Effects of Shear Rate and Consolidation Time on Undrained Behavior of Natural Sedimentary Clay

1. Introduction

The shear rate effects are significant for undrained shear behavior of clay (for example: Bjerrum, 1972; Vaid et al, 1977; Graham et al, 1983; Oshima et al, 1991; Di Benedetto et al, 1997). The undrained shear strength, for instance, increases with the shear rate and also with the sustained consolidation period prior to undrained shear. Moreover, the stress-strain behavior of a wide spectrum of geomaterials may be characterized with Isotach properties (see, Tatsuoka, 1999). In engineering practices, it is thus important to take account of such rate effects when applying the undrained strength from laboratory tests to in-situ problems such as the embankment on soft ground.

In this paper, the shear rate effects of Holocene clay from Kobe airport site (Hasegawa et al, 2006, Fujiwara
et al, 2008) were thoroughly examined in the laboratory in order to evaluate the short-term stability of sea-wall structure resting on soft clay at Kobe airport.

2. Experimental Methods

Fig. 1 shows the triaxial apparatus (for details, see Shibuya et al., 2001). In this apparatus, the specimen having the size of 50 mm in diameter and 100 mm in height is subjected to consolidation and shear in a fully automated manner. Fig. 2 shows the DSB apparatus developed at Kobe University. Like the triaxial apparatus, the vertical as well as the horizontal displacement of the disk-shaped specimen with 60 mm in diameter and 40 mm in height can each be controlled by using a digital servo-motor (Shibuya et al., 2005). Constant-volume conditions can be readily achieved by maintaining the vertical movement zero during the shear. The vertical load is measured at the bottom of the lower shear box, implying that the vertical stress is free from any frictions between the soil specimen and the shear box wall (Shibuya et al, 1997). In all the tests performed in this study, the clearance between the upper and lower shear boxes was maintained...
constant at 0.2 mm.

The clay samples were retrieved from Holocene deposit underneath Kobe airport (Fujiwara et al., 2008) by using a fixed-piston thin-wall sampler. Fig. 3 shows the depth-profile of basic properties of the Holocene clay deposit. The fully saturated natural specimens were each consolidated to the prescribed stress, and it was sheared under undrained conditions in triaxial test and constant-volume (i.e., undrained) conditions in DSB test, respectively. Tables 1 and 2 show the details of the triaxial and DSB tests performed, respectively. It should be mentioned that the end of primary consolidation was judged based on the 3t-method (JGS, 1979). However, in some tests, the consolidation period was longer than the standard so that the effects of consolidation period may be examined.

Table 1. Conditions of undrained triaxial test

<table>
<thead>
<tr>
<th>ID</th>
<th>K.P (m)</th>
<th>γt (kN/m³)</th>
<th>σw0 (kPa)</th>
<th>Initial water content (%)</th>
<th>Consolidation conditions</th>
<th>Shear speed (%/min)</th>
<th>Creep–strain–rate Δεv (×10⁴ %/min)</th>
<th>B value</th>
</tr>
</thead>
<tbody>
<tr>
<td>KTAC1</td>
<td>23.5</td>
<td>16.04</td>
<td>180</td>
<td>55.8</td>
<td>200</td>
<td>1629(3.11)</td>
<td>1.683</td>
<td>0.95</td>
</tr>
<tr>
<td>KTAE1</td>
<td>25.5</td>
<td>15.50</td>
<td>163</td>
<td>55.9</td>
<td>2264(9.71)</td>
<td>2.385</td>
<td>0.96</td>
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<tr>
<td>KTAC2</td>
<td>32.5</td>
<td>16.60</td>
<td>162</td>
<td>53.5</td>
<td>400</td>
<td>1520(11.4t)</td>
<td>7.070</td>
<td>0.95</td>
</tr>
<tr>
<td>KTAE2</td>
<td>26.5</td>
<td>17.17</td>
<td>172</td>
<td>57.5</td>
<td>2732(7.5t)</td>
<td>2.018</td>
<td>0.96</td>
<td></td>
</tr>
<tr>
<td>KTAC3</td>
<td>35.5</td>
<td>16.51</td>
<td>181</td>
<td>52.0</td>
<td>1426(2.9t)</td>
<td>7.717</td>
<td>0.95</td>
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<tr>
<td>KTAE3</td>
<td>25.5</td>
<td>16.11</td>
<td>163</td>
<td>53.8</td>
<td>2662(4.4t)</td>
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<td>KTAC4</td>
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<td>175</td>
<td>54.1</td>
<td>2620(15.7t)</td>
<td>2.600</td>
<td>0.98</td>
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<tr>
<td>KTAC5</td>
<td>34.5</td>
<td>16.29</td>
<td>175</td>
<td>53.3</td>
<td>1012(58.4t)</td>
<td>1.494</td>
<td>0.95</td>
<td></td>
</tr>
<tr>
<td>KTAE5</td>
<td>35.5</td>
<td>16.37</td>
<td>181</td>
<td>54.9</td>
<td>10082(18.9t)</td>
<td>3.528</td>
<td>0.95</td>
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<tr>
<td>KTAC6</td>
<td>38.5</td>
<td>16.02</td>
<td>198</td>
<td>55.5</td>
<td>2406(4.8t)</td>
<td>2.283</td>
<td>0.97</td>
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</tr>
<tr>
<td>KTAC7</td>
<td>39.5</td>
<td>16.15</td>
<td>198</td>
<td>56.5</td>
<td>2702(5.4t)</td>
<td>2.333</td>
<td>0.95</td>
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<tr>
<td>KTAE6</td>
<td>24.5</td>
<td>16.42</td>
<td>182</td>
<td>59.2</td>
<td>2408(4.8t)</td>
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<tr>
<td>KTAE7</td>
<td>24.5</td>
<td>16.37</td>
<td>182</td>
<td>59.6</td>
<td>2439(4.9t)</td>
<td>3.403</td>
<td>0.97</td>
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</table>

Table 2. Conditions of constant-volume direct shear box tests

<table>
<thead>
<tr>
<th>ID</th>
<th>K.P (m)</th>
<th>γt (kN/m³)</th>
<th>σvd (kPa)</th>
<th>Initial water content (%)</th>
<th>Consolidation conditions</th>
<th>Consolidation time (min)</th>
<th>Shear speed (mm/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>KD1</td>
<td>28.5</td>
<td>15.89</td>
<td>174</td>
<td>55.0</td>
<td>200</td>
<td>1433(611)</td>
<td>0.1</td>
</tr>
<tr>
<td>KD2</td>
<td>26.5</td>
<td>16.18</td>
<td>172</td>
<td>55.7</td>
<td>400</td>
<td>1486(231)</td>
<td>0.02</td>
</tr>
<tr>
<td>KD3</td>
<td>28.5</td>
<td>16.33</td>
<td>174</td>
<td>52.6</td>
<td>500</td>
<td>1483(321)</td>
<td>1</td>
</tr>
<tr>
<td>KD4</td>
<td>24.5</td>
<td>16.02</td>
<td>182</td>
<td>57.4</td>
<td>400</td>
<td>1606(271)</td>
<td></td>
</tr>
<tr>
<td>KD5</td>
<td>29.5</td>
<td>16.06</td>
<td>172</td>
<td>55.4</td>
<td>400</td>
<td>4321(721)</td>
<td></td>
</tr>
<tr>
<td>KD6</td>
<td>29.5</td>
<td>15.83</td>
<td>172</td>
<td>56.8</td>
<td>10083(1511)</td>
<td>39894(1981)</td>
<td></td>
</tr>
<tr>
<td>KD7</td>
<td>29.3</td>
<td>16.64</td>
<td>174</td>
<td>48.0</td>
<td>39894(1981)</td>
<td>1457(221)</td>
<td></td>
</tr>
<tr>
<td>KD8</td>
<td>25.5</td>
<td>15.92</td>
<td>163</td>
<td>55.6</td>
<td>400</td>
<td>1457(221)</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 4. Variation of void ratio, e, with time during consolidation as examined in triaxial tests

3. Experimental Results

Figs. 4 and 5 show the variation of void ratio, e, with...
time during consolidation as examined in triaxial and DSB tests, respectively. The stress-strain relationship of a series of triaxial tests using the axial strain rate of 0.02%/min is shown in Fig. 6. The undrained effective stress paths are shown in Fig. 7, in which the deviator stress, $q$, is plotted against the mean effective stress, $p'$.

The results of the constant-volume DSB tests are shown in Fig. 8, in which the variation of the horizontal shear stress, $\tau$, is examined against horizontal displacement and the vertical stress. Despite that the specimens were all normally consolidated, the stress-strain relationship exhibited softening behavior when sheared undrained.

4. Discussions

4.1 Effects of Consolidation Period

Figs. 9 and 10 show the results of triaxial tests performed using different consolidation periods. The consolidation time effects were more significant in compression tests for the peak strength increased with the consolidation period.

The trend may be attributed to the creep-strain-rate effect during the anisotropic consolidation (see Table 1), which considerably expanded the elastic region on compression side.

The effect of consolidation period on the undrained shear (i.e., peak) strength, $S_u$, is examined in Fig. 11. Noting that the ratio of $S_u$ to the consolidation stress, $\sigma_{cc}$, increased with the consolidation time, $t_{total}$ normalized using the time to reach the end of primary consolidation, $t_{3t}$, the consolidation time effect may be expressed in the
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Fig. 8. Results of constant-volume DSB tests with the horizontal displacement rate of 0.1 mm/min

Fig. 9. Stress–strain relationship of a series of triaxial tests using different consolidation times

(a) $\tau - \sigma_{vc}$ relationship
(b) $\tau - \Delta h$ relationship

Fig. 10. Undrained effective stress paths of a series of triaxial tests using different consolidation periods

Fig. 11. Effects of consolidation time on $s_u/\sigma_{vc}$ in a series of triaxial tests

following:

$$\frac{S_u}{\sigma_{vc}} = m_c \ln \left( \frac{t_{total}}{t_{vc}} \right) + \left( \frac{S_u}{\sigma_{vc}} \right)_{t_{vc}}$$  \hspace{1cm} (1)

where the $m_c$ denotes a non-dimensional coefficient regarding the consolidation time effect on $S_u/\sigma_{vc}$.

The effect of consolidation period in the DSB test is
shown in Fig. 12. Fig. 13 shows the consolidation time effect on $S_d/\sigma_{vc}$. A similar trend was observed for the $S_d/\sigma_{vc}$ value which increased with the consolidation period.

4.2 Shear Rate Effects

Figs. 14 and 15 show the results of a series of triaxial compression and extension tests using different, but fixed, axial strain rate. Similar results in DSB test are shown in Fig. 16.

