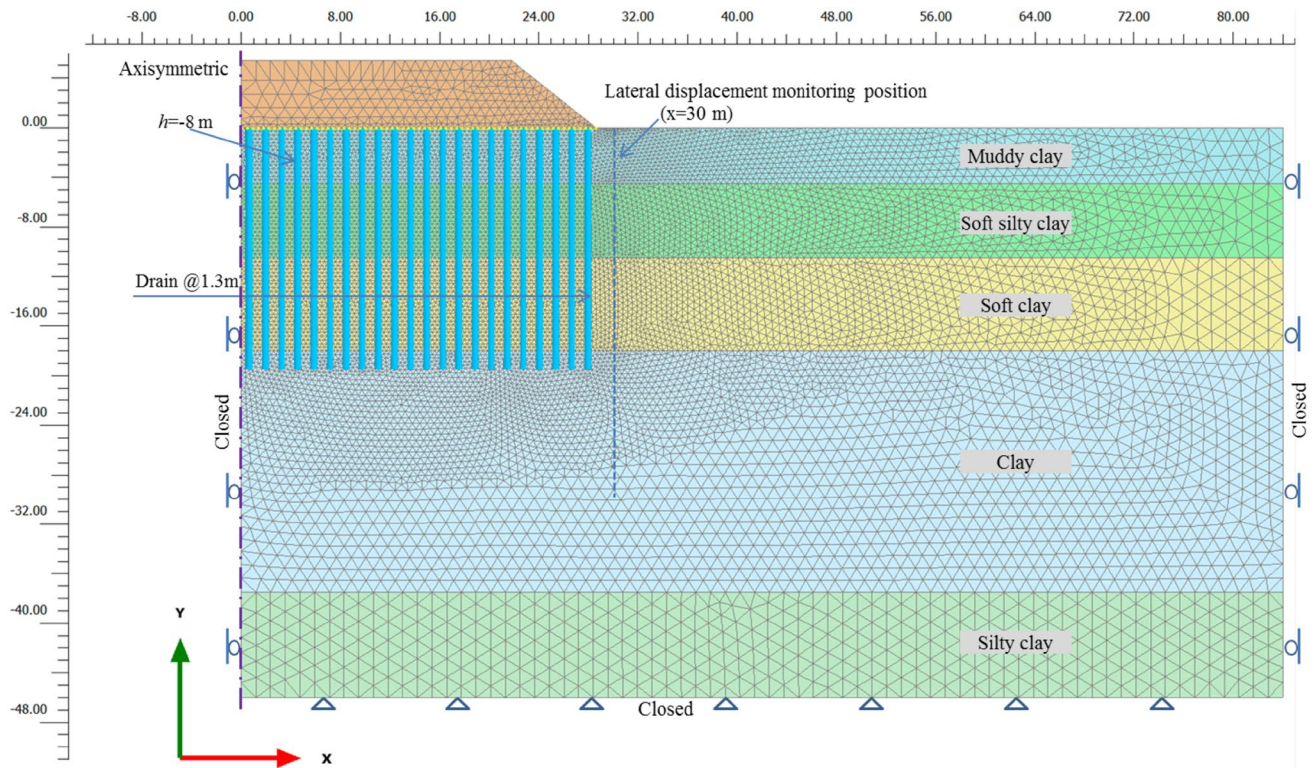
Fig. 7 Model geometry, mesh discretisation, and boundary conditions for Guangzhou project

Table 6 Parameters related to the behaviour of vertical drain

d_w (mm)	d_s (mm)	S (m)	d_e (m)	L (m)	$s = d_s/d_w$ (/)	$n = d_e/d_w$ (/)	k_h/k_s (/)
70	420	1.3	1.365	20	6	19.5	5

