Ground Deformation Induced by Vacuum Consolidation

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Abstract: The deformation characteristics of soil subjected to vacuum pressure are discussed and an approximate method is proposed for calculating settlement and lateral displacement of the ground induced by vacuum consolidation. Laboratory oedometer test results indicate that if the vacuum pressure alone is larger than the lateral stress required to maintain an at-rest (no horizontal strain) condition, there will be inward lateral displacement and the vacuum pressure will induce generally less settlement than a surcharge load of the same magnitude. In the case of field vacuum consolidation, the confining stress acting on a soil element can be regarded as consisting of two parts: Due to vacuum pressure and earth pressure. Assuming a value of the lateral earth pressure coefficient acting in the ground under vacuum consolidation (k_{ao}), somewhere between the active and at-rest values, an equation defining the depth—below which there will be no significant inward lateral displacement—is derived. Further, assuming that the volumetric strain induced by vacuum consolidation is the same as the one-dimensional consolidation induced by application of a surcharge load of the same magnitude, an approximate method is proposed for calculating the ground settlement and inward lateral displacement induced by vacuum consolidation. This method has been applied to two case histories reported in the literature, and it is shown that the field-measured data are simulated reasonably well, suggesting that the method may be useful for the design of vacuum consolidation projects.

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Introduction

Preloading is a common method used to improve soft clayey soil deposits. The effective surcharge pressure for preloading can arise from either the weight of imposed fill material (e.g., an embankment) and/or the application of a vacuum pressure applied to a saturated soil. Using a vacuum pressure has several advantages over embankment loading, e.g., no fill material is required, construction periods are generally shorter, and there is no need for heavy machinery. In addition, the vacuum pressure method does not put any chemical admixtures into the ground and, consequently, it is an environmentally friendly ground improvement method. Several applications of the use of the vacuum consolidation method to improve soft clayey deposits have been reported (e.g., Bergado et al. 1998; Chu et al. 2000; Tang and Shang 2000; Tran et al. 2004).

There are still differing opinions regarding the important characteristics of vacuum consolidation. For example, Mohamedelhassan and Shang (2002) reported that vacuum consolidation can result in settlements nearly identical to those induced by a surcharge loading applied under oedometer conditions. However, Chai et al. (2005) suggested that vacuum consolidation is also influenced by the drainage boundary conditions and will normally result in less settlement than application of a surcharge load with the same magnitude as the vacuum pressure. Vacuum consolidation generally induces inward lateral displacement and can cause cracks around the perimeter of the area to which the vacuum treatment is applied. There is a need to resolve these differing opinions and to develop a reliable method to predict the settlement and lateral displacement of the ground caused by vacuum consolidation.

In this paper, the results of a series of laboratory oedometer tests involving vacuum pressure loading and the conventional surcharge loading are reported and compared, in order to investigate the characteristics of vacuum consolidation. Based on these test results, an approximate method is presented for calculating the settlement and lateral displacement of the ground induced by vacuum consolidation. Two field cases are analyzed by the method proposed in this study and the predictions are compared favorably with field observations.

Oedometer Behavior

Chai et al. (2005) previously reported that under oedometer conditions, vacuum consolidation will yield less settlement than the application of an equivalent surcharge load. However, in those tests, the initial effective stress applied to the soil sample was zero, or at least close to zero for samples with B values (ratio of incremental pore pressure and corresponding applied incremental surcharge load applied under undrained conditions) greater than 0.9. In this study, a series of laboratory oedometer tests with one-way drainage conditions was conducted under either vacuum pressure or surcharge loading, on samples with different initial effective stresses, to further investigate the mechanism of vacuum consolidation. In tests with nonzero initial effective stress, the

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Table 1. Summary of the Loading and Drainage Boundary Conditions for the Laboratory Tests

Case	Initial vertical effective stress σ'_{vo} (kPa)	Incremental surcharge load $\Delta\sigma_v$ (kPa)	Incremental vacuum pressure $\Delta\sigma_{vac}$ (kPa)	Drainage boundary condition
1-a	0	80	—	One way
1-b		—	80	One way
2-a	40	80	—	One way
2-b		—	80	One way
3-a	80	80	—	One way
3-b		—	80	One way

sample was first consolidated under a predetermined stress for 24 h and then an incremental surcharge load or vacuum pressure was applied, and the settlement and excess pore pressure at the bottom of the sample were monitored (top of the sample was a free drainage boundary). Three representative scenarios were considered: (1) Samples at or near the ground surface, (2) samples at about middepth in a treated region, and (3) samples located deeper in the ground. The correspondence values of initial effective stress, incremental load, and drainage boundary conditions are summarized in Table 1. Considering that in the field the maximum achievable vacuum pressure is about 80 kPa (Bergado et al. 1998; Tang and Shang 2000), for all tests the incremental surcharge load and/or vacuum pressure was limited to this value.

The equipment used for this testing was a Maruto Multiple Odometer apparatus (Tokyo, Japan). Each sample was 60 mm in diameter and typically 20 mm thick. The soil tested was reconstituted Ariake clay, which was preconsolidated under a pressure of 30 kPa. The physical properties of the sample are listed in Table 2. For each test condition, two parallel tests were conducted to check repeatability. Before the start of each consolidation test, the soil sample was saturated to have a B value of greater than 0.9.

Comparisons of the settlement versus time curves for Cases 1-a and 1-b, 2-a and 2-b, and 3-a and 3-b (Table 1) are given in Figs. 1 to 3, respectively. In Fig. 1, all test data are given for Cases 1-a and 1-b. It can be seen that the scatter was small. For clarity, in Figs. 2 and 3 only the average values are reported. It can be seen that when the initial vertical effective stress is low (0 and 40 kPa), the vacuum pressure-induced settlement is less than that observed under the corresponding surcharge load (Figs. 1 and 2). For the case where the initial vertical effective stress is 80 kPa, the settlements induced by vacuum pressure and surcharge load are almost the same (Fig. 3). For Cases 1-b and 2-b (vacuum pressure), when disassembling the apparatus, it was observed that the soil samples had separated from the confining ring. This is probably because vacuum pressure applies an isotropic incremental stress to the soil which tends to induce an inward lateral displacement. Generally, whether the vacuum pressure can result in the same settlement as a corresponding surcharge load

Table 2. Physical Properties of the Laboratory Soil Samples

Soil particles (%)			Initial water content w (%)	Liquid limit w_l (%)	Plastic limit w_p (%)	Initial void ratio e_0
Clay (<5 μm)	Silt	Sand				
31.0	67.8	1.2	97.1	116.6	57.5	2.32