As seen in Fig. 17, the trend is obvious for the $S_d/\sigma_{vc}$ value which increases with the axial strain rate or shear displacement rate in a manner that

$$\left(\frac{S_d}{\sigma_{vc}}\right)_{\bar{r} \rightarrow r_v} = m \ln(V_r) + \left(\frac{S_d}{\sigma_{vc}}\right)_{\bar{r} \rightarrow 1}$$  \hspace{1cm} (2)
4.3 Isotach Behavior

As seen in Figs. 14 and 15, the axial strain rate was altered in steps between 0.02%/min and 1%/min in two triaxial tests (one in compression and the other in extension). Isotach behavior was obvious in that the stress-strain relationship as well as the undrained effective stress path shifted swiftly to the relevant stress-strain and stress path curves when the axial strain rate was changed in steps at 0.02%/min, 0.1%/min and 1%/min.

Such isotach behavior was also observed in a DSB test. As seen in Fig. 16, the shear stress-horizontal displacement relationship as well as the effective stress path shifted swiftly to the relevant curves when the axial strain rate was changed in steps at 0.02 mm/min, 0.1 mm/min and 1 mm/min.

4.4 Comparison of Undrained Shear Strength from Different Tests

A comparison of $S_u/\sigma_{vc}$ among triaxial compression and extension tests, direct shear test and unconfined compression test.
test is shown in Fig. 18. In this figure, the results of other tests, i.e., the unconfined compression test using very fresh samples from check boring and the triaxial compression and extension tests on isotropically consolidated samples are also shown for comparison.

When the samples were consolidated to a common 3t, the $S_u/\sigma_{uc}$ value was larger in the order of triaxial compression, DSB and triaxial extension tests. The $S_u/\sigma_{uc}$ value from unconfined compression test using fresh samples from check boring provided nearly the average of the $S_u/\sigma_{uc}$ of these three tests.

5. Conclusions

Rate as well as time effects were significant for undrained shear behavior of Holocene clay at Kobe airport. The $S_u/\sigma_{uc}$ value increased with the shear rate as well as the consolidation period as examined in both triaxial and DSB tests. Isotach properties were also observed in a manner that the stress-strain relationship as well as the undrained stress path was uniquely related to the current shear rate. When

\[ S_u/\sigma_{uc} = \text{design: } c_{u}/\rho_{cc} = 0.35 \]

Fig. 18. Comparison of undrained shear strength from different tests

the samples were consolidated to a common 3t, the $S_u/\sigma_{uc}$ value was larger in the order of triaxial compression, DSB and triaxial extension tests. The $S_u/\sigma_{uc}$ value from unconfined compression test using fresh samples from check boring provided nearly the average of the $S_u/\sigma_{uc}$ of these three tests.

References


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Comparative Analysis of Waste Landfill Stability Using Field Measurement Data

현장계측자료를 이용한 폐기물 매립지의 안정성 비교

Jang, Yeon-Soo1 장 연 수
Choi, Jong-Sig2 최 종 식
Ryu, Hye-Rim3 류 혜 림

요  지

폐기물 매립 과정에서의 매립지의 안정성을 평가하기 위하여 수도권 제2매립지의 현장계측자료를 분석하였다. 정량적인 안정성해석을 위하여 Tominaga-Hashimoto 법, Kuriharh 법, Matsuo-Kawamura 법을 사용하였으며 해석 결과 폐기물 매립과 유지관리가 대체로 양호하게 이루어지고 있음을 확인할 수 있었다. 북쪽지역에 위치하며 점토의 두께가 얇은 1-F블록의 경우 안정성이 매립하중에 민감하게 반응함을 알 수 있어 좀 더 세심한 관리가 필요한 것으로 나타났다. 안정적 매립관리를 위한 Kuriharh 법에 사용되는 수평변위속도는 2cm/day부터 2.5cm/day까지 늘릴 수 있을 것으로 판단되었으며 매립진행중 매립지의 정확한 안정성 판단을 위해서는 본 연구에서 수행한 바와 같은 복수의 안정성 검토를 통한 비교분석이 필요한 것으로 나타났다.

Abstract

Analysis of field measurement data of Gimpo #2 Metropolitan Landfill is performed to evaluate the stability of the landfill during waste disposal. For the stability analysis, Tominaga-Hashimoto method, Kuriharh method and Matsuo-Kawamura method, which may be able to manage the stability of the landfill quantitatively, are used. The results of the stability analysis showed that the landfill slope is generally maintained well with regard to the disposal and stability management. Stability of 1-F block, where the depth of clay layer is relatively thin in North block, was sensitive to the waste loading and it requires more thorough management. Proper standard of horizontal displacement velocity in this landfill by Kuriharh method can be increased from 2 cm/day to 2.5 cm/day for safe management of the landfill. Multiple check of the stability with various stability analysis methods is needed for accurate judgment of the stability of the landfill.

Keywords : Field measurement, Kuriharh method, Matsuo-Kawamura method, Stability analysis, Tominaga-Hashimoto method, Waste landfill

1. Introduction

Because of national economic growth and high environmental concerns, it is getting difficult to provide landfill sites which are indispensable for our living. To cope with this problem, plans to build the landfill on reclaimed land
along the shore or on soft land have been tried, e.g. Metropolitan area Landfill near Seoul, Gunsan Landfill, Seosan Landfill etc. In the case of the waste landfill built on weak clay, there is some risk of excessive settlements and slope failures of landfill. If a stable landfill is accomplished through measuring unpredictable problems at the time of designing, we can achieve the goal that can ensure maximum landfill capacity in a given site area. To ensure the stability of the landfill within a limited site area, it is necessary to investigate and understand the characteristics of soft land by identification of the requirements for waste filling and by quantitative field measurement and management of landfills.

In this study, the analysis of field measurement data of Gimpo #2 Metropolitan Landfill is performed to evaluate the stability of the landfill during disposal. For the stability analysis, Tominaga-Hashimoto method, Kuriharh method, and Matsuo-Kawamura method which may be able to manage the stability of the landfill quantitatively, are used.

2. Site Conditions

The Sudokwon Landfill Site is the first huge sanitary landfill of its total area 20 million m$^2$ in Korea. Sanitary and stable disposal of wastes about 15,000 ton per day (household wastes, 27%; construction wastes, 56%, and others, 17%) is performed from Metropolitan areas including Seoul, Gyeongido and Incheon City (Sudokwon Landfill Site Management Corp., 2006). It is located near Baekseok-dong, Seogu in Incheon as shown in Fig. 1, and constructed in the west coastal reclaimed land near the lower reaches of the Han River.

The landfill consists of the first landfill, 4.1 million m$^2$, the second landfill, 3.7 million m$^2$, the third landfill, 3.3 million m$^2$, and the forth landfill, 3.9 million m$^2$ and other facilities including the environmental research complex, 4.9 million m$^2$. The design of the first landfill was finished in December, 1988, and the construction of its infrastructure was started in 1989. After starting to accept wastes in February, 1992, the disposal of the waste continued for 8 years until October, 2000. The total weight of the disposed waste was 64 million tons. At present, the final covering has been completed, and the foundation work for a golf-course is going on in the upper surface area. The second landfill, with its capacity, 67 million ton, has begun to be filled since October, 2000, and disposal is going on now. The second landfill has been planned to have the height of 40 m, and a total of 8 stages.

3. Characteristics of Soils under the Landfill

The layer of the subject area consists of the marine sedimentary soil, sand, weathered residual soil, weathered rock layer, and the bedrock from the surface. The marine sedimentary layer consists of the upper weak clay layer, the middle medium strength clay layer, and the lower firm clay layer. The layer range and N value characteristics are listed in Table 1. and the sectional view of each soil
Comparative Analysis of Waste Landfill Stability Using Field Measurement Data

Table 1. Characteristics of Soil Layers under Gimpo #2 Landfill

<table>
<thead>
<tr>
<th>Layer Composition</th>
<th>Site Classification</th>
<th>Layer Range (m)</th>
<th>N Value</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>upper marine sedimentary layer</td>
<td>ML, CL</td>
<td>4.0 ~ 13.0</td>
<td>below 10</td>
<td>extremely weak ~ weak</td>
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<tr>
<td>lower marine sedimentary layer</td>
<td>ML, CL</td>
<td>3.0 ~ 15.3</td>
<td>above 10</td>
<td>firm ~ extremely firm</td>
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<td>stream sedimentary layer</td>
<td>SM</td>
<td>1.0 ~ 9.2</td>
<td>-</td>
<td></td>
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<tr>
<td>weathered residual soil layer</td>
<td>RS</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>bedrock</td>
<td>WR, SR, HR</td>
<td>-</td>
<td>-</td>
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</tbody>
</table>

Table 2. Geotechnical properties of the subsurface of the landfill

<table>
<thead>
<tr>
<th>Layer Classification</th>
<th>Depth (m)</th>
<th>Unit Weight (kN/m³)</th>
<th>Internal Friction Angle (°)</th>
<th>Cohesion (kg/cm²)</th>
<th>Hydraulic Conductivity (cm/sec)</th>
<th>Void Ratio</th>
<th>Compression Index</th>
<th>Coefficient of Consolidation c_v (cm²/sec)</th>
<th>Water Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft layer 1</td>
<td>Range</td>
<td>2.5 ~ 5</td>
<td>17 ~ 19.5</td>
<td>0.15 ~ 0.3</td>
<td>1×10^{-7} ~ 2×10^{-3}</td>
<td>6.3×10^{-4} ~ 4.5×10^{-1}</td>
<td>0.8 ~ 1.2</td>
<td>0.1 ~ 0.4</td>
<td>1×10^{-5} ~ 8×10^{-3}</td>
</tr>
<tr>
<td>Representative value</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5×10^{-7}</td>
<td>2.4×10^{-3}</td>
<td>1.0</td>
<td>0.2</td>
<td>3×10^{-3}</td>
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<tr>
<td>Soft layer 2</td>
<td>Range</td>
<td>2.5 ~ 5</td>
<td>17 ~ 19.5</td>
<td>0.2 ~ 0.35</td>
<td>7×10^{-9} ~ 6×10^{-7}</td>
<td>1.76×10^{-4} ~ 6.87×10^{-3}</td>
<td>0.8 ~ 1.2</td>
<td>0.1 ~ 0.4</td>
<td>1×10^{-3} ~ 8×10^{-3}</td>
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<tr>
<td>Representative value</td>
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<td></td>
<td></td>
<td></td>
<td>2×10^{-7}</td>
<td>2.4×10^{-3}</td>
<td>0.9</td>
<td>0.2</td>
<td>2×10^{-3}</td>
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<td>lower sedimentary layer</td>
<td>Range</td>
<td>5 ~ 10</td>
<td>18 ~ 19.5</td>
<td>0.50</td>
<td>8×10^{-9} ~ 5×10^{-7}</td>
<td>1.06×10^{-4} ~ 4.99×10^{-3}</td>
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<tr>
<td>Representative value</td>
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<td></td>
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<td></td>
<td>1×10^{-7}</td>
<td>2×10^{-3}</td>
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</tr>
<tr>
<td>Sand + Weathered soil</td>
<td>Range</td>
<td>0 ~ 17</td>
<td>19</td>
<td>30</td>
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<td>4.11×10^{-4} ~ 5.64×10^{-4}</td>
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<tr>
<td>Representative value</td>
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<td></td>
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<td></td>
<td>1.3×10^{-4}</td>
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</tr>
</tbody>
</table>

Fig. 3. Casagrande plasticity chart

To the depth of 5 m, average liquid limit is tested as 30 to 40 percent and liquidity index ranges from 0.5 to 2.0 and water content is 30 to 40 percent. At the depth below 5 m, average liquid limit is 35 to 50 percent and liquidity index is below 1. The Casagrande plasticity chart of the subject landfill in Fig. 3 shows that the subsurface soil of the landfill consists of inorganic clays of medium compressibility and inorganic clays of low plasticity.

