CHARACTERISTICS OF VACUUM CONSOLIDATION COMPARING WITH SURCHARGE LOAD INDUCED CONSOLIDATION

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ABSTRACT

The characteristics of vacuum consolidation have been investigated by comparing the consolidation induced by vacuum pressure with that of surcharge load under laboratory odometer condition. The results indicate that: (1) a vacuum pressure induced less or the same settlement as that by a surcharge load with the same magnitude; (2) under a vacuum pressure, two-way drainage resulted in less settlement than that of one-way drainage, but the rate of consolidation is the same for both one-way and two-way drainage because for both cases the water in the soil is only allowed to drain out from the top surface of the samples; (3) for a two-layer system and vacuum pressure applied under two-way drainage conditions, the order of the soil layers not only influenced the rate of consolidation but also the magnitude of settlement.

INTRODUCTION

Vacuum consolidation, as one of the preloading methods, has received more attention recently (Bergado et al. 1998; Tang and Shang 2000; Chu et al. 2000; Chai et al. 2006). Using a vacuum pressure to consolidate a soil deposit 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. However, there are still differing opinions regarding the important characteristics of vacuum consolidation. Mohamedelhassan and Shang (2002) reported that vacuum consolidation can result in settlements nearly identical to those induced by a corresponding surcharge loading applied under odometer conditions. However, Chai et al. (2005) suggested that vacuum consolidation applies an isotropic consolidation pressure increment to a soil and the deformation pattern is different from that under a surcharge load, and it will normally result in less settlement than application of a surcharge load with the same magnitude. Vacuum consolidation is also influenced by the drainage boundary conditions in a different way comparing to a surcharge load. Furthermore, natural deposits are generally layered and the response of a layered deposit under a vacuum pressure is not fully explored yet.

This paper presents the results of a series of laboratory tests involving vacuum pressure and surcharge load induced consolidation under odometer conditions. Both one-layer and two-layer soil systems were tested, and except the settlement measurement, for part of the soil samples, the excess pore water pressure at the bottom of the samples and the lateral earth pressure acting on the samples were measured. The characteristics of vacuum consolidation
are investigated and discussed by comparing with surcharge load induced consolidations in terms of the magnitude of settlement, rate of consolidation and lateral earth pressure variation.

LABORATORY TEST PROGRAM

The equipment used was a Maruto Multiple Odometer Apparatus with a sample size of 60 mm in diameter and 20 mm in height. Figure 1 illustrates the set-up of a soil sample and the locations for pore pressure and horizontal earth pressure measurement. The earth pressure gauge has a diameter of 10 mm and fixed at the middle height of the consolidation ring. The test procedures have been described elsewhere (e.g. Chai et al. 2005).

![Fig. 1 Illustration of the set-up of a soil sample](image)

![Fig. 2 Two-layer system](image)

<table>
<thead>
<tr>
<th>Soils</th>
<th>Soil particles (%)</th>
<th>Unit weight, $\gamma$ (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>31.0 67.8 1.2</td>
<td>13.9 116.6</td>
<td>57.5</td>
<td>3.63</td>
<td>0.88</td>
<td>1.44</td>
<td></td>
</tr>
<tr>
<td>Mixed soil</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.11</td>
<td>0.21</td>
<td>3.13</td>
</tr>
</tbody>
</table>

Two types of soil were used. One was reconstituted Ariake clays, and another was reconstituted mixed soil consisting of 50% the Ariake clay and 50% sand (passing 2 mm sieve) by weight. The soil samples were pre-consolidated under a pressure of 20 - 30 kPa. Some of the physical properties of the soils are listed in Table 1. The values of hydraulic conductivity ($k$) and compression index ($C_c$) were deduced from the standard odometer test results. The listed $k$ values represent average values in the normally consolidation region. At an average effective vertical stress of 100 kPa, the coefficient of consolidation ($C_c$) of the mixed soil sample is about 4 times of that of the Ariake clay sample.