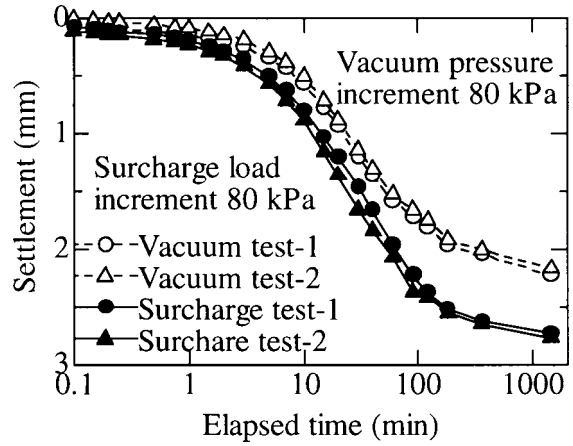

Fig. 1. Comparison of the settlement-time curves of Cases 1-a and 1-b

under oedometer conditions depends on whether a k_o condition (no horizontal strain) can be maintained. Under oedometer conditions and with an incremental vacuum pressure loading, if there is any lateral displacement in the sample, there will eventually be no confining stress applied by the odometer constraining ring, and the only horizontal stress will be due to the vacuum pressure. Therefore, if the vacuum pressure is larger than the stress required to maintain a k_o condition, there will be inward lateral displacement and the vacuum pressure will induce less settlement than the surcharge load. Otherwise, there will be no lateral deformation and the vacuum pressure will induce the same settlement as an equivalent surcharge load. This situation is illustrated schematically in Fig. 4 and the condition for inward lateral displacement to occur can be given as follows:

$$\Delta\sigma_{vac} > \frac{k_o \cdot \sigma'_{vo}}{1 - k_o} \tag{1}$$

where k_o=at-rest horizontal earth pressure coefficient; σ'_{vo} is in situ vertical effective stress; and $\Delta\sigma_{vac}$=incremental vacuum pressure. Mayne and Kulhawy (1982) proposed an equation to calculate k_o values as a function of stress history and shear strength, and their proposal is adopted in this study, i.e.,

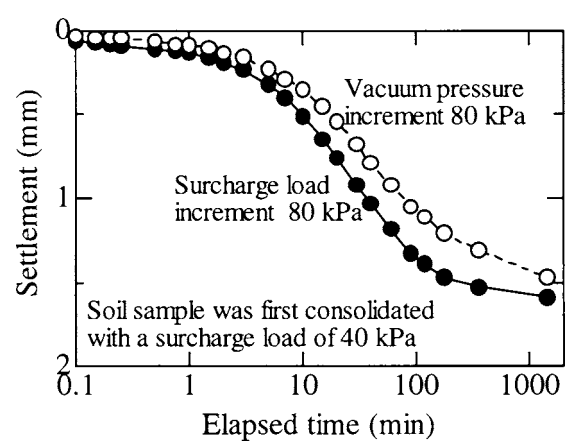

Fig. 2. Comparison of the settlement-time curves of Cases 2-a and 2-b

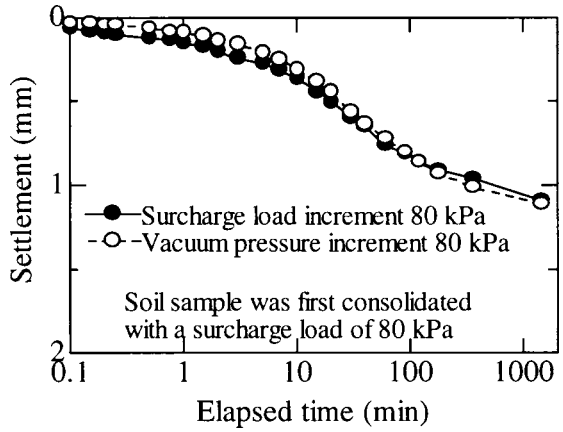

Fig. 3. Comparison of the settlement-time curves of Cases 3-a and 3-b

$$k_o = (1 - \sin\phi') \cdot (\mathrm{OCR})^{\sin\phi'} \tag{2}$$

where ϕ'=effective stress friction angle; and OCR=overconsolidation ratio of the soil. For reconstituted Ariake clay, the effective stress friction angle is about 30° and thus a value of 0.5 can be calculated from Eq. (2) for k_o, assuming OCR=1. For the laboratory tests conducted, Cases 1-b and 2-b satisfy condition (1) and inward lateral displacements were observed [as in Fig. 4(c)]. For case Case 3-b, the initial consolidation pressure, σ'_{vo}, was just sufficient to maintain the k_o condition of the sample when the vacuum was applied, and so no lateral displacement was observed.

If a stress ratio k is defined as follows:

$$k = \frac{\Delta\sigma_{\mathrm{vac}}}{\Delta\sigma_{\mathrm{vac}} + \sigma'_{v0}} \tag{3}$$

then, if $k \leqslant k_o$, there will be no lateral displacement and vice versa. Fig. 5 shows the relationship between the stress ratio k and the settlement ratio S_{vac}/S_l obtained from the oedometer test results (S_{vac} is the settlement induced by an incremental vacuum pressure and S_l is the settlement induced by a corresponding incremental surcharge load). It can be seen that the settlement ratio increases almost linearly with decreasing stress ratio. For the conditions tested, the minimum settlement ratio was about 0.81 for the case of soil at (or near) the ground surface.

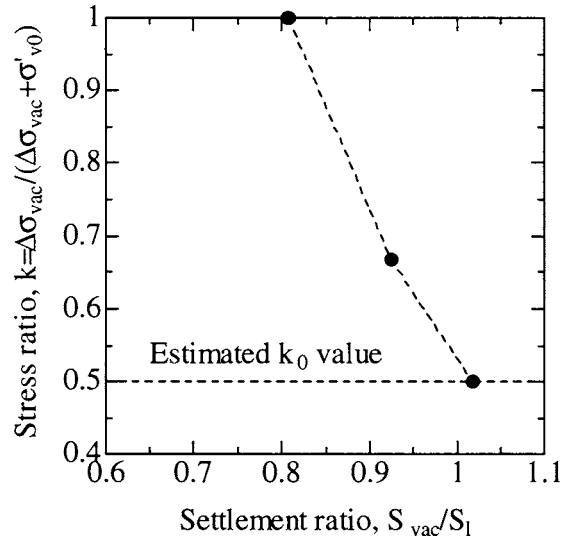

Fig. 5. Stress ratio versus settlement ratio

Field Behavior

Stress State in the Ground under Vacuum Consolidation

In the field, conditions are slightly different from those in an oedometer test. As illustrated in Fig. 6(a), at the ground surface, inward lateral displacement induced by vacuum pressure may cause tension cracks with a depth of z_c. For a soil element located at a depth less than z_c, the stress state can be approximated by that shown in Fig. 6(b), which is the same as in an oedometer test with a vacuum pressure larger than the lateral stress required to maintain a k_o condition. However, below z_c and above z_l, the depth at which no lateral displacement occurs (the value of z_l will be discussed later), the lateral effective stress consists of two parts: One is the vacuum pressure and the other is the earth pressure exerted by the adjacent soil mass. In this zone, the horizontal earth pressure will be between the values corresponding to the at-rest and active states. Denoting the earth pressure coefficient in this zone as k_{ao}, the stress state of a typical soil element will be like that shown in Fig. 6(c). In Figs 6(b and c), the same symbols, σ'_{vo} and $\Delta\sigma_{\mathrm{vac}}$, are used for the initial vertical effective stress and the