Average coefficient of consolidation, c_v, ranges from 2.0×10^{-3} cm²/sec to 3.0×10^{-3} cm²/sec, and average compression index, c_v, is from 0.15 to 0.25. The upper and lower layers are slightly over-consolidated and the middle layer is normally consolidated (Table 2).

The hydraulic conductivity of the marine clay layer is also listed in Table 2. In-situ hydraulic conductivity is usually ten to one hundred times larger than the laboratory hydraulic conductivity because the effect of sand seam contained in the layer (Jang and Lee, 2002). The in-situ hydraulic conductivity ranges mostly from 2.0×10^{-5} to 2.4×10^{-5} cm/sec, and the laboratory hydraulic conductivity from 1.0×10^{-7} to 5.0×10^{-7} cm/sec (Sudokwon Landfill Site Operation Management Union, 1995, 1998).

In the area, measurement instruments more than 10 types, about 290 instruments such as settlement plates, earth pressure cell, piezometer, inclinometer were installed (Fig. 4). These instruments have been measured and analyzed for confirming stability of landfill slope and settlement tendency of the ground and waste in the landfill. The representative cross section of instruments installation is

![Installation location map of the measurement instruments](image)

<table>
<thead>
<tr>
<th>Measuring Instrument Name</th>
<th>Symbol</th>
<th>Spot</th>
<th>Quantity</th>
<th>Establishing Purpose and Location</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Settlement Plates</td>
<td>A</td>
<td>11</td>
<td>35</td>
<td>Each Layer within Landfill B/L</td>
<td>Reclamation Plan Construction</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>2</td>
<td>2</td>
<td>The Highest Part of Wastes Layer</td>
<td>Reclamation Plan Construction</td>
</tr>
<tr>
<td>Settlement Cell</td>
<td></td>
<td>1</td>
<td>2</td>
<td>The Outer Slope Ground</td>
<td>Reclamation Plan Construction</td>
</tr>
<tr>
<td>Continued Settlement Gauges</td>
<td>A</td>
<td>2</td>
<td>2</td>
<td>The Outer Slope Ground</td>
<td>Reclamation Plan Construction</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>2</td>
<td>2</td>
<td>Intersection within Ground</td>
<td>Reclamation Plan Construction</td>
</tr>
<tr>
<td>Hydraulic Pressure Settlement Gauges</td>
<td></td>
<td>1</td>
<td>1</td>
<td>The Outer Slope Ground</td>
<td>Reclamation Plan Construction</td>
</tr>
<tr>
<td>Inclinometer</td>
<td>A</td>
<td>5</td>
<td>5</td>
<td>Horizontal Displacement of the Outer Part</td>
<td>Reclamation Plan Construction</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>12</td>
<td>12</td>
<td>Infrastructure Construction</td>
<td></td>
</tr>
<tr>
<td>Water Level Meter</td>
<td></td>
<td>6</td>
<td>18</td>
<td>The Outer Monitoring Well</td>
<td>Infrastructure Construction</td>
</tr>
<tr>
<td>Earth Pressure Cell</td>
<td>A</td>
<td>9</td>
<td>28</td>
<td>Each Layer within Landfill B/L</td>
<td>Reclamation Plan Construction</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>5</td>
<td>13</td>
<td>The Outer Slope Ground</td>
<td>Reclamation Plan Construction</td>
</tr>
<tr>
<td>Piezometer</td>
<td>A</td>
<td>6</td>
<td>18</td>
<td>Ground within Landfill B/L</td>
<td>Reclamation Plan Construction</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>5</td>
<td>22</td>
<td>The Outer Slope Ground</td>
<td>Reclamation Plan Construction</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>15</td>
<td>29</td>
<td>The Outer Road</td>
<td>Infrastructure Construction</td>
</tr>
<tr>
<td>Earth Pressure Cell or Piezometer</td>
<td></td>
<td>3</td>
<td>10</td>
<td>Ground within Slope</td>
<td>Reclamation Plan Construction</td>
</tr>
<tr>
<td>Control Room</td>
<td></td>
<td>3</td>
<td>3</td>
<td>The Outer Slope</td>
<td>Reclamation Plan Construction</td>
</tr>
<tr>
<td>Standard Rod</td>
<td></td>
<td>11</td>
<td>14</td>
<td>Each Layer Settlement Standard Rod</td>
<td>Reclamation Plan Construction</td>
</tr>
<tr>
<td>Vertical Exclusion Well</td>
<td></td>
<td>24</td>
<td>75</td>
<td>Ground within Landfill B/L</td>
<td>Reclamation Plan Construction</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td>123</td>
<td>291</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 4. Installation location map of the measurement instruments
shown in Fig. 5. It was established considering characteristics of landfill slope and the planned landfill height.

Using the data of field measurement, several studies that can predict the failure of the ground and waste body, quantitatively, are going on. For the stability analysis, Tominaga-Hashimoto method, Kuriharh method and Matsuo-Kawamura method, which may be able to manage the stability of the landfill quantitatively, are used (Choi and Park, 2007). Index of the waste management consists of the settlement \( \rho \) on the surface of the waste bank, the horizontal displacement \( \delta \) at the toe of the peripheral soil bank, and the waste bank load \( q \). Horizontal displacement of peripheral bank is regarded as positive. The analyzed locations are 1-A, 3-B, 4-D, 3-F, and 1-F of landfill areas (Sudokwon Landfill Management Corp, 2005; Choi, 2007).

### 4.1 Tominaga-Hashimoto Method

In the landfill slope of 1-A and 3-B block, the ratio of horizontal displacement to settlement inside block is relatively small and its behavior indicates to be safe (Fig. 6). The characteristics of the point “A” in the case of...
4-D block (Fig. 7), is that the settlement is superior to the horizontal displacement. It indicates that the stability is recovered because of stopping disposal within a certain period of time, as the earth bank of stage 2 was constructed in January, 2002. This is also shown as the point “A” in Fig. 9 analyzed by Kuriharh method. In the case of 3-F block, each point “A” and “B” indicates the condition of stage 2 and 3 during construction as shown in Fig. 8. The trace responds quite sensitively but indicates stable behavior in general.

In the case of 1-F block, dangerous tendency is shown (Fig. 8). At the point “A”, the displacement rapidly increases more than the settlement at the disposal of stage 2 from the landfill height of 6.5 m to 10.5 m between 12th and 29th of August, 2005. Also, at the point “B”, the ratio of horizontal displacement to settlement increases according to the disposal of stage 3 from April 14 to May 30 in 2005. However, it was found that the trace went to stable condition from about 18 months after reclamation of stage 3.

From the result of the soil layer survey, it was found that soft layer is deep in the seashore side (Zone 1). The inland side (3-F and 1-F) in Zone 2 has thin soft clay layer (Fig. 2). Predicting the trace of stability management in the thin clay layer area responded more sensitively than that in the deep clay layer area. The north block of landfill where the clay layer is thin needs more intensive control in the time and load of waste disposal.

4.2 Kuriharh Method

The management standard of displacement velocity in the subject area is 2 cm/day, but there is a case that the displacement velocity exceeded 5.5 cm/day. Construction was stopped within a certain period of time at the point “A” as shown in 4-D block in Fig. 9. 4-D block is the weakest block because old ditch was located on the shore of an inlet in reclaimed land before promoting landfill. Unstable behavior was recognized during construction stage 2 of earth bank.

However, in contrast with the analysis results by Tominaga-Hashimoto method in 4-D block at the same time, the extent of the settlement occurred in proportion to the horizontal displacement. Stability was not heavily influenced in this case.

In the case of management standard of the daily displacement velocity in Kuriharh method, there was a case that direct failure did not occur according to the increase of horizontal displacement velocity up to maximum 5 centimeter, and most of the areas had experiences exceeding 2 centimeter per day. Therefore, this landfill is considered to be safe if the measurement standard of 2.5 centimeter per day is selected. However, believing the results by this method blindly is dangerous. Measurement analysis should be performed and judged with other methods, e.g., Tominaga-Hashimoto method, etc.

4.3 Matsuo-Kawamura Method

Matsuo-Kawamura method is the method managed by management standard $s - \delta/s$. The failure criteria curve is decided by the relationship between $\delta/s$ and $s$. Failure, stable or unstable behavior of banking is judged by
Comparative Analysis of Waste Landfill Stability Using Field Measurement Data

0.00 0.50 1.00 1.50 2.00
0.0 1.0 2.0 3.0 4.0 5.0
Horizontal Displacement/Settlement

Fig. 10. Management results by Matsuo–Kawamura method in 1–A and 3–B block

0.0 1.0 2.0 3.0 4.0 5.0
Horizontal Displacement/Settlement

Fig. 11. Management results by Matsuo–Kawamura method in 4–D and 3–F block

0.00 0.50 1.00 1.50 2.00
0.0 1.0 2.0 3.0 4.0 5.0
Horizontal Displacement/Settlement

Fig. 12. Management results by Matsuo–Kawamura method in 1–F block

noticing whether the measuring trace of \( s \)-\( \delta \)/\( s \) reaches the standard curve during the construction of banking.

The result of the analysis in 1–A, 3–B and 3–F block showed that behavior conditions of their traces had a stable tendency and agreed well with the result of Tominaga-Hashimoto method or Kuriharh method (Fig. 10).

In the case of 4–D block, in contrast with the analysis results by Tominaga-Hashimoto method and Kuriharh method, in January, 2002, during the construction of stage 2 bank, there was a leaving period after stopping banking temporarily because the displacement velocity per day was 5.5 cm/day (Fig. 11). The result of analyzing the behavior of the measuring trace by Tominaga-Hashimoto method showed that there was no special risk sign. Therefore, this landfill seems to be appropriate for changing the measurement standard of the daily displacement velocity from 2 to 2.5 cm/day.