In the case of the two-layer soil system, the bottom of layer-1 is connected to the top of layer-2 as illustrated in Fig. 2, i.e. the value of the excess pore pressure at the bottom of layer-1 is always the same as that at the top of layer-2. For the case of an applied surcharge load, the same amount of load was applied at the tops of both layer-1 and layer-2. However, for the case of a vacuum pressure loading, the desired amount of vacuum pressure was only applied at the top of layer-1. Before the start of each consolidation test, the soil sample was saturated to have a $B$ value (ratio of incremental pore pressure to the corresponding incremental surcharge load applied under undrained conditions) of more than 0.9. Regarding the magnitude of the load applied, in the field, the achievable vacuum pressure is about 60 to 80 kPa (Bergado et al. 1998; Tang and Shang 2000; Chai et al. 2005), so the load increment applied during these tests was 80 kPa for both surcharge load and vacuum pressure.
The tests conducted are listed in Table 2. For the tests with non-zero initial effective vertical stress ($\sigma'_{v0}$), the sample was first consolidated under a predetermined stress for 24 hrs and then an incremental consolidation surcharge load or vacuum pressure was applied.

**TEST RESULTS FOR ONE-LAYER SYSTEM**

**Amount of Settlement**

Comparisons of the settlement/time curves are presented in Fig. 3. It can be seen that when $\sigma'_{v0}$ is low (less than 40 kPa), the vacuum pressure-induced settlements are less than that observed under the corresponding surcharge loads (Figs 3 (a), (b) and (c)). For the cases with $\sigma'_{v0} \geq 60$ kPa, the settlements induced by the vacuum pressure and the surcharge load are almost the same (Fig. 3 (d), (e) and (f)). For tests V1-1, V1-2 and V1-3, when disassembling the apparatus it was observed that the soil samples had separated from the confining ring. This is probably because vacuum pressure applied an isotropic incremental stress to the

### Table 2 Summary of the tests conducted

<table>
<thead>
<tr>
<th>No.</th>
<th>Sample condition</th>
<th>Soils</th>
<th>$\sigma'_{v0}$ (kPa)</th>
<th>Drainage condition</th>
<th>Consolidation pressure</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>V1-1, V1-1a</td>
<td>Ariake clay</td>
<td>0</td>
<td>One-way</td>
<td>Vacuum pressure</td>
<td>80</td>
<td>One-layer</td>
</tr>
<tr>
<td>V1-2</td>
<td></td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>V1-3, V1-3a</td>
<td></td>
<td>40</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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</tr>
<tr>
<td>V1-5, V1-5a</td>
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<td>80</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>V1-6</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>V1-7a</td>
<td></td>
<td>120</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>V1-8</td>
<td></td>
<td>80</td>
<td>Two-way</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1-1, S1-1a</td>
<td>Ariake clay</td>
<td>0</td>
<td>One-way</td>
<td>Surcharge load</td>
<td>80</td>
<td></td>
</tr>
<tr>
<td>S1-2</td>
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<td>20</td>
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<tr>
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<td>S1-5, S1-5a</td>
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<td>S1-7a</td>
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<tr>
<td>S1-8</td>
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<td>80</td>
<td>Two-way</td>
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<tr>
<td>V2-1</td>
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<td>Two-layer</td>
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<td>V2-3</td>
<td>C+M</td>
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<td>V2-4</td>
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<td>Surcharge load</td>
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<td>S2-2</td>
<td>M+C</td>
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<tr>
<td>S2-3</td>
<td>C+M</td>
<td>80</td>
<td>Two-way</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

1) $\sigma'_{v0}$ is initial vertical effective stress.
2) Test number with a letter “a” at the last indicates the test was with horizontal earth pressure measurement.
3) C+M: the first layer is the Ariake clay and the second layer is the mixed soil; M+C: reverse the order of
the soil layers. Sample which tends to induce inward lateral displacement. It is reasoned that generally, whether a vacuum pressure can induce the same settlement as a corresponding surcharge load under odometer conditions depends on whether a $k_0$ condition (no horizontal strain) can be maintained. Under odometer conditions and with an incremental vacuum pressure loading, if there is any inward lateral displacement in the sample, there will eventually be no confining stress applied to the sample by the odometer ring, and the only horizontal stress will be due to the vacuum pressure. Therefore, if the vacuum pressure alone is larger than the confining stress required to maintain a $k_0$ condition, there will be inward lateral displacement and the vacuum pressure will induce less settlement than the surcharge load. Otherwise there will be

