Vacuum Consolidation and Its Combination with Embankment Loading

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Abstract

The characteristics of vacuum consolidation and the advantages of preloading soft clayey deposits by combining vacuum pressure treatment with embankment loading are discussed. 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 cases where the bottom of the soft clayey deposit is drained (e.g., where the clay overlies a sand or gravel layer), application of a vacuum pressure will cause less consolidation than an equivalent surcharge load because a vacuum pressure can not be applied effectively at the bottom drainage boundary. For this type of subsoil condition it is suggested that if vacuum consolidation is combined with the use of prefabricated vertical drains (PVDs) for ground improvement, the PVDs should not penetrate the entire clayey layer. An equation for calculating the optimum penetration depth has been derived for this case. The advantages of combining vacuum pressure with embankment loading are discussed in terms of increasing the effective surcharge loading, reducing construction time in the case of road construction, and reducing the preloading-induced lateral displacement of the subsoil.

Introduction

For many engineering constructions on soft clayey soils some form of ground improvement is often required. Preloading is a common method used to provide ground improvement. The surcharge pressure required for preloading can be either

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due to the weight of imposed fill material (e.g., an embankment) and/or the application of a vacuum pressure to the pore fluid contained within the soil. Consolidating soft clayey deposits by applying 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. However, vacuum consolidation also has shortcomings, e.g., the applied vacuum is limited by atmospheric pressure and it may cause cracks in the surrounding surface area due to consolidation-induced inward lateral displacement of the ground. Therefore, in some cases, the combination of vacuum pressure and embankment loading may provide better overall ground improvement.

In this paper, the characteristics of vacuum consolidation are first discussed and illustrated with some laboratory test data, and the advantages of combining vacuum pressure with embankment loading are explained in general terms.

**Characteristics of Vacuum Consolidation**

_**Magnitude of vacuum pressure induced settlement.**_ Laboratory oedometer tests under surcharge load and vacuum pressure were conducted and compared to investigate the characteristics of vacuum consolidation. The equipment used was a Maruto Multiple Oedometer Apparatus with a sample size of 60 mm in diameter and typically 20 mm in height. Soil samples used were reconstituted Ariake clay, a very soft clay deposit in Japan. Some of the physical and mechanical properties of the samples are listed in Table 1. The value of the hydraulic conductivity represents the average values in the normally consolidation region.

**Table 1.** Some physical and mechanical properties of the soil samples

<table>
<thead>
<tr>
<th>Soil</th>
<th>Unit weight, $\gamma_t$ (kN/m$^3$)</th>
<th>Liquid limit $W_l$ (%)</th>
<th>Plasticity limit $W_p$ (%)</th>
<th>Void ratio $e_0$</th>
<th>Compression index $C_c$</th>
<th>Hydraulic conductivity $k$ ($10^{-9}$ m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ariake clay</td>
<td>13.9</td>
<td>116.6</td>
<td>57.5</td>
<td>3.63</td>
<td>0.88</td>
<td>1.44</td>
</tr>
</tbody>
</table>

Comparisons of the settlement versus time curves are given in Figs 1(a) and (b) for Ariake clay samples initially subjected, respectively, to 0 or 80 kPa effective vertical stress and allowed to consolidate under one-way drainage conditions. It can be seen that when the initial vertical effective stress is 0, the vacuum pressure-induced settlement is less than that observed under the corresponding surcharge load (Fig. 1(a)). 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. 1(b)). Generally, whether the application of a vacuum pressure will result in the same settlement as a corresponding surcharge load applied under oedometer conditions depends on whether $k_o$ conditions (no horizontal strain) can be maintained in the ground. Under oedometer conditions and with an incremental
Soil sample was first consolidated with a surcharge load of 80 kPa.

(a) $\sigma'_v = 0$
(b) $\sigma'_v = 80$ kPa

Figure 1. Comparison of the settlement-time curves

Figure 2. Vacuum pressure distribution within the ground

vacuum pressure loading, if there is any lateral displacement in the sample, there will eventually be no confining stress applied by the oedometer 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. The condition for inward lateral displacement to occur can be given as follows:

$$\Delta \sigma_{vac} > \frac{k_o \cdot \sigma'_v \cdot \Delta}{1 - k_o}$$  \hspace{1cm} (1)

where $k_o$ is the at-rest horizontal earth pressure coefficient, $\sigma'_v$ is initial (in situ) vertical effective stress, and $\Delta \sigma_{vac}$ is the incremental vacuum pressure.