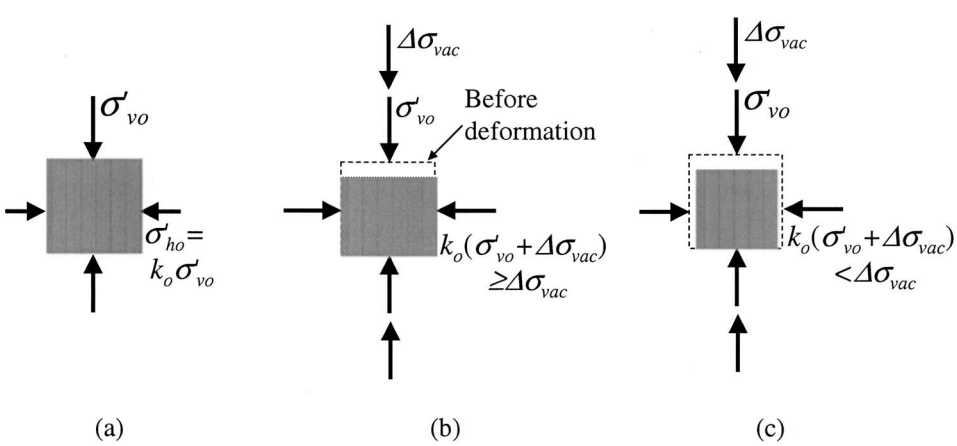

Fig. 4. Illustration of soil element deformation pattern; (a) initial stress state; (b) no lateral displacement; and (c) with lateral displacement

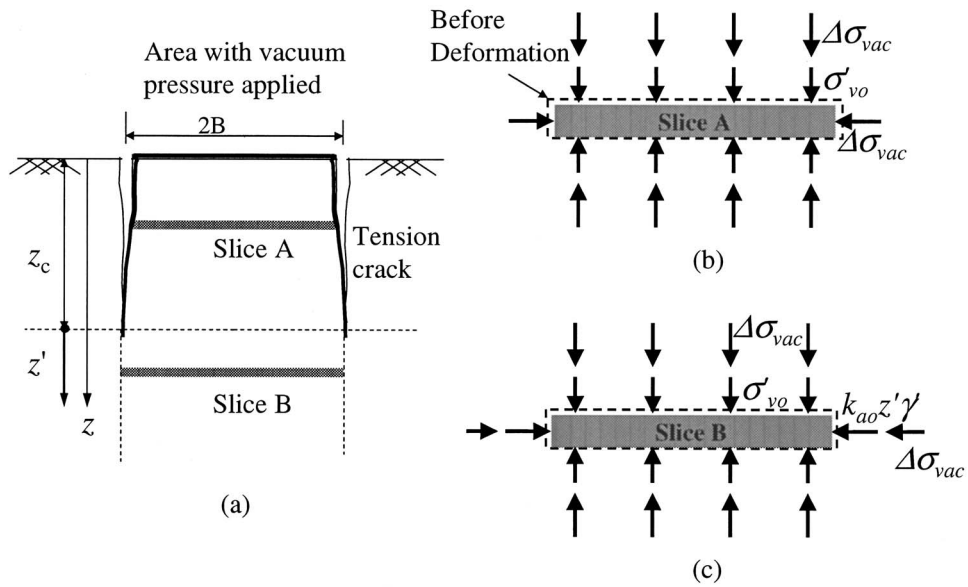

Fig. 6. Illustration of stress state and deformation pattern of soil slices in the ground under vacuum consolidation; (a) location of soil slices; (b) above the depth of tension crack; and (c) below the depth of tension crack;

applied vacuum pressure, but their numerical values can be different in each case. Intuitively, the value of k_{ao} should be close to the active earth pressure coefficient (k_a) at the depth just below z_c, and close to the at-rest earth pressure coefficient (k_o) at the depth just above z_l. Another factor which needs to be considered when determining a reasonable value of k_{ao} is that under field condition, the deeper layers which do not undergo lateral displacement will tend to restrict the inward lateral displacement of the layers above. Considering these factors, an expression is proposed for k_{ao}, as follows:

$$k_{ao} = \beta k_a + (1-\beta)k_o \tag{4}$$

where β=an empirical factor. Based on comparisons between calculated lateral displacements and field measurements (details will be presented later), it is suggested that β should normally be assigned a value in the range from 0.67 to 1.0.

From Rankine earth pressure theory, assuming that the groundwater level is z_w below the ground surface, the depth of cracking z_c can be expressed as follows:

$$z_c = \frac{2c'}{\gamma_t\sqrt{k_a}}, \quad \text{for } z_c < z_w \tag{5a}$$

$$z_c = \frac{1}{(\gamma_t - \gamma_w)}\left(\frac{2c'}{\sqrt{k_a}} - \gamma_w z_w\right), \quad \text{for } z_c > z_w \tag{5b}$$

where γ_t=total unit weight of soil; γ_w=unit weight of pore water; c' and ϕ'=effective stress cohesion and friction angle of the soil, respectively, and $k_a=\tan^2(45-\phi'/2)$=active earth pressure coefficient. Assuming c'=5–10 kPa, $\phi'=30°$, γ_t=15 kN/m^3, and z_w=1.0 m, Eq. (5) predicts z_c=1.47–4.93 m.

Conditions for One-Dimensional Deformation

In Fig. 6(c), if the horizontal stress ($\Delta\sigma_{vac}+k_{ao}z'\gamma'$) is larger than the effective stress required to maintain a k_o stress state, then inward lateral displacement will occur. Equating the effective stress required to maintain a k_o stress state, $k_o(\Delta\sigma_{vac}+\sigma'_{vo})$, and the lateral effective stress, $\Delta\sigma_{vac}+\langle z'\gamma' k_{ao}\rangle$, a condition is obtained for determining the depth below which no lateral displacement occurs in the soil, $z_l=z_c+z'$, i.e.,

$$\Delta\sigma_{vac} = \frac{k_o \cdot \sigma'_{vo} - \sigma'_{av}}{1-k_o} \tag{6}$$

where

$$\sigma'_{av} = \langle z'\gamma' k_{ao}\rangle = \begin{cases} 0 & \text{for } z < z_c \\ k_a z'\gamma' & \text{for } z_l > z > z_c \end{cases} \tag{7}$$

and where γ'=effective unit weight of soil, equal to γ_t above the ground water level and ($\gamma_t-\gamma_w$) below the ground water level. For multilayer soils, σ'_{av} should be calculated using an appropriate summation procedure. With a known value of $\Delta\sigma_{vac}$, the value of z_l can be determined from Eqs. (5) to (7).