![Graphs showing settlement vs. elapsed time for different vacuum pressures](image)

- (a) $\sigma'_{v0}=0$ (after Chai et al. 2005)
- (b) $\sigma'_{v0}=20$ kPa
- (c) $\sigma'_{v0}=40$ kPa (after Chai et al. 2005)
- (d) $\sigma'_{v0}=60$ kPa
- (e) $\sigma'_{v0}=80$ kPa (after Chai et al. 2005)
- (f) $\sigma'_{v0}=100$ kPa
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 (after Chai et al. 2005) and the condition for inward lateral displacement to occur can be written as follows:

\[ \Delta \sigma_{\text{vac}} > \frac{k_0 \cdot \sigma'_{v0}}{1 - k_0} \]

where \( k_0 \) is the coefficient of at-rest horizontal earth pressure, and \( \Delta \sigma_{\text{vac}} \) is the incremental vacuum pressure. Assuming a \( k_0 \) value of 0.5, for the laboratory tests conducted, tests V1-1, V1-2 and V1-3 satisfy the condition of Eq. (1) and inward lateral displacements were observed. Under odometer conditions, denote the ratio of the final settlement induced by a vacuum pressure to that induced by a surcharge load with the same magnitude as \( S_{\text{vac}}/S_l \), the values of \( S_{\text{vac}}/S_l \) are 0.81, 0.89 and 0.92 for \( \sigma'_{v0} = 0, 20 \) and 40 kPa, respectively. Test V1-4 also satisfies Eq. (1), i.e. the vacuum pressure of 80 kPa is larger than the estimated horizontal effective stress 60 kPa required to maintain a \( k_0 \) condition. Although Fig. 3 (d) shows that for both vacuum pressure and surcharge load cases, the settlements are almost the same, digital data give a \( S_{\text{vac}}/S_l \) value of 0.98. For test V1-5, with the assumed \( k_0 \) value, from calculation, \( \sigma'_{v0} \) plus the vacuum pressure was just sufficient to maintain a \( k_0 \) condition of the sample, and so no inward lateral displacement should have occurred and \( S_{\text{vac}}/S_l \) value is almost unity. For V1-6, the vacuum pressure was less than that estimated horizontal effective stress for maintaining a \( k_0 \) condition, and the consolidation ring had confinement on the soil sample and \( S_{\text{vac}}/S_l \) is also almost unity.

**Coefficient of Horizontal Earth Pressure**

To further investigate the effect of initial stress state on the deformation characteristics of vacuum consolidation, a series odometer tests (Test numbers with a letter “a” at the end in Table 2) were conducted with horizontal earth pressure measurement using the reconstituted Ariake clay samples. It is considered that the confining pressure between a soil sample and the confining ring is a good indicator of possible gap between the soil sample and the ring caused by a vacuum pressure. For the equipment used, since a flat surface 10 mm in
diameter earth pressure gauge is inlayed into an arc form wall of the ring, geometrically it forms an unsmooth part on the inner surface of the ring. If using the traditional method of cutting a soil sample and pushed into the ring, a perfect initial contact between the soil sample and the ring may not be easily achieved. To ensure a complete initial contact between a soil sample and the ring, the soil samples were directly consolidated inside the ring with a pressure of 20 kPa and then cut the excessive part to form the test samples.

Comparison of the settlements is summarized in Fig. 5. The settlement results of test V1-5a and S1-5a (σ′v0 = 80 kPa) are questionable and are excluded from this presentation. The same as the results in Fig. 3, with the increase of σ′v0, the settlement induced by a vacuum pressure becomes closer and finally equals to the settlement induced by a surcharge load with the same magnitude. The magnitude of the settlement in Fig. 5 is larger than that in Fig. 3 for a corresponding σ′v0 value because the pressure used to make the reconstituted soil sample was different. The results in Fig. 5 are from the samples reconstituted under 20 kPa, while for the results in Fig. 3, the pressure was 30 kPa.

