GROUND DEFORMATION INDUCED BY COMBINATION OF VACUUM PRESSURE AND SURCHARGE LOAD

Jin Chun CHAI¹ and Chin Yee ONG²

Laboratory large-scale model (length: 1.50m, width: ~0.62m, height: 0.85m) tests and finite element analyses (FEA) were conducted to investigate the deformation characteristics of soft clayey ground under the combination of vacuum pressure and surcharge load. Both the model test and FEA results indicate that the combination of vacuum pressure with surcharge load can reduce the lateral displacement of the model ground. The outward lateral displacement increases with the increase of the loading rate ($LR$) of surcharge load and the ratio of surcharge load to vacuum pressure ($RL$). The results also indicate that by adjusting $LR$ and $RL$, the lateral displacement of the ground can be minimized.

Keywords: Geosynthetics drain, vacuum consolidation, lateral displacement, and finite element analysis

1. INTRODUCTION

Prefabricated vertical drain (PVD) is normally combined with embankment loading, vacuum pressure or combination of both to accelerate the consolidation process of clayey ground. Vacuum preloading can result inward lateral ground displacement and cause cracks adjacent to the treatment area, while embankment preloading alone will result outward lateral ground displacement (Shang et al., 1998; Chu et al., 2000; Chai et al., 2006). Conceptually, it is possible to minimize the ground lateral displacement by combining surcharge load with vacuum pressure. There should be an optimum balance between vacuum pressure and surcharge preloading to maintain the lateral displacements close to zero (Indraratna et al., 2007). However, there is no sound theoretical base, laboratory or field data to support this proposal.

In this study, the laboratory model tests as well as numerical simulations were conducted to study the characteristics of the lateral deformation of soft clayey ground under the combination of vacuum pressure and surcharge load. The test conditions will be present first, followed by the finite element analyses (FEA) of simulating the model tests as well as parametric studies. Finally, the results are compared and the discussions are made on the main factors influencing the lateral displacement.

2. LABORATORY MODEL TESTS

(1) Model Box and the Material Used

The inner dimensions of the model box are: 1.50 m in length, about 0.62 m in width and 0.85m in height. The model box is made of steel except the front and the back face walls are transparent Acrylic glass. The test set-up is illustrated in Fig. 1. Along the longitudinal direction of the model, a 15 mm thick Acrylic glass was fixed in the middle of the box to create 2 independent 0.30 m wide model ground. The loading system consists of 3 Bellofram cylinders and 3 steel loading plates (length: 0.29 m, width: 0.166 m, thickness: 0.02 m). To simulate the embankment load, the load applied to two side loading plates was half of the value of the center one. The technique of applying vacuum pressure is called vacuum-cap drain method (Chai et al., 2008). The vacuum pressure is applied to each prefabricated vertical drain (PVD) and using a surface clayey soil layer as air-sealing layer. During the tests, 3 settlement gauges were set on the loading plates to measure the settlements and 2 piezometers, $P_1$ and $P_2$, were installed at 0.25 m and 0.50 m from the bottom of the model ground respectively to monitor the excess pore water pressure variations. Beside these, a 0.02 m x 0.02 m filter paper grid was attached to the inner face of the transparent Acrylic glass wall by grease to monitor

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weight: 130.6 g/m²) were placed at the bottom of the model as drainage layer. Next, filter paper for measuring the lateral displacement was attached to the transparent Acrylic glass wall. Then, the mixed Ariake clay soil with a water content of about 150% was placed into the model layer by layer. The piezometers, \( P_1 \) and \( P_2 \) were installed at predetermined locations. After the thickness of Ariake clay reached 0.80 m, 3 layers of geotextiles were placed on the top of the model ground as top drainage layer. The model ground was pre-consolidated by 10 kPa dead load under two-way drainage condition for about 60 days. Then, the pre-consolidation pressure was removed and 2 independent model ground (width: 0.30 m, thickness: ~0.65 m) were created. Before installing the mini-PVD, the cone (20 mm in diameter) penetration tests were conducted to investigate the strength of the model ground. The undrained shear strength \( (S_u) \) was estimated as follows:

\[
S_u = \frac{q_c - \sigma_v}{N_{kt}} \tag{1}
\]

where \( q_c \) = tip resistance; \( \sigma_v \) = total vertical stress; and the factor \( N_{kt} = 10 \) was adopted (Tanaka, 2000).