In case of 1–F block, the measuring trace has a tendency to move to risk management criteria curve because the horizontal displacement is relatively larger than settlement increasing landfill height like the point “A” in Fig. 12. This results have high correlation with the results by Tominaga-Hashimoto method (Fig. 8). The movement of the management trace had very sensitive tendency.
5. Conclusion

In this paper, the stability of Gimpo #2 landfill is analyzed through investigating the field measurement data. The results of the analyses are summarized as the following:

(1) Field measurement is necessary for the landfill constructed in the seashore of weak clay. The analysis results of stability management showed that the landfill slope is generally maintaining well with regard to the disposal and stability.

(2) The result of stability analysis of 1-F block was sensitive where the depth clay layer is relatively thin. It was found that the management of North block, where the depth of clay is relatively smaller, needs more thorough management.

(3) The proper standard of horizontal displacement velocity in this landfill by Kuriharh method can be increased to the value of 2.5 centimeter per day for safe management of the landfill.

(4) Judging the stability of the landfill should be done by comparing quantitative methods synthetically like the methods introduced in this study.

References

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(received on Feb. 6, 2009, accepted on Mar. 21, 2009)
Experimental Study of Bearing Capacity in Dredged and Reclaimed Ground

준설매립지반의 지지력 산정에 관한 실험적 연구

Yang, Tae-Seon¹ Yang Tae-Sean  Lee, Song² Lee Song  Baek, Won-Jin³ Baek Won-Jin  Kim, Ju-Hyun⁴ Kim Ju-Hyun

요  지


Abstract

In this study, two-dimensional model loading tests were carried out to analyze the problem of the bearing capacity evaluation in the early stage of ground improvement in marine clay dredged deposits. The bearing capacity was estimated using previously proposed methods for double-layered clay deposit, that is, Button’s (1953), and Brown & Meyerhof’s equation (1969). To estimate the application of these equations, the estimated bearing capacity was then compared with the results of the two-dimensional model loading tests.

Keywords : Bearing capacity, Marine clay dredged deposit, Two-dimensional model test

1. Introduction

Marine clay dredged deposits undergo the process of sedimentation and self-weight consolidation after dredging. Because the surface of marine clay dredged deposits are exposed to the atmosphere, the surface of some marine clay dredged deposits goes through radiant heat and wind effect.

So, an evaporative phenomenon happens, that is, the water is diffused into the formation of water vapor from surface to air. At this time, if evaporation loss of the surface is greater than the amount of drainage by self-weight consolidation, the water contents of the surface in marine clay dredged deposits decrease. This is defined as desiccation. Desiccation phenomenon causes unsaturated condition because of the shrinkage and decrease of moisture in marine clay dredged deposits. There are marine clay dredged deposits in which the surface forms in the complex shape of cracks. Because of this phenomenon, the surface of marine clay dredged deposits holds lower water contents and higher shear strengths than does their initial condition. These layers are generally defined as desiccated crust.

¹ Member, Associate Prof., Dept. of Construction Information Eng., Kimpo College  
² Member, Prof., Dept. of Civil Eng., The Univ. of Seoul  
³ Member, Assistant Prof., Dept. of Bio System&Agricultural Eng., Chonnam National Univ.  
⁴ Member, Ph. D, Dept. of Civil Eng., The Univ. of Seoul, soil1004@hotmail.com, Corresponding Author
layers. As a result, the dredged and reclaimed ground has double-layered deposits which mean that desiccated crust layers overlie very soft clay soils.

The lower layer of desiccated crust layer holds high water contents, and high compressibility. It takes long-term consolidation periods to stabilize its ground without stabilization methods. So, in such a case, it is necessary to apply preload, and vertical drain method, in marine clay dredged deposits for long-term stability. This should be considered during the estimation of stability and trafficability in the early stage of stabilization process in marine clay dredged deposits. The bearing capacity of homogeneous soils (It is limited to cases where water contents and shear strengths have a typical range except for a ultra-high water contents such as marine clay dredged deposits) has been the subject of extensive studies.

These have yielded results which either are correct for the stated assumptions, such as the Prandtl solution for a perfectly plastic foundation material, or contain approximations which have been found by field experience or model studies to be sufficiently accurate to permit their use with confidence. This situation does not obtain to the same degree in the case of non-homogeneous subsoils. According to Brown & Meyerhof (1969), it was indicated that the solution which has been advanced by Button (1953) is, for the most part, either admittedly very approximate, or unverified by tests or analyses of full-scale foundations, or else contain assumptions which do not appear to be entirely justifiable.

Therefore, in this study, two-dimensional model loading tests were carried out to analyze the problem of bearing capacity evaluation and its application. To estimate the application of Button's, and Brown & Meyerhof's equations, the estimated bearing capacity was also compared with the results of the two-dimensional model tests.

2. The Estimation Method of Double-layered Clay Deposits

2.1 Button's Equation (1953)

Button (1953) has derived the equation for the bearing capacity of foundations on layered clay soils, as shown in Fig 1. For undrained loading ($\phi = 0$ condition), let $c_1$ and $c_2$ be the shear strength of the upper and lower clay layers, respectively. For layered soils, the value of the bearing capacity factor, $N_c$, is not a constant. It is a function of $c_2 / c_1$ and $d / b$ ($d =$ depth measured from the bottom of the foundation to the interface of the two clay layers). It can be seen from Fig. 1 that if the lower layer of the clay is softer than the top (that is, $c_2 / c_1 < 1$), the value of the bearing capacity factor ($N_c$) is lower than
when the soil is not layered (that is, when \( c_2 / c_1 = 1 \)). This means that the ultimate bearing capacity is reduced by the presence of a softer clay layer below the top layer. Button assumed that the potential failure surface is cylindrical. So, for a homogeneous and isotropic foundation, the assumption of a cylindrical surface leads to a value of \( N_c \), which is about 7% higher than that obtained by the rigorous Prandtl solution. That is, when the soil is not layered, the value of the bearing capacity factor \( (N_c) \) is not 5.14 but 5.5. According to Meyerhof & Brown (1967), where the subsoil is neither homogeneous nor anisotropic, it is indicated that the failure surface will no longer be cylindrical, and that the error associated with the assumption of a cylindrical failure surface will increase with increasing nonhomogeneity.

2.2 Brown & Meyerhof’s Equation (1969)

Brown & Meyerhof (1969) have derived an equation such as Button’s for the bearing capacity of foundations on layered clay soils. The estimated equation was based on model tests using circular and strip footings, and using a range of layer thicknesses and clay strengths. For the value of the bearing capacity factor to be estimated, Brown & Meyerhof suggested using the relation between modified bearing capacity factors and shear strength ratio for strip and circular footings, as shown in Fig. 2.

3. Laboratory Tests

3.1 Lateral Vacuum Consolidation Tests

Lateral vacuum consolidation tests were carried out to form the stiff clay overlying soft clay. The details of the testing condition are shown in Table 1. As shown in Table 1, after dredged clays were filled up to heights of up to 40, 30, 20 cm, respectively into the test equipment, lateral vacuum drains with double mats were placed on the surface of marine clay dredged deposits. As soon as dredged clays were filled up to heights of up to 50 cm, lateral vacuum consolidation tests were then carried out. Water contents were measured for varied depth in this test equipment after lateral vacuum consolidation tests. As shown in Fig. 3, the water contents of the dredged clay adjacent to the double mats were distributed in the range of 60~80% and 65~80%, respectively. Water contents were suddenly moved 2~2.5 cm away from where the

Table 1. Matrix of Laboratory Model Test

<table>
<thead>
<tr>
<th>Initial Water Contents (%)</th>
<th>Vacuum Pressure (kPa)</th>
<th>The Size of Test Equipment (D×L×H, m)</th>
<th>Drains</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>The Depth of Installation (cm)</td>
</tr>
<tr>
<td>Case I</td>
<td>190</td>
<td>0.2×1.2×0.5</td>
<td>10, 20, 30</td>
</tr>
<tr>
<td>Case II</td>
<td>190</td>
<td>0.2×1.2×0.5</td>
<td>10, 20, 30</td>
</tr>
</tbody>
</table>
double mats were located. In case of the undrained shear strength distribution for varied depths using vane test device, as shown in Fig. 4, the undrained shear strengths of the dredged clay adjacent to the double mats in case I and II were distributed in the range of 7~9 kPa and 6~7.5 kPa, respectively. The undrained shear strengths were suddenly reduced 5 cm away from where the double mats were located. Then, the undrained shear strengths were gently reduced some 5 cm and 10 cm away from where the double mats were located. And, when moved 10 cm beyond the double mats, the undrained shear strengths of the dredged clay had less than 1 kPa.

3.2 Two-Dimensional Model Tests

Two-dimensional model tests were carried out to analyze the bearing behavior in marine clay deposits including desiccated crust layer. The width and length of the plate are 10 cm and 19.5 cm, respectively. Constant strain rates about 5 mm/min were applied to their tests. Load-settlement behaviors and the test results are shown in Fig. 5, and Table 2. In case I, based on the experimental results, it was estimated that the ultimate failure loads of the grounds were equal to 19 kPa, 28 kPa, 19 kPa, respectively. Their corresponding settlements were 9 mm, 13 mm, 4 mm, respectively.

And, in case II, it was estimated that the ultimate failure loads of the grounds were equal to 14.5 kPa, 19 kPa, 3.5 kPa, respectively. Their corresponding settlements were 6 mm, 5 mm, 5 mm, respectively. Once beyond the ultimate failure load, the increase of load was accompanied by a large increase of ground settlement. Therefore, the ultimate failure load was estimated based on this point.

4. The Comparison of the Existing Equations and Experimental Testing Results

In case I, the ultimate bearing capacity of the dredged and reclaimed ground was estimated using Button’s, and Brown & Meyerhof’s equation in case the installation depths of the drain were 10 cm and 20 cm, respectively,
as shown in Table 3. The undrained shear strengths of marine clay dredged deposits were estimated by separating their deposits into the average shears strengths of the upper and lower clay layers. According to Skempton (1951), if the shear strength within a depth of approximately 2/3B beneath the foundation level does not vary by more than about ± 50% of the average strength in that depth, then this average value of c may be used in $\phi = 0$ analysis. Therefore, it was estimated that the thickness of the upper clay layers was 10 and 13 cm, respectively, based on Skempton’s research. It was then estimated that the average shear strengths of the upper clay layers were equal to 7.0 kPa and 8.8 kPa, respectively. At this time, average shear strength of the lower clay layers was estimated from boundary of desiccated lower part to the 2B of footing width, generally, considering influence depth of plate loading test. So, those of the lower clay layers were equal to 0.16 kPa and 0.58 kPa, respectively.

Also, it was calculated that the values of the bearing capacity factor ($N_c$) were 1.60 and 2.30, respectively, when using Brown & Meyerhof’s equation. The factors were 2.50 and 3.80, respectively, when using Button’s equation. Therefore, the bearing capacity factor, $N_c$, that was calculated using Button’s equation was estimated about 1.6 times as large as the factor calculated using Brown & Meyerhof’s equation. When the installation depth of the drain was 30 cm (this can happen when the distribution of shear strengths and water contents are not scattered with depth), its bearing capacity was estimated using Terzaghi’s equation for single layered clay deposit, that is, using $N_c = 3.81(N_c = 3.81$ from $c = (2c/3) \times 5.71)$, assuming that local failure occurred in dredged and reclaimed ground.