The vacuum consolidation method is generally applied to deposits of soft normal to lightly overconsolidated clayey soils, so that at the end of vacuum consolidation the soil should be in a normally consolidated state. Therefore, the value of k_o corresponding to the normally consolidated state should be substituted in Eq. (6), and this value should typically be less than 1.0. In multilayer subsoils, the value of k_o for the layer closest to the depth z_l should be substituted in Eq. (6).

Vertical Deformation

From elasticity theory, the ratio between the incremental vertical strain occurring during one-dimensional (1D) consolidation ($\Delta\varepsilon_{v1D}$) and during isotropic consolidation ($\Delta\varepsilon_{viso}$) can be expressed as follows:

$$\frac{\Delta\varepsilon_{viso}}{\Delta\varepsilon_{v1D}} = \frac{1-\mu}{1+\mu} \tag{8}$$

where μ=Poisson's ratio of the soil skeleton. Adopting a typical value of μ=0.3 provides $\Delta\varepsilon_{viso}\approx 0.54\Delta\varepsilon_{v1D}$. When vacuum pressure is applied over a long strip area (e.g., for road construction), much of the ground will deform under plane strain conditions,

i.e., application of a vacuum pressure will result in a plane strain deformation. Using the symbol $\Delta\varepsilon_{v\text{plane}}$ to represent the vertical strain for this case, the following expression is obtained:

$$\frac{\Delta\varepsilon_{v\text{plane}}}{\Delta\varepsilon_{v1\text{D}}} = 1 - \mu \tag{9}$$

The plane strain condition induces larger vertical strain than the truly isotropic stress condition, i.e., typically $\Delta\varepsilon_{v\text{plane}} \approx 0.7\Delta\varepsilon_{v1\text{D}}$. The vertical strain caused by vacuum consolidation can be expressed as a portion of the vertical strain occurring under 1D consolidation, i.e.,

$$\varepsilon_{vv} = \alpha\frac{\lambda}{1+e}\ln\left(1 + \frac{\Delta\sigma_{\text{vac}}}{\sigma'_{v0}}\right), \tag{10}$$

where e=the voids ratio, λ=the virgin compression index in an e-$\ln p'$ plot (where p' is effective mean stress) and α=a factor with a value less than or equal to unity. It is proposed that α will have a minimum value ($\alpha_{\min}$) at the ground surface and it will be unity when $z \geqslant z_l$ (or $\Delta\sigma_{\text{vac}} \leqslant (k_o \cdot \sigma'_{vo} - \sigma'_{\text{av}})/(1-k_o)$). Based on laboratory oedometer test results (Fig. 4), and assuming a linear variation of α with depth, the following expression for α is obtained, i.e.,

$$\alpha = \alpha_{\min} + \frac{1-\alpha_{\min}}{\Delta\sigma_{\text{vac}}}\left(\frac{k_o\sigma'_{vo} - \sigma'_{\text{av}}}{1-k_o}\right) \text{for } \Delta\sigma_{\text{vac}} \geqslant \frac{k_0 \cdot \sigma'_{v0} - \sigma'_{\text{av}}}{1-k_0} \tag{11}$$

Eqs. (8) and (9) provide $\alpha_{\min}$ values from elasticity theory. For a soft clayey soil deposit, most of the deformation will be plastic not elastic, and the direction of plastic deformation is mainly influenced by the stress state rather than the stress increment. Laboratory oedometer tests yielded a value of $\alpha_{\min}$ of about 0.81 (Fig. 4), which is larger than the values obtained from elasticity theory ($\mu > 0.2$). Assuming that under vacuum consolidation, the volumetric strain is the same as for 1D consolidation, from measured settlements and lateral displacements of a field trial in China, Tang and Shang (2000) calculated the corresponding 1D compression of each layer. Using Tang and Shang's (2000) data, a ratio between the vacuum consolidation-induced compression and the corresponding 1D compression under surcharge loading is about 0.8 for a 2.5 m thick layer at the ground surface.

It is convenient to denote the values of $\alpha_{\min}$ for triaxial stress conditions and for plane strain conditions as $\alpha_{\min-T}$ and $\alpha_{\min-P}$, respectively. It is proposed that $\alpha_{\min-T}$=0.80 and that $\alpha_{\min-P}$ should be larger than $\alpha_{\min-T}$. If it is assumed that both the horizontal and volumetric strains are the same for triaxial and plane strain conditions (the vertical strain is larger for plane strain conditions), it can be shown that $\alpha_{\min-P}=(1+\alpha_{\min-T})/2$, and if $\alpha_{\min-T}$=0.8 then $\alpha_{\min-P}$=0.9. Actually, if the volumetric strain is the same in each case, it is more logical that both the vertical and horizontal strains for plane strain should be larger than for triaxial stress conditions. Hence, it is assumed that $\alpha_{\min-P}$=0.85. $\alpha_{\min-T}$ and $\alpha_{\min-P}$ can now be substituted for $\alpha_{\min}$ in Eq. (11) for the relevant cases of triaxial or plane strain deformation. If the vertical strain is calculated from Eqs. (10) and (11), the settlement can then be calculated by integrating the vertical deformation of each soil layer.

Volumetric Strain under Vacuum Consolidation

The effective stress path experienced by a soil element undergoing vacuum consolidation varies with depth. For a soil element at or near the ground surface, the effective stress path is close to isotropic consolidation, while at deeper locations it is closer to 1D consolidation. Strictly speaking, to calculate the volumetric strain accurately, it is necessary to follow the effective stress path experienced by all soil elements with an appropriate elastoplastic constitutive model. However, for simplicity, we consider a semi-empirical equation for calculating the vertical strain [Eq. (10)] and also assume that under vacuum consolidation (for both triaxial and plane strain conditions), the volumetric strain (ε_{vol}) in the ground is the same as that occurring during 1D consolidation, so that

$$\varepsilon_{\text{vol}} = \frac{\lambda}{1+e}\ln\left(1 + \frac{\Delta\sigma_{\text{vac}}}{\sigma'_{v0}}\right) \tag{12}$$

Horizontal Strain due to Vacuum Consolidation

With known values of the vertical [Eq. (10)] and volumetric strains [Eq. (12)], the average inward (compressive) horizontal strain (ε_h) can be expressed as follows:

$$\varepsilon_h = \frac{1}{2}(\varepsilon_{\text{vol}} - \varepsilon_{vv}), \text{ for triaxial stress conditions} \tag{13}$$

$$\varepsilon_h = (\varepsilon_{\text{vol}} - \varepsilon_{vv}), \text{ for plane strain conditions} \tag{14}$$

Once ε_h is known, the lateral displacement (δ_h) can be approximated quite simply as follows:

$$\delta_h = B \cdot \varepsilon_h \tag{15}$$

where B=half-width of the area treated by vacuum consolidation. Of course, Eq. (15) is a gross simplification of the lateral deformation conditions likely to occur in the field, but as Eq. (13) gives the average lateral strain, Eq. (15) probably provides a reasonable assessment of the overall lateral response. This assertion will be tested later in the paper.