The variations of \( S_u \) with depth are shown in Fig. 2. For model ground (MG)-1, the test was conducted immediately after the pre-consolidation, but for MG-2, the test was conducted about 3 weeks after the consolidation and it showed a higher strength. According to plasticity theory (Terzaghi et al., 2008), the estimated ultimate bearing capacity \( (q_u) \) of the model ground was about 5.14 - 25.7 kPa.
b) Installation of Mini-PVD

The mini-PVDs were installed into the model ground by a stainless steel rod. After the mini-PVD reached the desired depth, the steel rod was removed and the mini-PVD was left in the model ground. The mini-PVD was installed in such a way that there was 0.1 m at the bottom and at the surface of the model ground without mini-PVD. The mini-PVDs were arranged in a rectangular pattern with 0.16 m x 0.10 m spacing as shown in Fig. 1 (b). To avoid air leakage through the hole created during the installation, the hole was sealed with the slurry of Ariake clay and bentonite mixture.

(3) Cases Tested

The cases tested and the conditions adopted are listed in Table 1. In the table, $RL$ means the ratio of surcharge load to vacuum pressure.

<table>
<thead>
<tr>
<th>Case</th>
<th>Surcharge Load (kPa)</th>
<th>Vacuum Pressure (kPa)</th>
<th>Ratio of Load, $RL$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>40</td>
<td>-40</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>60</td>
<td>-40</td>
<td>1.5</td>
</tr>
</tbody>
</table>

For both cases, 40 kPa of vacuum pressure was applied within 2 hours. For Case-1, after the application of the vacuum pressure, 4 kPa of surcharge load was applied on the center loading plate (2 kPa on the both side loading plates) in the first day. In the following days, surcharge load was applied at a rate of 6 kPa/day (at the center) up to 40 kPa. While, for Case-2, surcharge load was applied at a rate of 6 kPa/day at the center up to 60 kPa.

3. FINITE ELEMENT MODELING

The laboratory model tests were simulated by finite element analyses (FEA) first and then the numerical parametric study were performed. The program used was a modified form of the original CRISP program (Britto and Gunn, 1987). The finite element mesh and the boundary conditions are shown in Fig. 3. For the displacement boundary conditions, both vertical and horizontal displacements were fixed at the bottom; while the horizontal displacement was fixed at the left and right vertical boundaries. For the drainage boundary conditions, both the top and the bottom of the model were drained while the left and right boundaries were undrained. The effect of mini-PVD was modeled by the 1-D plain strain drainage elements (Chai et al. 1995). The loading condition of laboratory test was simulated closely.

The behavior of soft clayey soil was simulated by modified Cam clay model (Roscoe and Burland 1968). The adopted parameters are listed in Table 2. The hydraulic conductivities listed in Table 2 were initial values, and during the consolidation, they were allowed to vary with void ratio ($e$) according to Taylor’s equation (Taylor 1948):

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

where $k_0 =$ initial hydraulic conductivity, $e_0 =$ initial void ratio, and the constant $C_k$ was adopted as 0.4$e_0$. For mini-PVD, the adopted values of the parameters related to mini-PVD performance were: unit cell diameter $D_e = 0.175$ m, diameter of drain $d_w = 0.024$ m, and discharge capacity of 1 m$^3$/year. No smear effect was considered.

The initial stress state and the size of the initial yield locus adopted are listed in Table 3. The 4 kPa isotropic initial effective stress was added to the gravity force to simulate the effects of thixotropic hardening and partial suction after removing the pre-consolidation load. The initial yielding loci were assigned in a manner that the simulated $S_u$ values of the soil match the tested values. The simulated $S_u$ values are included in Fig. 2.