To calculate the coefficient of earth pressure, the effective stress condition at the earth pressure gauge location (Fig. 1) has to be evaluated. Since the pore pressure is only measured at the bottom of a sample, and therefore, an assumption on the pore pressure distribution within the sample needs to be made. Generally, it is assumed that the pore pressure distributed in a parabolic form in the sample with the following expression.

\[
\Delta u = \Delta u_b \left(1 - \frac{z^2}{h^2}\right) \tag{2}
\]

where \(h\) is the thickness of a sample, \(z\) is the distance from the bottom undrained surface, \(\Delta u_b\) is the measured pore pressure increment at the bottom of the sample, and \(\Delta u\) is the pore pressure increment at \(z\). Then, in case of a surcharge load increment of \(\Delta \sigma_v\), the effective stress increments in the vertical and horizontal directions at the earth pressure gauge location (\(\Delta \sigma_v\) and \(\Delta \sigma_h\)) will be as followings:
\[
\Delta \sigma_v = \Delta \sigma_v - \left(1 - \frac{H_0^2}{4(H_0 - \Delta h)^2}\right) \Delta u_b
\]

(3)

\[
\Delta \sigma_h' = \Delta \sigma_h - \left(1 - \frac{H_0^2}{4(H_0 - \Delta h)^2}\right) \Delta u_b
\]

(4)

where \(\Delta \sigma_h\) is the measured horizontal earth pressure, \(H_0\) is the initial thickness of the sample, and \(\Delta h\) is the vertical settlement of the sample. In case of vacuum consolidation with a vacuum pressure increment of \(\Delta \sigma_{\text{vac}}\) (minus in value), the corresponding effective stress increments at the earth pressure gauge location are:

\[
\Delta \sigma_v' = -\Delta \sigma_{\text{vac}} + \left(1 - \frac{H_0^2}{4(H_0 - \Delta h)^2}\right) (\Delta \sigma_{\text{vac}} - \Delta u_b)
\]

(5)

\[
\Delta \sigma_h' = -\Delta \sigma_h
\]

(6)

Equations (5) and (6) are quite different with Eqs (3) and (4) because in case of a vacuum pressure increment, the measured \(\Delta u_b\) at the bottom of a sample is the consolidated effective stress increment rather than unconsolidated excess pore pressure. Also, the value of \(\Delta \sigma_h\) will be minus in value. Equations (3) – (5) are derived assuming that the earth pressure gauge is located at the middle height of a sample initially. If \(H_0 < 20\) mm (\(\sigma_{v0} > 0\)), the settlement caused by \(\sigma_{v0}\) needs to be included into \(\Delta h\). For \(\sigma_{v0} = 40, 80,\) and \(120\) kPa case, the corresponding \(\Delta h_0 = 1.6, 2.9\) and \(3.7\) mm, respectively.

By the definition, the coefficient of horizontal earth pressure (\(k\)) is the ratio of the effective horizontal earth pressure due to the confinement from the surrounding soil mass (or structure) to the vertical effective stress. \(\Delta \sigma_h'\) in Eq. (6) is not only due to the confinement from the consolidation ring but also the effect of the vacuum pressure. In this case, the confinement increment from the ring is minus with a value of \(\Delta \sigma_h\). Then \(k\) values can be calculated as follows:

\[
k = \frac{\sigma_{h0} + \Delta \sigma_h'}{\sigma_{v0} + \Delta \sigma_v} \quad \text{(Surcharge load)}
\]

(7a)

\[
k = \frac{\sigma_{h0} + \Delta \sigma_h'}{\sigma_{v0} + \Delta \sigma_v} \quad \text{(Vacuum pressure)}
\]

(7b)

The variations of calculated \(k\) values are given in Fig. 6 (a) and (b) for surcharge load and vacuum pressure consolidations, respectively. Generally there is a tendency of reduction of \(k\) value with the elapsed time (\(t\)). For \(\sigma_{v0} = 0\) case, at about 10 min. elapsed time, \(k\) shows a minimum value and which coincides with the highest settlement rate (Fig. 5). As explained previously, inlaying the earth pressure gauge on the wall of the ring formed a part of unsmooth surface on the ring. It is considered that when the deformation rate is high, there might be stress localization and partial arching around the earth pressure gauge and reduced the earth pressure on the gauge. Then, when the deformation stabilized, the \(k\) value reached a stable value also. For the surcharge load case, the final values of \(k\) are about 0.3
regardless the value of $\sigma^\prime_{v0}$ but for the vacuum pressure case, the final $k$ values are negative and the absolute values reduced with the increase of $\sigma^\prime_{v0}$ and for $\sigma^\prime_{v0} = 120$ kPa case, the final $k$ value is very close to 0. The final $k$ values with the corresponding $\sigma^\prime_{v0}$ values are summarized in Fig. 7. The negative $k$ value indicates the possible tendency of forming a gap between the soil sample and the confining ring and therefore, certain lateral displacement of the sample and less settlement than that induced by a surcharge load with the same magnitude as shown in Fig. 5.