For clarity, the analysis presented above is for normally consolidated soils only. For a lightly overconsolidated deposit, the same type of calculation can be made in two steps. Step 1 is from the in situ stress state to the maximum preconsolidation stress previously experienced by soil, using the unloading-reloading compression index (κ) instead of λ in Eqs. (10) and (12). Step 2 is for stress states from the maximum preconsolidation stress experienced by the soil to the final stress state. The total horizontal strain will be the summation of the values obtained from Steps 1 and 2.

The information needed to calculate the deformations induced by vacuum consolidation with the method proposed in this study is as follows.

(1) Vacuum consolidation conditions. Two parameters are required: The magnitude of the vacuum pressure ($\Delta\sigma_{\text{vac}}$) at each depth and the half-width of the improvement area (B).
(2) Groundwater level and soil parameters. The groundwater level (z_w) and seven soil parameters are required for each soil layer: the total unit weight (γ_t), the initial voids ratio (e), the OCR, the virgin and unloading-reloading compression indices in e-$\ln p'$ space (κ and λ, respectively), the effective stress friction angle (ϕ') and cohesion (c').
(3) Model parameter $\alpha_{\min}$. It is suggested that for triaxial stress conditions, $\alpha_{\min-T}$=0.80 and for plane strain conditions, $\alpha_{\min-P}$=0.85.

It should be noted that the method proposed here does not consider the interaction between soil strata. In particular, the constraining effect of a deeper layer on an overlying layer of soil is

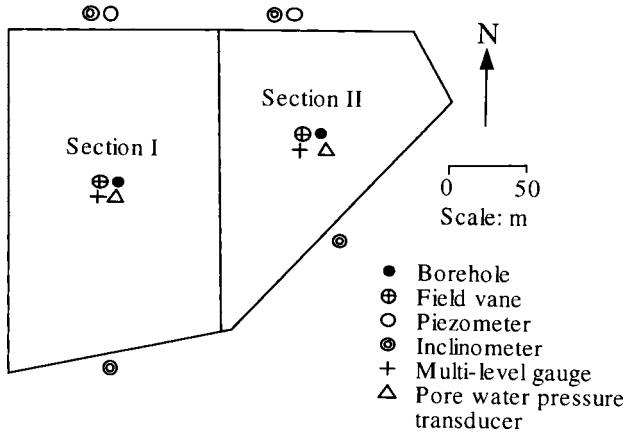

Fig. 7. Plan view of treated area and the location of instrumentation at the Tianjin site

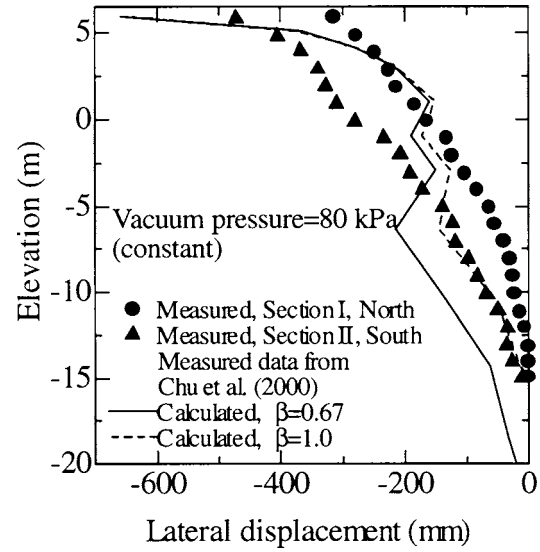

Fig. 8. Comparison of measured and calculated lateral displacements at the Tianjin site, constant vacuum pressure with different β values

not considered. This may result in a predicted zig-zag lateral displacement profile if the compressibility of adjacent soil layers varies significantly.

Analysis of Vacuum Consolidation Field Tests

Vacuum Preloading for an Oil Storage Station at Tianjin, China

This case history was first reported by Chu et al. (2000). The site was reclaimed land and the thickness of the reclaimed soft clay layer was about 4–5 m. Below it was a marine clay layer with an overall thickness of about 10–16 m. This layer can be further divided into a silty clay layer 2–4 m thick, a clay layer 7–8 m thick, and a silty clay layer. The Marine clay layer was underlain by a stiff sandy silt layer. The treated zone covers an area of approximately 50,000 m^2 and for the purpose of ground improvement it was divided into two sections: Section I (about 30,000 m^2) and Section II (about 20,000 m^2) (see Fig. 7, after Chu et al. 2000).

Before vacuum consolidation, partially dried clay fill about 2 m thick (on average) was placed on the site, on top of which was placed a 0.3 m sand mat. Prefabricated vertical drains (PVDs) were then installed on a square grid at spacings of 1.0 m to a depth of 20 m (elevation of about −13.5 m). There is no information about the duration of the fill loading before the vacuum consolidation. Chu et al. (2000) reported that the ground settled about 0.15 m and 0.27 m during installation of the PVDs for Sections I and II, respectively. The vacuum pressure applied at the surface was about 80 kPa and the duration was 4 months (Chu et al. 2000). During application of this vacuum consolidation, the settlement and pore pressure at different depths and the lateral displacement at the edge of the treated area were monitored. From the measured pore water pressures, it was interpreted that the groundwater level was about 1.0 m below the top surface (elevation about 5.0 m) after the 2.0 m thick clay fill and 0.3 m thick sand mat were constructed, i.e., the groundwater level was almost at the ground surface before application of the clay fill. Pore water pressure measurements at the center of Section I indicate that before vacuum consolidation, the consolidation induced by the surcharge load (fill) was not finished. At about −6 m elevation (approximately the middle of the soft clay layer), the measured initial excess pore water pressure was about 20 kPa. However, it is considered that the outward lateral displacement induced by the fill loading was almost finished when vacuum consolidation commenced, i.e., most of the measured lateral displacement was therefore due to vacuum consolidation.