In case II, the ultimate bearing capacity of the dredged and reclaimed ground was estimated from the same method as case I, as shown in Table 4. And, it was estimated that the thickness of the upper clay layers was 13 and 13 cm, respectively, when the installation depths of the drain were 10 and 20 cm, respectively. It was then estimated that the average shear strengths of the upper clay layers were equal to 5.3 kPa and 6.9 kPa, respectively, and the values of the lower clay layers were equal to 0.10 kPa and 1.29 kPa, respectively. Also, it was estimated that the values of the bearing capacity factor ($N_c$) were equal to 2.05 and 2.93, respectively, when using Brown & Meyerhof’s equation. Those factors were equal to 3.10 and 4.60, respectively, when using Button’s equation. Therefore, the bearing capacity factor, $N_c$, that was also calculated using

### Table 3. Comparison of the Existing Equations and the Results of Tests (Case I)

<table>
<thead>
<tr>
<th>The Installation Depth of Drain (cm)</th>
<th>The Average Shear Strengths within the Depth of 2/3B (kPa)</th>
<th>The Average Shear Strengths of the Upper Clay Layers (kPa)</th>
<th>The Average Shear Strengths of the Lower Clay Layers (kPa)</th>
<th>The Thickness of Desiccated Crust Layers (cm)</th>
<th>$N_c$</th>
<th>The Equation (kPa)</th>
<th>The Experimental Results (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>-</td>
<td>7.0</td>
<td>0.16</td>
<td>10</td>
<td>-</td>
<td>2.5</td>
<td>1.60</td>
</tr>
<tr>
<td>20</td>
<td>-</td>
<td>8.8</td>
<td>0.58</td>
<td>13</td>
<td>-</td>
<td>3.8</td>
<td>2.30</td>
</tr>
<tr>
<td>30</td>
<td>5.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3.81</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

### Table 4. Comparison of the Existing Equations and the Results of Tests (Case II)

<table>
<thead>
<tr>
<th>The Installation Depth of Drain (cm)</th>
<th>The Average Shear Strengths within the Depth of 2/3B (kPa)</th>
<th>The Average Shear Strengths of the Upper Clay Layers (kPa)</th>
<th>The Average Shear Strengths of the Lower Clay Layers (kPa)</th>
<th>The Thickness of Desiccated Crust Layers (cm)</th>
<th>$N_c$</th>
<th>The Equation (kPa)</th>
<th>The Experimental Results (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>-</td>
<td>5.3</td>
<td>0.10</td>
<td>13</td>
<td>-</td>
<td>3.1</td>
<td>2.05</td>
</tr>
<tr>
<td>20</td>
<td>-</td>
<td>6.9</td>
<td>1.29</td>
<td>13</td>
<td>-</td>
<td>4.6</td>
<td>2.93</td>
</tr>
<tr>
<td>30</td>
<td>0.7</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3.81</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Experimental Study of Bearing Capacity in Dredged and Reclaimed Ground
Button’s equation was estimated about 1.6 times as large as the factor calculated using Brown and Meyerhof’s equation as in case I. When the installation depth of the drain was 30 cm, its bearing capacity was estimated from the same method as in case I.

The estimated bearing capacity using Brown & Meyerhof’s equation was in almost all cases underestimated compared to the results of two-dimensional model tests, while its value using Button’s equation was in almost all cases overestimated compared to the experimental results.

Meanwhile, according to Meyerhof & Brown (1967), based on comparing experimental result with Button’s equation, as shown in Fig. 6, it was indicated that where the lower layer is softer, Button’s analysis gave results which were in all cases higher than those obtained experimentally. Therefore, Bowles (1996) suggested that circular arcs be limited to cases where the strength ratio $c_2 / c_1$ is on the order of $0.6 < c_2 / c_1 \leq 1.3$; where $c_2 / c_1$ is much out of this range it indicates a large difference in the shear strengths of double layers and one might obtain $N_c$ using a method given by Brown & Meyerhof’s equation (1969) based on model tests.

These results may be caused by the difference between the two equations in the type of bearing capacity failure. Button’s equation is based on the assumption that a circular failure of the ground occurs, while Brown & Meyerhof’s equation is estimated, considering the footing punching through the top layer, and with full development of the bearing capacity of the lower layer.

Because punching shear failure occurred in the ground where two-dimensional model tests were performed and shear strength ratio was much out of Bowles’s recommended range, it was assumed that the estimated bearing capacity using Button’s equation was overestimated compared to the experimental results. Meanwhile, In Meyerhof & Brown’s equations based on model tests (1969), the ratio of upper layer thickness to width or diameter of footing, $H/B$ or $H/2R$, varied from 0.5 to 3.0, and the strength of the upper layer was up to 4 times the strength of the lower layer, that is, shear strength ratios were from 0.25 to 1, not including model tests with shear strength ratio less than 0.25. But, in case of marine clay dredged deposits, generally, their ground has extremely low shear strength and ultra-high water contents of more than 100% except for a part of desiccated upper layer. So, based on this viewpoint, it was assumed that the estimated bearing capacity using Meyerhof & Brown’s equation was in disagreement with the results of two-dimensional model test. So, further study and additional model tests are necessary to evaluate bearing behavior in double-layered marine clay deposits with shear strength ratio less than 0.25.

Meanwhile, according to a study of embankment stability...
analysis on soft ground including crust layer formed by desiccation and weathering process, it is indicated that the average shear strengths of desiccated crust layers are not mobilized perfectly from the viewpoint of bearing behavior. So, when applying properly reduced undrained shear strength to the desiccated crust layer, it was confirmed that the stability analysis was in agreement with real behavior in field (La Rochelle, 1974; Lefebvre, 1987; Tavenas, 1980). Therefore, All things considered, it is recommended that the ultimate bearing capacity using Brown & Meyerhof’s equation be adopted as the values of the dredged and reclaimed ground for the conservative evaluation.

When the installation depth of drain was 30 cm in case I and II, the ultimate bearing capacity was estimated to be similar to the experimental results. This was done using Terzaghi’s equation for single layered clay deposit, that is, using \( N_c = 3.81 \), assuming that local failure occurred in dredged and reclaimed ground. From this result, it was shown that \( N_c = 3.81 \) for single-layered clay deposit could be applied to estimation of the bearing capacity in marine clay dredged deposits.

Meanwhile, if the thickness of the top layer from the base of the footing is less than \( H = \frac{B}{2 \tan(45 + \phi/2)} \), theoretically, the ground should be considered as a layered soil. And, according to Fig. 2 (a), when shear strength ratio \( > 1 \), if \( H/B \) is more than 0.7, the ground could be considered as a single layer. So, in this study, based on these cases, the average shear strength for single-layered clay deposit was estimated within the depth of \( 2/3B \) (B = footing width). Also, From Fig. 2 (a), as shear strength ratios are low values, that is, about 0.2, it is assumed that the rupture zone (failure depth) is extended, compared with theoretical failure depth. So, Brown & Meyerhof’s research (1969) shows that rupture zone is strongly influenced by the thickness of upper layer and shear strength ratio.

5. Conclusions

(1) Based on comparing experimental results with Terzaghi’s equation for single layered clay deposit, that is, using \( N_c = 3.81 \), assuming that local failure occurred in dredged and reclaimed ground, it was shown that \( N_c = 3.81 \) for single-layered clay deposit could be applied to estimation of the bearing capacity in marine clay dredged deposits.

(2) It is recommended that the ultimate bearing capacity using Brown & Meyerhof’s equation be adopted as the values of the dredged and reclaimed ground for the conservative evaluation.

(3) The estimated bearing capacity using Brown & Meyerhof’s equation was in almost all cases underestimated compared to the results of two-dimensional model tests, while its value using Button’s equation was in almost all cases overestimated compared to the experimental results. Therefore, further study and additional model tests are necessary to evaluate bearing behavior in double-layered marine clay deposits with shear strength ratio less than 0.25.

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Pile Capacities Determined by Osterberg Cell Tests at Incheon Grand Bridge

Kim, Myoung-Mo1  김 명 모  Jung, Sung-Jun2  정 성 준
Kim, Jeong-Hwan3  김 정 환  Hwang, Dae-Jin4  황 대 진

1 Prof., Dept. of Civil and Environmental Eng. Seoul National Univ., geotech@snu.ac.kr, Corresponding Author
2 Graduate Student, Dept. of Civil and Environmental Eng. Seoul National Univ.
3 General Manager, Incheon Bridge Project Design Team, Samsung Corporation
4 Geotechnical team, Institute of Technology, Samsung Corporation

1. Introduction

Incheon Grand Bridge Project is a marine highway bridge with total length of about 12 km, linking New Songdo City to Incheon International Airport. The project involves a 1.48 km-long cable-stayed bridge section, a 1.78 km-long approach bridge section, and 8.4 km-long viaduct. RCD piles of various diameters (1.8 m, 2.4 m, 3.0 m) were designed as foundations of the bridge, which were all embedded in bedrocks. To verify the bearing capacities of the RCD piles, 4 test piles were constructed and O-cell tests were performed on the piles. In this paper, the procedures determining bearing capacity parameters along the pile depth using the O-cell test results are illustrated.

Keywords: Pile capacity, O-cell tests, Incheon Grand Bridge
section. In accordance with the CSR (Concessionaire Supplementary Requirements), 4 numbers of full-scaled piles were constructed for the preliminary pile load test.

O-cell tests were performed on the test piles. Maximum load of 289,580 kN (28,958 ton) was applied to 3 piles of 2.4 m diameter and 1 pile of 3.0 m diameter.

2. Test Site and Soil Profiles

The testing was carried out at the location, 300 m north from the navigational route in Viaduct, since we failed to obtain the right of occupation of main navigational route in CSB section. The location of test piles is shown in Figure 1.

In order to understand the quantitative condition of bed rock effecting the calculation of the bearing capacity on design and the actual bearing capacity of load testing pile, several tests such as Field Loading Test (LLT, PMT, GMJ, etc.) and Indoor Rock Testing (unconfined compression test, point load test, etc.) were carried out at the center of respective pile as well as soil investigation. Following Figure 2 shows the results of soil investigation.

3. Test Outline

The test concept, purpose and method as well as the pile toe condition and estimated bearing capacity are summarized in Table 1.