4. MODEL TEST AND FEA RESULTS

Figures 4 (a) and (b) present the comparison of settlement curves at the center of Case-1 and Case-2 respectively. In both of the cases, the simulated results have higher rate of settlements and larger final settlements than the measured ones. There are two possible reasons: (1) vacuum pressure leakage during the tests, especially for Case-2 as shown in Fig. 5 (b), (2) partially unloading during the test. The partial unloading was occurred due to the
Table 2. Parameters for finite element analysis.

<table>
<thead>
<tr>
<th>Description</th>
<th>$v$</th>
<th>$\kappa$</th>
<th>$\lambda$</th>
<th>$M$</th>
<th>$e_o$</th>
<th>$\gamma_t$</th>
<th>$k_h$</th>
<th>$k_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clayey Soil</td>
<td>0.3</td>
<td>0.03</td>
<td>0.3</td>
<td>1.2</td>
<td>3.39</td>
<td>13.5</td>
<td>6.10E-5</td>
<td>6.10E-5</td>
</tr>
</tbody>
</table>

**Note:** $v$, Poisson’s ratio; $\kappa$, slope of unloading-reloading line in $e$-$\ln p'$ plot; $\lambda$, slope of consolidation line in $e$-$\ln p'$ plot ($p'$ is effective mean stress); $M$, slope of critical state line in $q$-$p'$ plot ($q$ is deviator stress); $e_o$, initial void ratio; $\gamma_t$, total unit weight; $k_h$, hydraulic conductivity in horizontal direction; $k_v$, hydraulic conductivity in vertical direction.

stroke of the loading piston reached the limit and failed to notice it in an early time. As for the rate of settlement, the parameters estimated for mini-PVD may not represent the actual conditions. In the analysis, a constant discharge capacity ($q_w$) was assumed but the result of laboratory long-term discharge capacity test on PVD showed that $q_w$ reduced significantly with elapsed time (Chai and Miura, 1999). For both of the cases, the variations of excess pore pressure are compared in Figs 5 (a) and (b). For Case-1, at $P_1$ location the simulated results are less than the measured values. At $P_2$, the agreement between the simulated and measured one is good. As for Case-2, at $P_2$ location, the measured values are considerably less than the simulated ones. But at $P_1$ location, the agreement is reasonably good. The possible reason for this can be the vacuum leakage occurred during the test.

Figures 6 (a) and (b) present the comparison of simulated and measured final lateral displacement profiles under the edge of loading plates. The lateral displacements observed from the movement of the filter paper grid are not symmetric, especially for Case-1, the difference is large. Generally, the simulation under-predicted the lateral displacements of the model ground. Nevertheless, both measured and FEA results indicated that the lateral displacement increased with the increase of ratio of surcharge load to vacuum pressure ($RL$). As mentioned previously, the undrained bearing capacity of the model ground was 5.14 - 25.7 kPa,

Table 3. Initial stresses in the clayey soil

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$\sigma_{ho}'$ (kPa)</th>
<th>$\sigma_{vo}'$ (kPa)</th>
<th>$\sigma_{ho}'$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>4.00</td>
<td>4.00</td>
<td>12.7</td>
</tr>
<tr>
<td>0.10</td>
<td>4.37</td>
<td>4.37</td>
<td>12.7</td>
</tr>
<tr>
<td>0.20</td>
<td>4.74</td>
<td>4.74</td>
<td>9.7</td>
</tr>
<tr>
<td>0.30</td>
<td>5.11</td>
<td>5.11</td>
<td>6.4</td>
</tr>
<tr>
<td>0.40</td>
<td>5.48</td>
<td>5.48</td>
<td>6.3</td>
</tr>
<tr>
<td>0.50</td>
<td>5.85</td>
<td>5.85</td>
<td>8.6</td>
</tr>
<tr>
<td>0.60</td>
<td>6.22</td>
<td>6.22</td>
<td>14.0</td>
</tr>
<tr>
<td>0.65</td>
<td>6.41</td>
<td>6.41</td>
<td>17.2</td>
</tr>
</tbody>
</table>

**Note:** $\sigma_{ho}'$ and $\sigma_{vo}'$, initial effective stress in horizontal and vertical directions respectively.
under the conditions of 40 kPa vacuum pressure and 40 - 60 kPa surcharge load, there was no any sign of bearing capacity failure problem of the model ground.