The coefficients of permeability of the soil on this site should be transformed from 3D to a plane strain condition. Indraratna et al. [27] propose an analytical method to convert the permeability of undisturbed and smeared soil, whereas Tran and Mitachi [28] propose a similar method by introducing the combined permeability of undisturbed and smeared soil after being transformed. For convenience, Tran and Mitachi's method [28] is used here and the calculation of permeability for plane strain condition is carried out using the following expression:

$$k_{hp} = \frac{2B^2}{3R^2} \frac{k_{ha}}{\ln\frac{n}{s} - \frac{3}{4} + \frac{k_h}{k_s}\ln s} \tag{16}$$

where k_{ha} and k_{hp} are the horizontal coefficients of permeability of the undisturbed soil before and after being transformed, respectively; B is the half-width of the plane strain unit cell, i.e. $B = S/2$; S is spacing between the drains; R is the radius of the axisymmetric unit cell, i.e. $R = d_e/2$. By using the parameters in Table 6 it can be calculated

that $k_{hp} = 0.0645k_{ha}$. The equivalent plane strain parameters determined from Eq. (16) are also shown in Table 5. The adjustments factors are selected via the following procedure:

- z_c: Using $z_w = 0$ and Eq. (1) with parameters listed in Table 3, z_c is calculated to be 1.9 m. Note that a vacuum pressure has been applied before placing the fill material, which means the cracked zone formed before surcharge preloading.
- z_l: According to the initial stress state of the ground, and based on Eq. (6), z_l is calculated as 12 m.
- α_c: For a plane strain condition $\alpha_{c,min} = 0.85$, and then the value α_c for each layer of soil is calculated by Eq. (5).
- α_s: Using $\nu = 0.4$ we have $\alpha_{s,min} = 0.43$ from Eq. (12), and then α_s for each soil layer can be calculated by Eq. (8).
- α_k is set as 1.25.

Table 7 Adjustment factors and modified parameters for Guangzhou project

Soil layers	α_c	α_s	α_k	λ^*	κ^*	C_k^*
Muddy clay	0.86	0.44	1.25	0.188	0.018	1.03
Soft silt clay	0.93	0.72	1.25	0.111	0.015	0.83
Soft clay	0.98	0.95	1.25	0.147	0.029	0.95
Clay	1.00	1.00	1.00	0.140	0.020	0.63
Silty clay	1.00	1.00	1.00	0.120	0.020	0.54
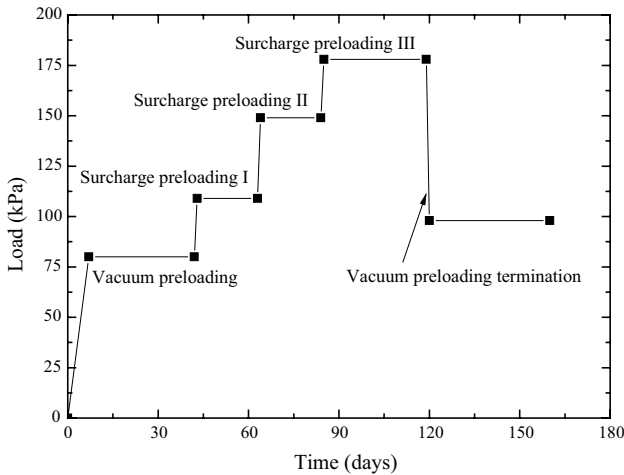
Fig. 8 Loading sequences for the Guangzhou project

Fig. 9 Comparison between measured and simulated settlements in Guangzhou project

- The modified parameters λ^*, κ^*, and C_k^* are calculated by Eqs. (3), (7), and (13), respectively, as summarised in Table 7

The initial stress state is simulated with the self-weight of soil elements using the OCR and POP values listed in Table 5. The mesh and boundary conditions are shown in Fig. 7. The 2D finite element mesh consists of 22,683 triangular 6-node elements and 45,736 nodes, and the bottom and side boundaries for water flow are set as closed. Figure 8 shows the loading history of this combined preloading process. Note that the vacuum pressure applied at the ground surface is almost 80 kPa and it was maintained for 4 months. Fill (silty clay) was applied in three stages (i.e., I, II and III) after 40 days of applying a vacuum pressure.