After a thorough survey of pile-soil conditions with respect to the settlement and bearing capacity, the loading
Table 1. Loading plan for testing piles

<table>
<thead>
<tr>
<th>Items</th>
<th>TP-1(E5) / (Ø 3,000 mm)</th>
<th>TP-2(W6) / (Ø 2,400 mm)</th>
<th>TP-3(W8) / (Ø 2,400 mm)</th>
<th>TP-4(E7) / (Ø 2,400 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intention</td>
<td>Confirms End Bearing of</td>
<td>Confirms End Bearing of</td>
<td>Confirms End Bearing of</td>
<td>Confirms End Bearing of</td>
</tr>
<tr>
<td></td>
<td>Hard Rock</td>
<td>Weathered Rock</td>
<td>Soft Rock</td>
<td>Soft Rock</td>
</tr>
<tr>
<td></td>
<td>Confirms Skin friction of</td>
<td>Confirms Skin friction of</td>
<td>Confirms Skin friction of</td>
<td>Confirms Skin friction of</td>
</tr>
<tr>
<td></td>
<td>Weathered Rock</td>
<td>Weathered Rock</td>
<td>Weathered Rock</td>
<td>Weathered Rock</td>
</tr>
<tr>
<td></td>
<td>Confirms Skin friction of</td>
<td>Confirms Skin friction of</td>
<td>Confirms Skin friction of</td>
<td>Confirms Skin friction of</td>
</tr>
<tr>
<td></td>
<td>Soft Rock</td>
<td>Soft Rock</td>
<td>Soft Rock</td>
<td>Soft Rock</td>
</tr>
<tr>
<td>Estimated Bearing Capacity</td>
<td>Ultimate Bearing Capacity</td>
<td>Ultimate Bearing Capacity</td>
<td>Ultimate Bearing Capacity</td>
<td>Ultimate Bearing Capacity</td>
</tr>
<tr>
<td></td>
<td>241,600 kN</td>
<td>35,000 kN</td>
<td>155,900 kN</td>
<td>97,300 kN</td>
</tr>
<tr>
<td></td>
<td>Skin Friction : 75,200 kN</td>
<td>Skin Friction : 24,000 kN</td>
<td>Skin Friction : 75,500 kN</td>
<td>Skin Friction : 51,600 kN</td>
</tr>
<tr>
<td></td>
<td>End Bearing : 166,400 kN</td>
<td>End Bearing : 11,000 kN</td>
<td>End Bearing : 80,400 kN</td>
<td>End Bearing : 45,700 kN</td>
</tr>
<tr>
<td>Test Method and O-Cell Place</td>
<td>Single-Level Test</td>
<td>Single-Level Test</td>
<td>Single-Level Test</td>
<td>Single-Level Test</td>
</tr>
<tr>
<td></td>
<td>Pile Toe (Hard Rock)</td>
<td>Pile Toe (Weathered Rock)</td>
<td>Pile Toe (Soft Rock)</td>
<td>Pile Toe (Soft Rock)</td>
</tr>
<tr>
<td>Effective Pile Length</td>
<td>40.143 m</td>
<td>44.2 m</td>
<td>45.1 m</td>
<td>40.01 m</td>
</tr>
<tr>
<td>Other Attachment</td>
<td>Gauge, Sonic Tube</td>
<td>Strain Gauge, Sonic Tube</td>
<td>Strain Gauge, Sonic Tube</td>
<td>Strain Gauge, Sonic Tube</td>
</tr>
<tr>
<td>Target Load</td>
<td>210,000 kN</td>
<td>90,000 kN</td>
<td>170,000 kN</td>
<td>120,000 kN</td>
</tr>
<tr>
<td>Load Achieved</td>
<td>289,580 kN</td>
<td>137,990 kN</td>
<td>245,310 kN</td>
<td>173,690 kN</td>
</tr>
<tr>
<td>Test Pile Property</td>
<td>- O-cell Quantity : 5 EA</td>
<td>- O-cell Quantity : 2 EA</td>
<td>- O-cell Quantity : 4 EA</td>
<td>- O-cell Quantity : 3 EA</td>
</tr>
<tr>
<td></td>
<td>- Concrete level : SeaBed</td>
<td>- Concrete level : SeaBed</td>
<td>- Concrete level : SeaBed</td>
<td>- Concrete level : SeaBed</td>
</tr>
</tbody>
</table>

The plan for the test pile was established so that loading and settlement would be the same on both sides (bottom/top) of the O-cell. Approximately 1.4 ~ 3.8 times the estimated ultimate capacity of the working piles was loaded on the test pile to confirm the ultimate capacity of the test piles. Vibrating Wire Type Strain Gauges, an instrument to measure the load transfer, were installed at intervals of 1.0 ~ 2.0 m accounting for the thickness of soil layer to measure the shaft friction of the socket at the supporting layers of weathered rock, soft rock and hard rock. In addition to Telltale used to measure the displacement of each part of the test pile, Embedded Compression Telltale (ECT-integrated of strain gauge and measuring rod) were installed to measure the displacements of weathered rock and soft rock. Expansion Transducers were installed between the bearing plates (bottom/top side) to measure the expansion of the O-cell during load period. Figure 3. Schematic of instrumentation of TP-1
3 shows the locations of the instruments on the test pile.

4. Analysis of Test Results

4.1 Ultimate Capacities

From the results of the O-cell tests, equivalent load-settlement curves can be produced, as shown in Figure 4. The derived ultimate capacities of piles based on these curves are only applicable to those particular pile types and conditions of geological strata. To obtain typical resistances of each rock strata, a generalized unit resistant value has to be evaluated. Figure 5 shows unit side resistances that were obtained from the measurement of strain gauges, which were installed at each layer of the rock strata.

In accordance with the ASCE Standard (ASCE 20-96, 1997), the ultimate capacity of a pile is defined as the load when the amount of movement of the pile head is equivalent to the summation of the elastic compression of the pile shaft, 1% of the pile diameter and a constant value of 0.15. In other words, the ultimate capacity will be mobilized when the relative movements between the ground and pile shaft surrounded by the ground reach more than 1% of the pile diameter. In this study, the ultimate resistance is defined as the load at which the relative pile movement reaches 1% of the pile diameter. The maximum resistance measured during testing was considered as the resistant capacity, even though the pile movement did not reach 1% of the pile diameter.

4.2 Evaluation of Resistant Capacity of Rock Socket

In brittle rock, the maximum shaft resistance is generally mobilized at the early stage of displacement and the maximum resistant value drops abruptly as the displacement increases further. On the other hand, the maximum load capacity of the end bearing is mobilized at large displacement. Therefore, the total capacity of a pile in brittle rock could be overestimated if the maximum shaft resistances and the maximum end bearing values are summed.

However, if there is no displacement softening, then the maximum shaft resistances and end bearing capacities can be summed, regardless of the amount of displacement.

For the cases of weathered and soft rock in this project, displacement softening does not occur, as shown in Figure 5. Thus, the total resistance of the piles was obtained by summing the shaft and the point resistance.

4.3 Resistance Factors for O-cell Tests

4.3.1 Resistance Factors for Conventional Load Tests

Resistance factors are suggested in NCHRP REPORT 507, according to the variable ground conditions at the site (site variability) and the number of pile load tests, as shown in Table 2.

<table>
<thead>
<tr>
<th>Number of pile load tests</th>
<th>Site variability</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Low</td>
</tr>
<tr>
<td>1</td>
<td>0.8</td>
</tr>
<tr>
<td>2</td>
<td>0.9</td>
</tr>
<tr>
<td>3</td>
<td>0.9</td>
</tr>
<tr>
<td>≥ 4</td>
<td>0.9</td>
</tr>
</tbody>
</table>
4.3.2 Resistance Factors for O-cell Tests

From a number of codes and specifications, it is known that the difference in the resistance factors in conventional load tests and dynamic load tests is about 0.1. The reliability of an O-cell test is generally considered to be in-between that of conventional load tests and that of dynamic load tests. Therefore, in this study, 0.05 is deducted from all values suggested above.

Resistance factors are proposed in this study in accordance with the suggestions in NCHRP 507, provided that the site variability is high.

5. Ultimate Resistant Capacity

Shaft resistances in weathered rocks, soft rocks and hard rocks are evaluated from four O-cell test results. The end bearing in hard rock is also evaluated.

5.1 Weathered Rocks

5.1.1 Shaft Resistance

Resistance factors and unit skin friction in weathered rock are evaluated, as shown in Table 3, in relation to N values.

5.1.2 End Bearing Capacity in Weathered Rock

For weathered rock with \( N \leq 50/10 \), the resistance factor, \( \phi = 0.5 \) and unit end bearing capacity = 300 t/m² are adopted.

5.2 Soft Rocks

5.2.1 Shaft Resistance

5.2.1.1 For Point Load Test Data

Regression curve of shaft resistances vs point load strength is shown in Figure 6. If the point load strength is greater than 3.5 Mpa, then the unit shaft resistance = 120 t/m² and resistance factor \( \phi = 0.7 \) are applied. If point load strength is smaller than 3.5 Mpa, then Equation 1 and \( \phi = 0.7 \) are applied.

Unit shaft resistance (t/m²)
\[
= 120 - 0.22 \times [3.5 - \text{Point load strength (Mpa)}] \quad (1)
\]

5.2.1.2 For Uniaxial Compressive Strength (UCS) Data

Uniaxial compressive strengths and shaft resistances are correlated, as shown in Figure 7. Equation 2 represents the correlation.

Table 3. Shaft resistances in weathered rock

<table>
<thead>
<tr>
<th>N</th>
<th>Shaft resistances (t/m²)</th>
<th>Resistance factors</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>N&lt;50/5</td>
<td>60</td>
<td>0.75</td>
<td>No. of PLT&gt;4</td>
</tr>
<tr>
<td>50/5 ≤ N ≤ 50/10</td>
<td>30</td>
<td>0.6</td>
<td>No. of PLT=2</td>
</tr>
<tr>
<td>50/10 ≤ N ≤ 50/15</td>
<td>20</td>
<td>0.5</td>
<td>Design of specification of bridge for roads (2001)</td>
</tr>
</tbody>
</table>

Table 4. End bearings resistance in weathered rocks

<table>
<thead>
<tr>
<th>N</th>
<th>Unit end veering capacities (t/m²)</th>
<th>Resistance factors</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>N ≤ 50/10</td>
<td>300</td>
<td>0.5</td>
<td>Design specification of bridge for roads (2001)</td>
</tr>
</tbody>
</table>
If UCS is greater than 12 Mpa, then unit shaft resistance = 120 t/m² and resistance factor $\phi = 0.7$ are applied. If UCS is less than 3.5 Mpa, then Equation 2 and $\phi = 0.7$ are applied.

Unit shaft resistance (t/m²)
$$= 120 - 1.84 \times [12 - \text{UCS (Mpa)}]$$ \hspace{1cm} (2)

5.2.1.3 For Core Recovery (TCR) Data
TCRs of soft rock and shaft resistances are correlated, as shown in Figure 8. Equation 3 represents the correlation.

If TCR is greater than 52, then unit shaft resistance = 120 t/m² and resistance factor $\phi = 0.7$ are applied. If TCR is less than 52, then Equation 3 and $\phi = 0.7$ are applied.