Regarding the magnitude of the final $k$ values in Fig. 7, they seem too low. For the soil samples used, an internal friction angle ($\phi^\prime$) of about 30° can be estimated, and in case of surcharge load, using Mayne and Kulhawy’s (1982) equation of $k = 1 - \sin \phi^\prime$, $k = 0.5$ can be evaluated. The reasons considered for the lower $k$ values from the odometer tests are: (a) the effect of friction between the soil sample and the ring (Teerachaikulpanich et al. 2007); and (b) the imperfect geometrical shape at the earth pressure gauge location. However, the $k$ values are used as a direct evidence to show that under vacuum pressure, even under odometer condition, a soil sample can deform horizontally if the value of $\sigma^\prime_{v0}$ in a soil sample is small, and as a result, the less settlement.

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![Figure 6 Variation of the coefficient of earth pressure](image6.png)

(a) Surcharge load  
(b) Vacuum pressure

**Fig. 6** Variation of the coefficient of earth pressure

![Figure 7 Effect of initial effective vertical stress on k value](image7.png)

**Fig. 7** Effect of initial effective vertical stress on $k$ value
Fig. 8 Effect of drainage boundary condition on vacuum pressure induced settlement

Effect of Drainage Boundary Condition

Under odometer conditions, there are normally two types of drainage boundary conditions: one-way and two-way drainage. For a given vacuum pressure, ideally the final vacuum pressure distribution in a soil sample will be uniform for the case of one-way drainage. However, in the case of two-way drainage, at the bottom of the sample the excess pore pressure is fixed at zero and effectively no vacuum pressure can be applied. The vacuum pressure distribution within the soil sample at steady state (end of consolidation) will be linear with the maximum value at the surface and zero at the bottom of the sample (triangular distribution). It is obvious therefore that vacuum consolidation involving two-way drainage should result in less settlement than one-way drainage. In this case, Darcy’s law implies that a steady state condition will involve uniform upward water flow through the sample.

The tests for investigating the effect of drainage boundary condition were conducted under $\sigma'_0 = 80$ kPa to avoid possible gap forming between the soil sample and the consolidation ring and therefore vacuum pressure leakage in case of two-way drainage. The test results are compared in Fig. 8 (V1-5 and V1-8). With the same $\sigma'_0$ value and the same magnitude of applied incremental vacuum pressure, the settlement under the two-way drainage (V1-8) was about the half of that under the one-way drainage (V1-5). Using a linear $e-lnp'$ relation ($e$ is void ratio and $p'$ is consolidation pressure), theoretically the settlement for two-way drainage should be about 55% of that for one-way drainage, and the experimental result is slightly less than this theoretical value.

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

TEST RESULTS FOR TWO-LAYER SOIL SYSTEM

For two-layer system, the tests were conducted under $\sigma'_0 = 80$ kPa. The effects of the order of soil layers on the rate of consolidation, the magnitude of settlement and the pore pressure distribution within the soil samples are studied.
One-way Drainage

As for the one-layer case, with $\sigma'_v = 80$ kPa, the settlements induced by a vacuum pressure are almost the same as those induced by a surcharge load of the same magnitude. Only the results of vacuum consolidation are presented here. The settlement/time curves of two-layer soil systems under vacuum pressure with one-way drainage are compared in Fig. 9 with different orders of the soil layers. It can be seen that, the order of the soil layers did not have an influence on the final settlement. However, the order of the soil layers did have an obvious effect on the rate of consolidation. This phenomenon had been discussed by Pyrah (1996) and Zhu and Yin (1999) for consolidations under a surcharge load. The case where the mixed soil sample was located at the drainage boundary (M+C, V2-2 in Table 2) had a faster consolidation rate. This is because the water in layer-2 could only be drained out through layer-1. The Ariake clay sample had a lower hydraulic conductivity than that of the mixed soil sample, and when it was located at the drainage boundary, it required more time for the water in the mixed soil sample to be drained out. Let’s designate the time for reaching 50% of the final settlement as $t_{50}$. For C+M case (V2-1 in Table 2) $t_{50}$ was about 37 min. and for M+C case (V2-2 in Table 2) was about 28.5 min., and these values will be compared later with those for the two-way drainage cases.