The soil parameters required to calculate the deformation induced by vacuum consolidation are given in Table 3 for Section I. Elevations, unit weights, and void ratios were read from the soil profile figures given by Chu et al. (2000). The values of λ were back-calculated from the measured compression of each soil layer assuming vacuum pressure of 80 kPa and OCR of 1.0. The back-calculation was made assuming 1D deformation condition. Considering that field vacuum consolidation might yield less settle-

Table 3. Soil Strata and Parameters at the Tianjin Oil Storage Site

Number	Description	Depth (m)	Unit weight γ_t (kN/m^3)	Void ratio e	Compression index $(e\text{-}\ln p')\lambda$	Overconsolidation ratio	Effective friction angle ϕ' (°)	Effective cohesion c'(kPa)
1	Fill+sand mat	6.50 to 4.69	19.0	0.85	0.056	1	31	12
2	Soft clay (reclamation)	4.69 to −0.37	17.9	1.15	0.068	1	30	12
3	Silty clay	−0.37 to −4.28	18.3	1.05	0.092	1	31	—
4	Soft clay	−4.28 to −12.57	17.5	1.30	0.252	1	30	—
5	Stiff silty clay	−12.57 to −16.0	19.0	0.85	0.114	1	31	—
6	Sandy silt	−16.0 to −21.0	19.0	0.85	0.114	1	31	—

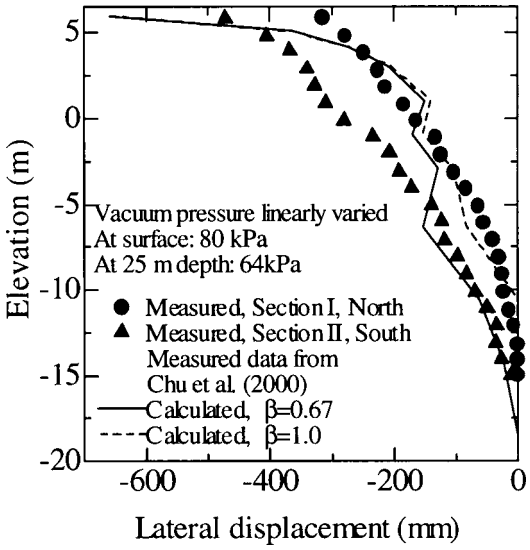

Fig. 9. Comparison of measured and calculated lateral displacements at the Tianjin site, vacuum pressure varied with depth and with different β values

ment than 1D consolidation, the λ values listed in Table 3 are 1.05 times greater than the back-calculated values. Also, it was assumed that the settlement below elevation −12.57 m was due to the compression of the 3.43 m thick stiff silty clay layer and the 5.0 m thick sandy silt underlying it. The measured compression of the clay fill and the sand mat at Section I seems problematic (the amount of compression reduced with elapsed time and was almost zero at the end of vacuum consolidation). Referring to Section II, the compression of the clay fill and the sand mat of 100 mm was assumed in calculations. For this site, although the surcharge fill-induced consolidation may not have finished by the time vacuum consolidation was applied, assuming OCR=1.0 at the commencement of the vacuum treatment should not introduce significant error. Values of the effective stress friction angle (ϕ') and cohesion (c') were assumed. With the parameters listed in Table 3, the depth of tension cracks was calculated as about 2.8 m.

Considering the shape of the treated area, in this case triaxial stress conditions with $\alpha_{\min-T}=0.8$ and a half-width of the vacuum consolidation area of 110 m were adopted in calculations. The effects on the calculated deformation of the ground of the value assumed for β (and therefore the value of k_{ao}) in Eq. (4) and of the variation of vacuum pressure with depth were investigated by comparing predictions with field measurements.

Fig. 8 shows the effect of β varied within the range 0.67 to 1.0, assuming a constant vacuum pressure with depth of 80 kPa. The measured data are from Chu et al. (2000) and correspond to the end of vacuum consolidation. The zig-zag shape of the calculated lateral displacement profile is probably due to a shortcoming of the method which does not consider interaction between soil layers. It can be seen that the value of β mainly influences the lateral displacement at deeper locations and the calculated depth at which the lateral displacement becomes insignificant (z_l). The smaller the β value (and hence the larger the value of k_{ao}), the larger the calculated lateral displacement, and the larger the z_l value. When $\beta=1.0$, $z_l \cong 21.5$ m (elevation of −15.0 m), and when $\beta=0.67$, $z_l \cong 29.5$ m (elevation of −23.0 m). It seems that overall $\beta=1.0$ provides a better simulation of the field data in cases where the vacuum pressure does not decrease with depth. Of course $\beta=1.0$ corresponds to the active earth pressure state and it will obviously underestimate the earth pressure for soil at depths near z_l. However, as mentioned previously, the proposed method does not consider the interaction among soil strata. In field, the deeper layers which do not undergo lateral displacement will tend to restrict the inward lateral displacement of the layers above. Using $\beta=1.0$ indirectly takes into account this restricting effect.

Near the ground surface, the calculation method overestimated the lateral displacement. This is simply because near the ground surface the initial effective stress due to gravity forces alone is assumed in the analysis to be quite low. In reality, prior to application of a vacuum pressure, there may be some suction above the groundwater level which would tend to increase the initial effective stress. In addition, weathering can increase the stiffness of soil at shallow depth. None of these effects was considered in these calculations.

From the measured pore water pressures at locations near the center of Section I, it was calculated that there was no significant vacuum pressure reduction with depth, down to about 20.5 m (elevation −14.0 m). However, under the edge of vacuum pressure improved area, there could be significant vacuum pressure reduction with depth. Assuming that the vacuum pressure varies linearly from 80 kPa at the ground surface to 64 kPa (80% of the value at the ground surface) at a depth of 25 m (elevation −18.5 m), the lateral displacements were also calculated, and are compared in Fig. 9. In this case, for both $\beta=1.0$ and 0.67, the calculated results are comparable with the field data. The corresponding values of z_l are calculated as 17.0 m (elevation −10.5 m) for $\beta=1.0$, and 22.0 m (elevation −15.5 m) for $\beta=0.67$.

Table 4. Comparison of Measured and Calculated Vertical Compression of Soil Strata at the Tianjin Oil Storage Site

Elevation (m)	Soil layer	Vertical compression (mm)				
		Measured	Calculated			
			$\Delta\sigma_{vac}=80$ kPa		$\Delta\sigma_{vac}=80$–64 kPa[b]	
			$\beta=1.0$	$\beta=0.67$	$\beta=1.0$	$\beta=0.67$
6.50 to 4.69	Fill+sand mat	100[a]	87	87	86	86
4.69 to −0.37	Soft clay (reclamation)	154	148	147	145	145
−0.37 to −4.28	Silty clay	113	110	109	105	104
−4.28 to −12.57	Soft clay	417	425	414	392	382
−12.57 to −21.0	Stiff silty clay and sandy silt	167	176	170	148	147

[a]Assumed by referring to the measured value for Section II.