Effect of drainage boundary condition. For one-dimensional (1D) consolidation problems there are normally two types of drainage boundary conditions: one-way
drainage and two-way drainage (Figs 2(a) and (b), respectively). Consider an ideal 1D situation where it is possible to apply a suction pore pressure at the top surface of the clay layer where drainage is also possible. For a given applied vacuum pressure, the final vacuum pressure distribution in the clay layer will be uniform for the case of one-way drainage, as illustrated in Fig. 2(c). However, in the case of two-way drainage, at the bottom of the clay layer the excess pore pressure is fixed at zero and effectively no vacuum pressure can be applied at this boundary. The vacuum pressure distribution at steady state will be linear with the maximum value at the clay surface and zero at the bottom (Fig. 2(c)). In this case, Darcy’s law implies that the steady state condition will involve uniform upward water flow through the clay layer. It is obvious therefore that vacuum consolidation involving two-way drainage should result in less settlement than one-way drainage. The laboratory oedometer test results for two-way drainage of Ariake clay samples are shown in Fig. 3. It can be seen that the total settlement in the case of two-way drainage is about half the settlement under one-way drainage.

![Figure 3. Drainage boundary condition effect on vacuum consolidation](image)

**Figure 3.** Drainage boundary condition effect on vacuum consolidation

With regard to the degree of consolidation, it is well known that theoretically for similar drainage conditions, the degree of consolidation for both rectangular and triangular initial excess pore pressure distributions is the same. Under vacuum pressure, for both one-way and two-way drainage, water drains only from the top surface of the clay. Therefore, for both cases, the rate of consolidation should be the same.

The above argument is made on the assumption that the clayey deposit is uniform. In the case of a non-uniform deposit, the final vacuum pressure distribution within a deposit with two-way drainage may not be linear. For steady upward water flow in a layered deposit the following condition must be satisfied in order to maintain the continuity of the flow:

\[ i_1 k_{v1} = i_2 k_{v2} = \cdots = i_i k_{vi} \]  \hspace{1cm} (2)
where $i_i$ and $k_{vi}$ = the hydraulic gradient and hydraulic conductivity of the $i$th layer, respectively. A layer with a lower value of $k$ must have a higher $i$ value, and a higher $i$ value implies a larger variation of vacuum pressure within the layer.

In most field applications of vacuum consolidation, the subsoil drainage is improved by installation of prefabricated vertical drains (PVDs). For a clay deposit with two-way drainage, the PVDs should only partially penetrate the clayey layer to prevent vacuum pressures loss through the bottom boundary (Fig. 4). For this case the possible long-term vacuum pressure distribution within the layers is also illustrated in Fig. 4. Conceptually, there is a PVD penetration depth at which the clay layer will exhibit the largest consolidation settlement under a given vacuum pressure applied at the surface. This depth is defined as the optimum PVD penetration depth. At a steady state, the vacuum pressure at the ground surface is $p_{v0}$, and $p_{v1}$ at the base of the PVD-improved zone (Fig. 4). Defining the integration of vacuum pressure with depth as $A$, and applying Eq. (2) (flow continuity), we obtain an expression for $A$ as follows:

$$A = \frac{1}{2} p_{v0} \cdot (H_1 + H) - \frac{1}{2} H \frac{p_{v0} \cdot k_{v1} \cdot k_{v2} \cdot H_1}{k_{v1} \cdot H - (k_{v1} - k_{v2}) H_1}$$

(3)

where $k_{v1}$ and $k_{v2}$ = the vertical hydraulic conductivities of layers 1 and 2, respectively, $H_1$ = the thickness of the PVD-improved zone, and $H$ = the thickness of the soft clayey deposit. Differentiating $A$ with respect of $H_1$ and equating this to zero, we will get an expression for the optimum penetration depth $H_1$ as follows.

$$H_1 = \left(\frac{k_{v1} - \sqrt{k_{v1} k_{v2}}}{k_{v1} - k_{v2}}\right) H$$

(4)

Chai et al. (2001) proposed a method to calculate the equivalent vertical hydraulic conductivity of PVD-improved subsoil, which can be used to evaluate the value of $k_{v1}$, i.e., the mass vertical hydraulic conductivity of the PVD-improved zone:

$$k_{v1} = \left(1 + \frac{2.5l^2}{\mu D_e^2} \frac{k_h}{k_v}\right) k_v$$

(5)

where $D_e$ = the diameter of a unit cell (containing a PVD and its improvement area), $k_h$ and $k_v$ = the horizontal and vertical hydraulic conductivities of the natural soil, respectively, and $l$ ($=H_1$) = the drainage length of the PVDs. The parameter $\mu$ represents the effects of spacing, smear and well resistance, that can be expressed as follows (Hansbo 1981):

$$\mu = \ln \frac{n}{s} + \frac{k_h}{k_s} \ln(s) - \frac{3}{4} + \pi \frac{2l^2 k_h}{3q_w}$$

(6)
where \( n = D_e/d_w \) (\( d_w \) is the diameter of the drain), \( s = d_e/d_s \) (\( d_s \) is the diameter of the smear zone), \( q_w \) = the discharge capacity of the PVDs and \( k_s \) = the hydraulic conductivity of the smear zone. Since \( H_1 \), in Eq. (4), and \( l \), in Eqs (5) and (6), have the same meaning, some iteration is needed to obtain the correct value of \( H_1 \).