Since the conditions investigated by the model tests are limited, the effect of loading rate \( LR \) of surcharge load as well as \( RL \) on the lateral displacement was investigated by FEA.

5. PARAMETRIC STUDIES BY FEA

(1) Effect of \( LR \)

The analyses were conducted under 40 kPa vacuum pressure and 40 kPa surcharge load. The adopted \( LR \) values in the analyses were 3 kPa/day, 6 kPa/day, and 12 kPa/day at the center. The simulated settlement-time curves are presented in Fig. 4. With the increased of \( LR \), there is some increase on final settlement which is considered due to the increase of lateral displacement of the model ground as shown in Fig. 8. Figure 8 shows that inward lateral displacement occurred when the \( LR \) is 3 kPa/day, while outward lateral displacement occurred when the \( LR \) is 12 kPa/day. Larger \( LR \) means the ground condition behaves closer to undrained condition during loading and larger lateral displacement occurs.

(2) Effect of \( RL \)

The analyses were conducted under the conditions that 40 kPa vacuum pressure was fixed and surcharge load was varied from 20 kPa to 80 kPa \(( RL = 0.5\sim2.0)\) and \( LR \) was 6 kPa/day. The simulated settlement-time curves are given in Fig. 9. For the conditions adopted, increasing \( RL \) means
Fig. 6 Comparison of final lateral displacement profiles

Fig. 7 Settlement curves at different LR

Fig. 8 Final lateral displacement profiles at different LR

Fig. 9 Settlement curves at different RL

Fig. 10 Final lateral displacement profiles at different RL

(a) Case-1

(b) Case-2
increasing total load, and as a result the final settlement is increased with $RL$. The simulated final lateral displacement profiles are depicted in Fig. 10. The maximum lateral displacement varied from about 3 mm inward for $RL = 0.5$ to about 8 mm outward for $RL = 2.0$. With the fixed amount of vacuum pressure, increasing $RL$ means increasing the magnitude of surcharge load which mobilizing the shear stress level (shear stress/shear strength) in the ground, and as a result there will be more lateral displacement.

6. CONCLUSIONS

Both the laboratory test and finite element analysis (FEA) results indicated that the combination of vacuum pressure with surcharge load can reduce the lateral displacement of the model ground. The detailed conclusions are as follows:

(1) Define the ratio between surcharge load to vacuum pressure as $RL$, the outward lateral displacement increased with the increase of $RL$. Model test results show that for $RL = 1.0$, the lateral displacement was very small. Further, the FEA results indicate that for $RL$ from 0.5 to 2.0 (40 kPa vacuum pressure, and surcharge load varied from 20 to 80 kPa), the maximum lateral displacement under the edge of the loading plates varied from about 3 mm inward to about 8 mm outward.

(2) Under the condition that the vacuum pressure is applied in a short time (few hours) and $RL = 1.0$, the lateral displacement increased with the increase of loading rate ($LR$) of the surcharge load. FEA results show that for $LR$ varied from 3 kPa/day to 12 kPa/day, the maximum lateral displacement varied from about 3 mm inward to about 2 mm outward.

(3) The model test results also indicate that with the combination of vacuum pressure and surcharge load, it is possible to prevent the bearing capacity failure.

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REFERENCES


真空圧と載荷荷重の組合せによる地盤の変形特性

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真空圧と載荷荷重（盛土）を組合せた場合の粘性土地盤の変形特性について室内大型モデル（長さ：1.50m、幅：〜0.62m、高さ：0.85m）試験及び有限要素法解析（FEA）によって、検討した。試験及び解析結果から、真空圧と載荷荷重の併用により、地盤の側方変位が抑制できることが分った。載荷荷重と真空圧の比（LR）及び載荷荷重の載荷率（RL）の増加に伴い、地盤の側方変位量外向きは増加することが明らかになった。さらに、LRとRLの調整によって地盤の側方変位を最小にすることが可能であることを示した。