[b]Vacuum pressure linearly varied from 80 kPa at the ground surface to 64 kPa at 25 m depth.

Table 5. Soil Strata and Parameters at the Yaoqiang Airport Site

Number	Description	Depth (m)	Unit weight γ_t (kN/m^3)	Void ratio e	Compression index $(e\text{-}\ln p')\lambda$	Overconsolidation ratio	Effective friction angle ϕ' (°)	Effective cohesion c' (kPa)
1	Silty sand	0–2.5	17.3	0.91	0.046	1	33	12
2	Silty clay	2.5–5.0	19.3	0.87	0.046	1	33	—
3	Silt	5.0–7.5	19.1	0.77	0.036	1	33	—
4	Soft clay	7.5–10.0	18.2	1.35	0.25	1	30	—
5	Silty clay	10.0–16.0	19.8	0.65	0.034	1	33	—

The calculated values of vertical compression of each soil layer are compared with the field measurements in Table 4. Because the values of λ were based on the back-calculated values from the measured compression of each layer, comparisons between measured and calculated values are good. The calculated settlements also varied with the assumed β value, especially for the deeper soil layers. The smaller the value of β (i.e., the larger the k_{ao} value), the larger the predicted lateral displacement (Figs. 8 and 9), and the smaller the predicted compression of the soil layer. Reducing the vacuum pressure with depth reduced the predicted vertical compression of the deeper layers.

Field Trial at Yaoqiang Airport Site, China

The field trial of vacuum consolidation at Yaoqiang Airport was reported by Tang and Shang (2000). The test area was 60 $\times$40 m. The soil at the site consists of alternate layers of silty sand (2.5 m), silty clay (2.5 m), silt (2.5 m), and soft clay (2.5 m). The groundwater level was about 2.5 m below the ground surface. Before the vacuum consolidation treatment was applied, PVDs were installed to a depth of 12 m over a square pattern at 1.3 m spacings, in an attempt to accelerate the consolidation process. At the ground surface, there is a 2.5 m thick silty sand layer. To minimize vacuum pressure loss through this layer, an in situ deep mixing slurry cut-off wall, 1.2 m thick and 4.5 m deep, was constructed around the perimeter of the treated area. The vacuum pressure monitored at the ground surface was 70 to 80 kPa, and the groundwater drawdown was measured as being about 65 kPa at depths of 2 m and 14 m. However, at 8 to 10 m depth, the measured pore water pressure drawdown was only 40 to 50 kPa (Tang and Shang 2000). There was some scatter in the measured pore water pressures. The duration of vacuum consolidation was recorded as 83 days (Tang and Shang 2000). The thickness and soil parameters for each significant layer at this site are given in Table 5. Unit weights and voids ratios are average values of the ranges reported by Tang and Shang (2000). There are no reported values of the virgin compression index λ. The values listed in Table 5 were back-calculated from the reported 1D compression of each layer (Tang and Shang 2000), assuming that the vacuum pressure in the ground was 65 kPa, and OCR was 1.0. Because the PVDs only penetrated to 12 m depth, Tang and Shang (2000) attributed the settlement below 10 m to only a 2 m thick layer (from 10 to 12 m depth) of silty clay. However, because the vacuum pressure measured at 14 m depth was still about 65 kPa, significant compression in a layer from 10 to 16 m was assumed in these calculations. The values of the internal friction angle (ϕ') and cohesion (c') were assumed. The value of c' may seem high for a silty sand layer but, as described previously, an in situ deep mixing slurry cut-off wall was constructed around the vacuum

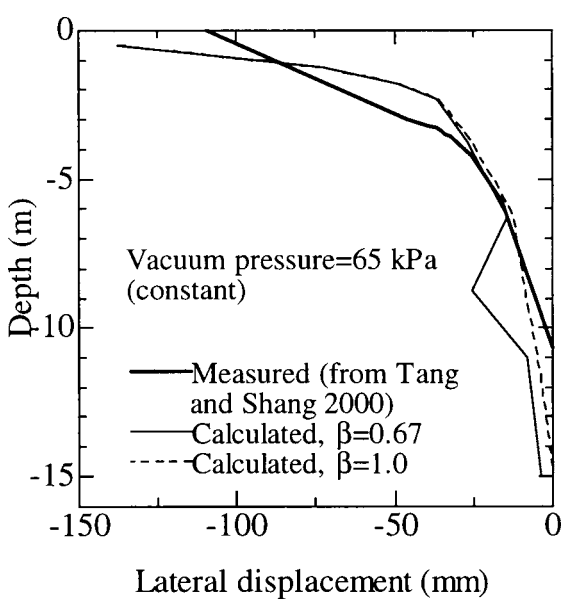

Fig. 10. Comparison of measured and calculated lateral displacement at the Yaoqiang airport site, constant vacuum pressure with different β values

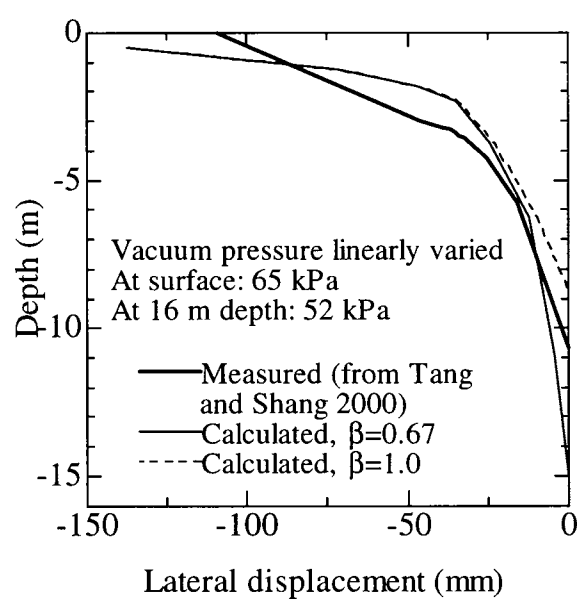

Fig. 11. Comparison of measured and calculated lateral displacement at the Yaoqiang airport site, vacuum pressure varied with depth and with different β values

Table 6. Comparison of Measured and Calculated Compression of Soil Strata at the Yaoqiang Airport Site

Depth (m)	Soil layer	Vertical compression (mm)				
		Measured	Calculated			
			$\Delta\sigma_{vac}$=65 kPa		$\Delta\sigma_{vac}$=65–52 kPa[a]	
			β=1.0	β=0.67	β=1.0	β=0.67
0–2.5	Silty sand	75	83	83	82	82
2.5–5.0	Silty clay	43	44	44	43	42
5.0–7.5	Clayey silt	27	28	28	27	27
7.5–10.0	Soft clay	133	133	131	122	121
10.0–16.0	Silty clay	48	47	46	41	41

[a]Vacuum pressure linearly varied from 65 kPa at the ground surface to 52 kPa at 16 m depth.

consolidation area and the value of c' was therefore selected to take its effect into account. From Eq. (5*a*), the depth of tension crack was calculated as 2.1 m.