Fig. 10 Profiles of horizontal displacements at the toe of embankment (x = 30 m) in Guangzhou project

Simulation Results

Figure 9 shows the settlement of ground surface recorded at the centre line of the embankment, as well as the simulation results using the conventional method and the proposed method. Note that the simulated final settlement obtained by the proposed method is slightly smaller than that obtained by the conventional method. The proposed method generally leads to a more accurate prediction of
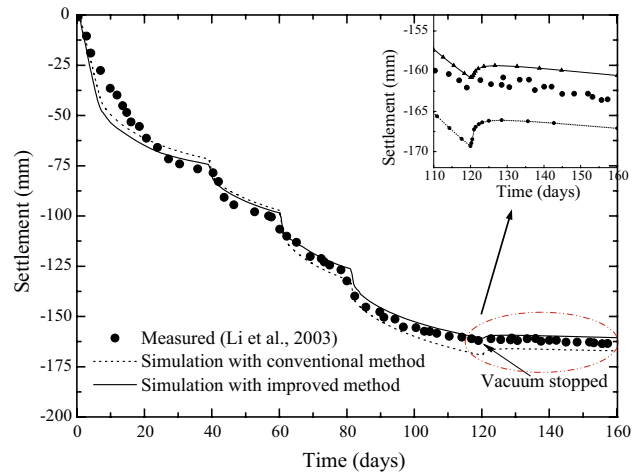
settlement, especially in the latter part of the preloading stage. After the vacuum is stopped, both methods show an obvious rebound, with the conventional method and the proposed method having a rebound of around 4 and 2 mm,

respectively. The difference between the numerical rebound and measured rebound obviously decreases using the proposed method.

Figure 10 shows the horizontal displacement at the toe of the embankment (see Fig. 7; $x = 30$ m) and compares the simulation results of conventional and proposed methods. The lateral displacement after 40 days of vacuum application is also shown in the figure; with the proposed method having a more accurate lateral displacement profile, especially at a shallower depth. Before the entire fill has been placed (e.g. after 40 days of vacuum application), the profile of the entire horizontal displacement is inwards, but when fill is applied at the later stages, it reduces inward displacement and causes outward displacement in the soft ground at depths deeper than 9 m. It is worth pointing out that when the vacuum stopped (after 120 days), the proposed method has a larger outward displacement than the conventional method, which is more consistent with the results of Rondonuwu et al. tests [7].

Conclusions

Field studies and laboratory tests indicate that the consolidation of soil under vacuum pressure differs from that under a surcharge loading, and the parameters obtained from conventional laboratory tests may bring inaccuracies into the simulation of vacuum consolidation. To address this gap, a novel method is proposed based on three adjustment factors for the material parameters. Simulations on a model test and two case histories have been carried out to illustrate the application of this new method, from which the following conclusions are drawn.

1. Adding vacuum preloading onto a soft ground may result in three zones developing, namely the crack zone, the active zone, and the at-rest zone; and due to different confining conditions and initial stress states, the consolidation of soil in these zones may also be affected.

2. Factor α_c is used to capture the phenomenon whereby the compression of soil under a vacuum pressure is smaller than or equal to that induced by positive pressure of the same magnitude in a one dimensional condition. The application of this factor (α_c between 0.8–1.0) results in a decrease in final settlement.

3. The rebound of soil after stopping the vacuum is smaller than when stopping a positive pressure; and the newly introduced factor α_s (≤ 1.0) can capture this difference. According to the theory of elasticity, the minimum value of α_s relates to Poisson's ratio and is between 0.33–1.0. The simulated rebound will decrease when α_s is applied in the simulation.

4. Unlike surcharge preloading, a vacuum pressure results in a smaller deviatoric stress, so the soil skeleton is well maintained for vacuum consolidation; this then results in a smaller decrease of permeability and a faster consolidation rate under vacuum preloading. The newly introduced factor α_k (≥ 1.0) captures this phenomenon. According to the existing test results, a value between 1.0–1.3 for α_k is recommended, and the consolidation rate will increase as $\alpha_k > 1$ is applied.

5. Three examples in this paper have shown a good accuracy in the simulation of vacuum consolidation with the proposed method. This new method captures more realistic responses in numerical simulations of vacuum consolidation and is therefore recommended for practical use.

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