**Lateral deformation of subsoil.** Embankment loading will cause both settlement of the underlying soft subsoil and generally outward lateral displacement (Fig. 5(a)). This lateral displacement is mainly caused by the shear stresses induced by the embankment load, and if these shear stresses are big enough they will cause shear failure within the subsoil. By contrast, the vacuum pressure technique tends to apply an isotropic incremental consolidation pressure to the soft subsoil, which will induce settlement and inward lateral displacement (Fig. 5(b)). Even with an isotropic pressure increment, the ground may not respond isotropically because deformation is also influenced by the initial stress state. The inward deformation may cause some surface cracks around the improvement area, but normally there is no possibility of general shear failure. Tang and Shang (2000) reported a set of field data of vacuum pressure-induced inward and embankment load-induced outward lateral displacements.

![Figure 5. Lateral deformation of subsoil](image)

**Benefits of Combining Vacuum Pressure with Embankment Load**

As discussed in the previous section, both embankment and vacuum preloading have certain limitations and an improved preloading effect can be achieved by combining the two methods. The advantages of combined preloading can be discussed in terms of preloading pressure, construction time and lateral deformation of the ground.

**Preloading pressure.** Theoretically, the maximum vacuum pressure that may be applied is one atmosphere (about 100 kPa), but practically achievable values are normally in the range from 60 to 80 kPa (Bergado et al. 1998, Tang and Shang 2000). In cases where the required preloading pressure is larger than about 60 to 80 kPa, application of a vacuum pressure only cannot satisfy this requirement, and combining vacuum pressure with embankment loading can be used to solve the problem. Adopting a preloading pressure larger than the permanent structural load can also eliminate or reduce residual settlements after the structure is constructed.
Construction time. Most soft clay deposits are found in lowland regions. After vacuum preloading, the surface elevation of the treated area will be generally lower than the surrounding area. For road, railway, airport runway, and other civil engineering structures, an embankment will usually have to be constructed on the improved ground. Constructing the embankment during the preloading period not only increases the preloading pressure but should also save construction time for the whole project.

Lateral deformation. In situations where existing structures are adjacent to the preloading area, both outward lateral movement of the treated area induced by embankment loading and inward lateral deformation induced by vacuum pressure are undesirable. To avoid or minimize lateral deformations during the preloading period, it is possible to combine embankment loading with application of a vacuum pressure (Bergado et al. 1998; Tran et al. 2004). From elasticity theory, under plane strain conditions the lateral strain increment \( \Delta \varepsilon_{\text{fill}} \) due to an incremental load of embankment fill \( \Delta \sigma_{\text{fill}} \) is:

\[
\Delta \varepsilon_{\text{fill}} = -\frac{\nu \cdot \Delta \sigma_{\text{fill}}}{E} (1 + \nu) \tag{7}
\]

where \( E \) = Young’s modulus and \( \nu \) = Poisson’s ratio of the soil skeleton. The lateral strain increment \( \Delta \varepsilon_{\text{vac}} \) due to a plane strain-type isotropic stress increment caused by application of a vacuum pressure \( \Delta \sigma_{\text{vac}} \) will be:

\[
\Delta \varepsilon_{\text{vac}} = \frac{\Delta \sigma_{\text{vac}}}{E} (1 + \nu)(1 - 2\nu) \tag{8}
\]

For the desired condition, \( \Delta \varepsilon_{\text{fill}} + \Delta \varepsilon_{\text{vac}} = 0 \) (no lateral strain increment), a relationship between \( \Delta \sigma_{\text{fill}} \) and \( \Delta \sigma_{\text{vac}} \) can be deduced as follows:

\[
\Delta \sigma_{\text{fill}} = \left( \frac{1 - 2\nu}{\nu} \right) \Delta \sigma_{\text{vac}} \tag{9}
\]

In the field, the response of the ground is neither elastic nor isotropic, so Eq. (9) provides only an approximate estimate of the ratio between embankment load and vacuum pressure to minimize lateral soil displacement.

Conclusions

The characteristics of vacuum consolidation and the advantages of preloading soft clayey deposits by combining vacuum pressure treatment with embankment loading have been discussed.
Characteristics of vacuum consolidation. (a) 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. (b) In cases where a sand layer underlies a soft clayey deposit, vacuum pressure treatment will generally cause less consolidation settlement than an equivalent embankment load. For this kind of subsoil condition, if vacuum consolidation is combined with the use of prefabricated vertical drains (PVDs), the PVDs should not penetrate the entire clay layer. An equation for calculating the optimum penetration depth has been presented (Eq. (4)). (c) Vacuum consolidation will induce inward lateral displacement of the subsoil while embankment loading will generally induce outward lateral displacement.

Advantages of combining vacuum pressure with embankment load. This combination can increase the overall effective surcharge load, substantially reduce or limit the preloading induced lateral displacement of the subsoil, and allow shorter construction times, especially for road construction.

References