Shaft resistance (t/m²)
$$= 120 - 0.56 \times [52 - \text{TCR (%)}]$$ \hspace{1cm} (3)

In Table 5, unit shaft resistances are summarized for soft rock.

5.2.2 End Bearing Resistance
5.2.2.1 For Elastic Modulus Data
Elastic modulus and end bearing resistances are correlated, as shown in Figure 9. Equation 4 represents the correlation.

If the elastic modulus is greater than 2100 Mpa, then unit end bearing resistance = 700 t/m² and resistance factor $\phi = 0.6$ are applied. If the elastic modulus is less than 2100 Mpa, then Equation 4 and $\phi = 0.6$ are applied.

Unit end bearing resistance (t/m²)
$$= 700 - 0.22 \times [2100 - \text{elastic modulus (Mpa)}]$$ \hspace{1cm} (4)

5.2.2.2 For Point Load Test Data
Point load strengths and end bearing resistances are correlated, as shown in Figure 10. Equation 5 represents the correlation.

Unit end bearing resistance (t/m²)
$$= \text{The smaller between } 120 - 0.22 \times [3.5 - \text{Point load strength (MPa)}] \text{ and } 120 - 1.84 \times [12 - \text{UCS (MPa)}]$$

Table 5. Shaft resistances in soft rock

<table>
<thead>
<tr>
<th>Point load strength (MPa)</th>
<th>UCS (MPa)</th>
<th>TCR (%)</th>
<th>Ultimate unit shaft resistances (t/m²)</th>
<th>Resistance factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Above 3.5</td>
<td>Above 12</td>
<td>–</td>
<td>120</td>
<td>0.7</td>
</tr>
<tr>
<td>Below 3.5</td>
<td>No test results or above 12</td>
<td>–</td>
<td>$120 - 0.22 \times [3.5 - \text{Point load strength (Mpa)}]$</td>
<td>0.7</td>
</tr>
<tr>
<td>No test results or above 3.5</td>
<td>Below 12</td>
<td>–</td>
<td>$120 - 1.84 \times [12 - \text{UCS (MPa)}]$</td>
<td>0.7</td>
</tr>
<tr>
<td>Below 3.5</td>
<td>Below 12</td>
<td>–</td>
<td>The smaller between $120 - 0.22 \times [3.5 - \text{Point load strength (Mpa)}]$ and $120 - 1.84 \times [12 - \text{UCS (MPa)}]$</td>
<td>0.7</td>
</tr>
<tr>
<td>No test results</td>
<td>No test results</td>
<td>52 above</td>
<td>120</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>52 below</td>
<td>$120 - 0.56 \times [52 - \text{TCR (%)}]$</td>
<td>0.7</td>
</tr>
</tbody>
</table>

Fig. 8. TCR vs shaft resistance in soft rocks

Fig. 9. Elastic modulus vs end bearing resistance

Fig. 10. Point load test strength vs end bearing resistance
If the point load strength is greater than 20 Mpa, then unit end bearing resistance = 700 t/m² and resistance factor $\phi = 0.6$ are applied. If the elastic modulus is less than 20 Mpa, then Equation 5 and $\phi = 0.6$ are applied.

Unit end bearing resistance (t/m²)  
= $700 - 11.33 \times [20 - \text{Point load strength (Mpa)}]$  \hspace{5cm} (5)

5.2.2.3 For Uniaxial Compressive Strength (UCS) Data
UCSs and end bearing resistances are correlated, as shown in Figure 11. Equation 6 represents the correlation.

If UCS is greater than 12 Mpa, then unit end bearing resistance = 700 t/m² and resistance factor $\phi = 0.6$ are applied. If UCS is less than 12 Mpa, then Equation 6 and $\phi = 0.6$ are applied.

Unit end bearing resistance (t/m²)  
= $700 - 13 \times [12 - \text{UCS (Mpa)}]$  \hspace{5cm} (6)

5.2.2.4 For Core Recovery (TCR) Data
TCRs and end bearing resistances in soft rock are correlated, as shown in Figure 12. Equation 7 represents the correlation.

If TCR is greater than 80, then unit end bearing resistance = 700 t/m² and resistance factor $\phi = 0.5$ are applied. If TCR is less than 80, then Equation 7 and $\phi = 0.5$ are applied.

Unit end bearing resistance (t/m²)  
= $700 - 0.98 \times [80 - \text{TCR (%)}]$  \hspace{5cm} (7)

End bearing resistances for soft rock are summarized in Table 6.

5.3 Hard Rocks

5.3.1 End Bearing Resistance

Based on the test result of TP-1, if UCS is greater than 100 Mpa and point load strength is bigger than 80 Mpa, then unit end bearing resistance = 1800 t/m² and resistance factor $\phi = 0.5$ are applied (only one pile load test). If UCS is less than 100 Mpa, then Equation 8 $\phi = 0.5$ (only one

---

Table 6. End bearing resistances in soft rock

<table>
<thead>
<tr>
<th>UCS (MPa)</th>
<th>Point load strength (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>TCR (%)</th>
<th>End bearing capacities (t/m²)</th>
<th>Resistance factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Above 12</td>
<td>Above 20</td>
<td>–</td>
<td>–</td>
<td>700</td>
<td>0.6</td>
</tr>
<tr>
<td>Below 12</td>
<td>No test results or above 20</td>
<td>–</td>
<td>–</td>
<td>$700 - 13 \times [12 - \text{UCS (MPa)}]$</td>
<td>0.6</td>
</tr>
<tr>
<td>No test results or above 12</td>
<td>Below 20</td>
<td>–</td>
<td>–</td>
<td>$700 - 11.33 \times [20 - \text{Point load strength (MPa)}]$</td>
<td>0.6</td>
</tr>
<tr>
<td>Below 12</td>
<td>Below 20</td>
<td>–</td>
<td>–</td>
<td>The smaller between $700 - 13 \times [12 - \text{UCS (MPa)}]$ and $700 - 11.33 \times [20 - \text{Point load strength (MPa)}]$</td>
<td>0.6</td>
</tr>
<tr>
<td>No test results</td>
<td>No test results</td>
<td>Above 2100</td>
<td>–</td>
<td>700</td>
<td>0.6</td>
</tr>
<tr>
<td>No test results</td>
<td>No test results</td>
<td>Below 2100</td>
<td>–</td>
<td>$700 - 0.22 \times [2100 - \text{Elastic modulus (MPa)}]$</td>
<td>0.6</td>
</tr>
<tr>
<td>No test results</td>
<td>No test results</td>
<td>No test results</td>
<td>Above 80</td>
<td>700</td>
<td>0.6</td>
</tr>
<tr>
<td>No test results</td>
<td>No test results</td>
<td>No test results</td>
<td>Below 80</td>
<td>$700 - 0.98 \times [80 - \text{TCR (%)}]$</td>
<td>0.6</td>
</tr>
</tbody>
</table>
End bearing resistance \((t/m^2)\) is applied as in Figure 11.

\[
\text{End bearing resistance } (t/m^2) = 1800 - 13 \times [100 - USC (Mpa)] \tag{8}
\]

End bearing resistances for hard rock are summarized in Table 7.

### 6. Comparisons of Ultimate Capacities

To evaluate the ultimate capacities, Davisson method and Debeer method are applied for the equivalent load-displacement curve (Figure 14), which is reproduced from O-cell test results (Figure 13). The capacities obtained from these curves are then compared to the capacities calculated from the correlations proposed above. The summary of the comparisons are listed in Table 8. In Table 9, settlements of pile heads are compared. These comparisons show that the capacities calculated for each layer from the correlations proposed above are on the conservative side.

### 7. Conclusions

Conclusions are summarized as below.

1. Test loads of 140000, 170000, 240000, 290000 kN
(14000, 17000, 24000, 29000 ton) were achieved successfully from 4 full scaled pile load tests with O-cells.

(2) The results of the four O-cell tests confirmed the displacement hardening behavior in shaft resistance-displacement curves of weathered and soft rocks. In this regard, shaft resistances and end bearing resistance were summed to evaluate the total resistance in the Incheon bridge project. However, in hard rock, only one resistant value from the two was recommended according to the relative displacements.

(3) Multiple resistance factors accounting for site variability and the number of pile load tests were proposed referring to NCHRP REPORT 507. The overall reduction in resistance factors considering the reliability of the O-cell tests, which is higher than that of dynamic load tests and lower than that of conventional load tests, was also applied.

Acknowledgment

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(received on Feb. 12, 2009, accepted on Mar. 15, 2009)
Effect of Cementation on Deformation Modulus of Sand

사질토의 변형계수에 미치는 고결영향 분석

Lee, Moon-Joo1 이문주
Hong, Sung-Jin2 홍성진
Kim, Jae-Jeong3 김재정
Lee, Woo-Jin4 이우진

요  지

본 연구에서는 사질토의 고결이 미소변형 전단탄성계수\(G_{\text{max}}\), 딜라토미터 계수\(E_D, M_D\) 그리고 횡방향구속 변형계수\(M\)과 같은 변형특성에 미치는 영향을 평가하였다. \(G_{\text{max}}\)와 \(E_D, M_D\)는 각각 챔버에 조성된 석고 고결시료에 대한 벤더엘리먼트 시험과 딜라토미터 시험으로 측정하였고, \(M\)은 동일한 과정으로 준비된 시료에 대한 압밀셀 압축시험으로 결정하였다. 시험결과, \(G_{\text{max}}\)는 석고함유율에 따라 지수적으로 증가한 반면 \(E_D, M\)은 선형 증가하였다. \(G_{\text{max}}\)는 \(E_D\), \(M\)의 순서로 고결에 민감한 것으로 관찰되었으며, 딜라토미터 관입으로 인한 고결결합의 손상으로 \(E_D\)에는 고결영향이 상대적으로 적게 반영되었다. 따라서, \(E_D\)로부터 추정된 딜라토미터 횡방향변형계수\(M_D\)는 실측치\(M\)보다 상당히 과소평가되었다. 고결여부에 상관없이 \(G_{\text{max}}/M_D\)값은 \(K_D\)가 증가하였다고 감소하지 않지만 고결토의 \(G_{\text{max}}/M_D\)값은 비고결토에 비해 상당히 큰 것으로 관찰되었다. 따라서 \(G_{\text{max}}/M_D-K_D\)관계를 이용하면 현장시험결과로부터 지반의 고결여부를 판단할 수 있다.