Comparison of the measured and calculated lateral displacements at the end of the field test, assuming a constant vacuum pressure (65 kPa) with depth, is shown in Fig. 10. The measurements were made by an inclinometer located at the middle of one of the longer sides of the treated area and 2.5 m outside the edge of this zone. Plane strain conditions were therefore assumed in the calculations, with a half-width of treated area of 20 m and α_{min-P}=0.85. As for the Tianjin site, it seems that assuming β=1.0 yielded a better simulation of the field data. However, it should be noted that the predictions of lateral displacements were made at the edge of the treated zone, while the measurements were recorded 2.5 m away from this edge. The calculated depth (z_l) for no lateral displacement is 14.5 m for β=1.0, and this is larger than the field measurement. This is possibly because the method does not consider the restraining effect of a deeper stiff layer undergoing no lateral displacement on an overlaying layer. For the silty sand layer, the proposed method predicted larger lateral displacements near the ground surface and smaller values near the bottom of the layer, relative to the corresponding field measurements. This is probably because assuming an OCR of 1.0 and using a back-calculated compression index for the whole layer will overestimate the compressibility of the soil near the ground surface (lower initial effective stress), and underestimate the compressibility of the soil near the bottom of the layer.

The calculations for the case in which it was assumed that the vacuum pressure decreased with depth, i.e., 65 kPa at the ground surface and linearly reduced to 52 kPa (80% of the value at the ground surface) at 16 m depth, were also conducted. The results are compared in Fig. 11. It can be seen that as for the Tianjin site, the results for both β=1.0 and 0.67 are comparable with the field data.

A comparison of the calculated and measured vertical compression of each soil layer is given in Table 6. It can be seen that there is good agreement for the constant vacuum pressure (65 kPa) cases because the λ values were back-calculated under the same assumption. For cases of reducing vacuum pressure with depth, the calculated compressions in the deeper layers are smaller than the measured data.

From these comparisons, it is suggested that the value of β to be substituted into Eq. (4) should be within the range from 0.67 to 1.0. However, it is also noted that for the two case histories investigated here, the effective stress cohesions (c') and internal friction angles (ϕ') were assumed values. Further calibration with other field data is required in order to substantiate the recommended β values.

Conclusions

Laboratory oedometer test results show that vacuum pressure induces less or about the same settlement compared to an applied surcharge load of the same magnitude. If the applied vacuum pressure is larger than the lateral stress required to maintain a k_o condition (no horizontal strain), there will be inward lateral displacement and the vacuum pressure will induce less settlement.

In the case of field vacuum consolidation, the confining stress applied to a soil element can be thought of as consisting of two parts: The vacuum pressure and the static earth pressure. Due to vacuum pressure-induced inward lateral displacement, the resulting earth pressure in the ground will be less than the at-rest earth pressure. Assuming a value of the lateral earth pressure coefficient in the ground under vacuum consolidation, somewhere between the active and at-rest values, it is proposed that if the vacuum pressure and earth pressure together are larger than the stress required to maintain the at-rest state, there will be inward lateral displacement. An equation is derived for calculating the depth over which this effect occurs.

Further, assuming that the volumetric strain due to vacuum consolidation is the same as for 1D consolidation with a surcharge load of the same magnitude, an approximate method was proposed for calculating the ground settlement and inward lateral displacement induced by vacuum consolidation.

The proposed method has been applied to two case histories in China reported in the literature, from which it was deduced that the value of the earth pressure parameter β [Eq. (4)] should be in the range from 0.67 to 1.0. It is shown that the proposed calculation method simulated the field measured data reasonably well, suggesting that the method may be useful for the design of vacuum consolidation projects.

Acknowledgment

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LATERAL DISPLACEMENT OF GROUND CAUSED BY SOIL-CEMENT COLUMN INSTALLATION

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Erroneous correction

Recently the writers have found that Eq. (11) given in the original paper is wrong, and it also affect the Eqs (14) and (16). Further due to this problem, the predicted lateral displacement reduces faster with increase of radius distance in elastic zone. To fit the field measured data, an artificial partial "plane strain" effect was introduced (Page 627 of the original paper). After correcting this mistake, this artificial correction is no longer needed. Therefore, Eqs (19), (20) and (21) should not be used anymore. The corrections for Eqs (11), (14) and (16) are given in Table C1. The corrected equations result in a slightly larger lateral displacement than the original equations when the distance from an improve zone is larger than the radius of plastic zone. For the cases reported in the original paper, the corrected equations give a comparable result as that reported in the original paper.

The writers apologize for the inconvenience caused by our mistake.

Table C1 Corrections for Eqs (11), (14) and (16)

Eq. No.	Correct form	Wrong form (in original paper)
Eq. (11)	$\delta = \frac{R_p}{r}\delta_p \qquad (r>R_p)$	$u = \frac{R_p^3}{r^3}\delta_p \qquad (r>R_p)$
Eq. (14)	For $D < R_p$ and $D^2 + L^2 > R_p^2$ $\delta_{xA} = \frac{2D(2R_p+\delta_p)\delta_p}{S\sqrt{4D^2+2R_p\delta_p}} \cdot \tan^{-1}\sqrt{\frac{2(R_p^2-D^2)}{2D^2+R_p\delta_p}} + \frac{2R_p}{S}\delta_p\left(\tan^{-1}\frac{L}{D} - \tan^{-1}\sqrt{\frac{R_p^2}{D^2}-1}\right)$	For $D < R_p$ and $D^2 + L^2 > R_p^2$ $\delta_{xA} = \frac{2D(2R_p+\delta_p)\delta_p}{S\sqrt{4D^2+2R_p\delta_p}} \cdot \tan^{-1}\sqrt{\frac{2(R_p^2-D^2)}{2D^2+R_p\delta_p}} + \frac{R_p^3}{S\cdot D^2}\delta_p(\tan^{-1}\frac{L}{D} - \tan^{-1}\sqrt{\frac{R_p^2}{D^2}-1} + \frac{D\cdot L}{D^2+L^2} - \frac{D\sqrt{R_p^2-D^2}}{R_p^2})$
Eq. (16)	For $D \geq R_p$ $\delta_{xA} = \frac{2R_p}{S}\delta_p(\tan^{-1}\frac{L}{D})$	For $D \geq R_p$ $\delta_{xA} = \frac{R_p^3}{S\cdot D^2}\delta_p(\frac{D\cdot L}{D^2+L^2} + \tan^{-1}\frac{L}{D})$