Two-way Drainage

Under vacuum pressure with two-way drainage, the order of the soil layers not only influenced the rate of consolidation but also the magnitude of settlement. The settlement/time curves for two-layer and two-way drainage cases are included in Fig. 9 also. The M+C case (V2-4) had a faster consolidation rate and a larger final settlement. The final settlement was 0.80 mm for the C+M case (V2-3) and 0.94 mm for the M+C case (V2-4), and the values of $t_{50}$ were 43.3 min and 28.5 min, respectively. The values of $t_{50}$ are comparable with the corresponding values of two-layer soil system under one-way drainage conditions (37.0 min and 28.5 min accordingly), and it supports the argument made previously that under vacuum pressure, the rates of consolidation for one-way and two-way drainage should be the same if the coefficient of consolidation is a constant during the consolidation. As discussed for one-layer systems, the final condition under two-way drainage is a steady upward flow in a soil sample, and this condition is applicable to a
two-layer soil system also. At the end of consolidation, to satisfy the flow continuity, the following equation must hold.

\[ i_1 k_{v1} = i_2 k_{v2} \]  

(8)

where \( i_1 \) and \( i_2 \) are the hydraulic gradients in layer-1 and layer-2, and \( k_{v1} \) and \( k_{v2} \) are the vertical hydraulic conductivities of layer-1 and layer-2, respectively. As can be seen from Eq. (8), a layer with a lower \( k \) value must have a higher \( i \) value in order to maintain the continuity of flow. The measured isochrones of the excess pore pressures are given in Fig. 10. It can be seen that for both the C+M and M+C cases, at the end of the consolidation, the variation of excess pore pressure (vacuum pressure) in the Ariake clay layer was larger (higher \( i \) value) than that in the mixed soil layer. A shown in Fig. 10 (b), for M+C case, the \( u \) value was \(-52.0 \) kPa at time \( t = 60 \) min and increased to \(-45.6 \) kPa at \( t = 2880 \) min, which indicate there was a possibility of leakage of vacuum pressure for \( t > 60 \) min due to the possible vacuum pressure induced inward lateral displacement of the samples.

![Fig. 10 Excess pore pressure isochrones (two-layer with two-way drainage, vacuum pressure)](image)

CONCLUSIONS

The characteristics of vacuum consolidation have been investigated by comparing the vacuum consolidation with the surcharge load induced consolidation under odometer condition. Based on the laboratory test results, the following conclusions can be drawn.

Vacuum pressure induced less or the same settlement as surcharge load of the same magnitude. If the initial effective vertical stress in a soil sample is small and a vacuum pressure increment alone is larger than the lateral pressure required to maintain a \( k_0 \) condition (no lateral displacement), there will be inward lateral displacement and the vacuum pressure will induce less settlement than a surcharge load with the same magnitude. This argument is supported by the evidences of both the measured settlements and lateral earth pressures acting on the soil sample during the consolidation.

Under a vacuum pressure, the drainage boundary condition influenced the amount of settlement. A two-way drainage condition resulted in less settlement than that of a one-way drainage. Theoretically, under vacuum pressure, the rate of consolidation is the same for
both one-way and two-way drainage because for both cases, the water in soil samples is only allowed to drain out from the top surface where the vacuum pressure is applied.

For a two-layer soil system with one-way drainage, the order of soil layers only influenced the rate of consolidation but not the final settlement (for both a surcharge load and a vacuum pressure). When a layer with a relative lower value of hydraulic conductivity ($k$) was located at the drainage boundary, the consolidation rate was lower. However, for two-way drainage with a vacuum pressure, the order of the soil layers not only influenced the rate but also the magnitude of the consolidation settlement.

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