Abstract

This study investigates the effect of cementation on deformation characteristics of sands such as small-strain shear modulus \(G_{\text{max}}\), dilatometer modulus \(E_D\) or \(M_D\), and constrained modulus \(M\). A series of DMT and bender element tests are performed on cemented sands prepared in calibration chamber, and one dimensional compression tests in oedometer cell are conducted. It is shown from the test results that, as the cementation degree increases, \(G_{\text{max}}\) increases exponentially while \(M\) and \(E_D\) increase almost linearly. Among three moduli, \(G_{\text{max}}\) appears most sensitive to the cementation while \(E_D\) is least sensitive due to the damage of cementation bonds induced during DMT penetration. It is also observed that \(M_D\) significantly underestimates \(M\) of cemented sand. Although both cemented and un-cemented sands show similar decreasing trends of \(G_{\text{max}}/M_D\) with increasing \(K_D\), the cementation induces the significant increase in \(G_{\text{max}}/M_D\) value. Therefore, it is expected that \(G_{\text{max}}/M_D-K_D\) relation can be used to determine if the deposit is cemented or not.

Keywords : Cementation, Constrained modulus, Dilatometer modulus, Small-strain shear modulus

1. Introduction

The deformation modulus is an important soil parameter for predicting the behavior of granular soil under applied loads (Lambrechts and Leonards, 1978). Generally, the compressibility of sand is affected by two principle groups:

1 Member, Research Scholar, School of Civil, Environmental and Architectural Engrg., Korea Univ.
2 Member, PhD Student, School of Civil, Environmental and Architectural Engrg., Korea Univ.
3 Member, Graduate Student, School of Civil, Environmental and Architectural Engrg., Korea Univ.
4 Member, Associate Prof., School of Civil, Environmental, and Architectural Engrg., Korea Univ., woojin@korea.ac.kr, Corresponding Author
state and intrinsic variables. Well-known state variables affecting compressibility are the stress level, the pre-stressing and void ratio. And the group of intrinsic variables includes the particle characteristics such as angularity, particle size and composition (Clayton et al., 1985).

Cementation is another dominant influencing factor on the deformation modulus of sand. From various laboratory tests using artificially cemented specimen, the effect of cementation on the deformation modulus of sand has been widely discussed. Coop and Atkinson (1993), Huang and Airey (1993), Cuccovillo and Coop (1999), Schnaid et al. (2001), Haeri et al. (2005) and Lee et al. (2009) observed that the cemented sand shows a stiff behavior and the stiffness increases with the increase in the content of cementing agent. Cementation is also an important influencing factor on small-strain stiffness of sand. From various studies (Chiang and Chae, 1972; Acar and El-Tahir, 1986; Saxena et al., 1988; Chang and Woods, 1992; Baig et al., 1997; Fernandez and Santamarina, 2001; Mohsin and Airey, 2005; Yun and Santamarina, 2005), it was shown that the increase in the cementing agent content results in a significant increase in the small-strain shear modulus, G_{max}, due to strong cementation bonds between particle contact points. Unlike the un-cemented sand, the deformation modulus of cemented sand is only partially affected by confining stress (Schnaid et al., 2001; Haeri et al., 2005; Baig et al., 1997; Fernandez and Santamarina, 2001; Mohsin and Airey, 2005). However, because the cementation bonds are broken at about 1% strain (Saxena and Lastrico, 1978), the application of a confining pressure larger than the seating pressure will cause the damage of cementation bonds and reduce the deformation modulus of cemented sand (Baig et al., 1997; Fernandez and Santamarina, 2001; Yun and Santamarina, 2005).

Although the non-linear behavior of soil before failure makes the interpretation of in situ test results complicate, the deformation characteristics of soil sediments are practically evaluated from various in-situ tests because it is difficult and unreliable to obtain the undisturbed sample (Bellotti et al., 1986). Especially, the dilatometer test (DMT) is an effective in-situ test for evaluating the deformation characteristic of soil. However, only a few papers on dilatometer test on cemented soil have been published (Cruz and Fonseca, 2006; Fonseca et al., 2008). Cruz and Fonseca (2006) tried to evaluate the stiffness and strength property of residual soil, which shows similar behavior to cemented soil, from DMT and CPT. They suggested that DMT insertion causes the smaller disturbance and DMT is more sensitive to the stiffness variation than CPT. More recently, Fonseca et al. (2008) observed that the dilatometer horizontal stress index (K_D) of residual soil is larger than that of un-cemented sand at the same value of q_c/\sigma_v'.

This study investigates the effect of cementation of granular soil on three deformation moduli. One is the small-strain shear modulus, G_{max}, which is a fundamental soil property for predicting the liquefaction potential and the dynamic response of soil. Another is the constrained modulus, M, which is a simple and efficient property estimating the deformation characteristics of soil sediment. Finally, the dilatometer modulus (E_D or M_D) of cemented sand is included because it is a practical one obtained from the in-situ measurements to evaluate the stiffness of the soil. From experimental results, the influence of cementation on G_{max}, M and E_D (or M_D) is compared and, then, relationships between each modulus are analyzed.

2. Experimental Program

2.1 Sand and Cementing Agent

The artificially crushed sand (K-7 sand) is used for tests in this study. The particle size distribution and physical properties of K-7 sand are presented in Fig. 1 and Table 1. K-7 sand is classified as SP according to the unified soil classification system (USCS) and its D_{50} is 0.17 mm. The roundness of K-7 sand is identified as sub-angular and the content of SiO_2 is about 98%. In this study, gypsum, which is generally used for manufacturing ceramics, is used as a cementing agent because the behavior of gypsum-cemented sand is similar to that of naturally cemented sand (Ismail et al., 2002). The compressive strength of gypsum cured at 40% water content is approximately 20 MPa. The expansion rate of the gypsum
during curing is about 0.03%, which is relatively small compared with that of ordinary gypsum.

2.2 Calibration Chamber Tests

2.2.1 Specimen Preparation
The cemented specimens are prepared in a double wall cylindrical calibration chamber of 1.0 m high and 1.2 m wide. The chamber system has a hydraulic piston at the bottom and a top plate with adaptors (Fig. 2). The hydraulic pressures in the inner and outer cells of the chamber control the horizontal boundary condition of the specimen. Vertical stress is applied by the piston assembly located below the specimen. For the fabrication of a uniform specimen in the chamber, a rainer system, which consists of a 1.0 m high split mold, a 1.0 m high extension tube, a 1.2 m high sand storage and two diffuser sieves, is used. A constant drop height is maintained during pluviation by using four strings, which connect the diffuser system to the cover plate.

To minimize the potential of particle segregation between sand and gypsum particles during air pluviation, the pre-wetting method (Rad and Tumay, 1986; Puppala et al., 1995) is adopted in this study. An amount of water equivalent to 0.5% water content is manually mixed with dry sand. After 5, 7, and 10% weight of gypsum is added to the pre-wetted sand, both materials are re-mixed. The pre-wetting process moistens the surface of the sand particles and allows the grains to be uniformly and homogeneously coated with gypsum particles. After pluviating the sand or sand-gypsum mixture, the chamber system is assembled,

Table 1. Engineering properties of K-7 sand

<table>
<thead>
<tr>
<th>Gs</th>
<th>D10 (mm)</th>
<th>D50 (mm)</th>
<th>Cu</th>
<th>Cc</th>
<th>θmax</th>
<th>θmin</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.647</td>
<td>0.09</td>
<td>0.17</td>
<td>2.111</td>
<td>0.988</td>
<td>1.054</td>
<td>0.719</td>
<td>SP</td>
</tr>
</tbody>
</table>

Table 2. Test program for calibration chamber study

<table>
<thead>
<tr>
<th>Content of gypsum (Cg, %)</th>
<th>Relative density* (Dr, %)</th>
<th>Vertical confining stress (σv', kPa)</th>
<th>No. of specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 (un-cemented)</td>
<td>33–76</td>
<td>50, 100, 200, 400</td>
<td>20</td>
</tr>
<tr>
<td>5</td>
<td>36, 57, 74</td>
<td>50, 100, 200</td>
<td>9</td>
</tr>
<tr>
<td>7</td>
<td>36, 51, 63</td>
<td>50, 100, 200</td>
<td>9</td>
</tr>
<tr>
<td>10</td>
<td>21, 39, 52</td>
<td>50, 100, 200</td>
<td>9</td>
</tr>
</tbody>
</table>

* Relative density of soil skeleton after applying confining stress.

Fig. 1. Particle size distribution of K-7 sand

Fig. 2. Calibration chamber system for bender element and dilatometer tests
and then vertical stress and corresponding $K_0$ horizontal stress are applied to the specimen under boundary condition 1, which leads to the constant horizontal and vertical stresses during penetration. To induce the cementation of the sand-gypsum mixture, distilled water is injected from the bottom of the specimen with 30 kPa hydraulic pressure after the application of confining stress, and the specimen is cured for 24 hours. Table 2 shows the condition of prepared un-cemented and cemented specimens.

### 2.2.2 Bender Element Test

After curing, the shear wave velocity is measured from the bender element test. This study uses parallel-type bender elements to minimize electronic coupling between the source and receiver, which is called crosstalk. Crosstalk is a very important factor in conductive soils such as wet marine clays (Lee and Santamarina, 2005). The dimensions of each bender element are 20 mm in length, 10 mm in width, and 0.6 mm in thickness. The cantilever length within the soil is 4 mm. Based on Lee (2003), the bender elements are connected to coaxial cables and coated by polyurethane, and are electrically shielded with the conductive paint, grounded, and anchored into nylon set screws as shown in Fig. 3. To measure the velocity of the shear wave propagating in the horizontal direction, $V_{sHV}$, two pairs of bender elements are mounted in the rod at 35 cm and 65 cm depths from the top of the specimen.

A 20 MHz waveform generator is used to generate a single sinusoidal signal, which is then amplified by a power amplifier. The shear wave produced by bending the source element propagates through the specimen and is detected by the receiver bender element. The detected signal is fed through a filter-amplifier to remove the noise. The travel time of the shear wave is evaluated from the amplified signal recorded by the digital oscilloscope. The traveling distance is the tip-to-tip distance between the source and the receiver bender elements (Dyvik and Madshus, 1985; Viggiani and Atkinson, 1995; Fernandez, 2000). The traveling distance in this study is relatively long (104 cm), and the ratio of the tip-to-tip distance ($L$) to the shear wave wavelength ($\lambda_s$) is approximately 20. Therefore, no near field effect is expected (Lee and Santamarina, 2005).

### 2.2.3 Dilatometer Test

The dilatometer is a 14 mm thick, 95 mm wide and 220 mm long flat plate with 20° apex angle. A flexible stainless steel membrane of 60 mm diameter is located on one face of the blade. After the completion of bender element test, the flat dilatometer is penetrated with a 2 cm/sec penetration rate. By measuring $P_0$ and $P_1$ pressures at every 10 cm penetration from 30 to 70 cm, the horizontal stress index, $K_D=(P_0-u_0)/\sigma_{v0}'$, the material index, $I_D=(P_1-P_0)/(P_0-u_0)$, and the dilatometer modulus, $E_D=34.6\,(P_1-P_0)$, are evaluated. Here, $u_0$ is the hydrostatic pressure and $\sigma_{v0}'$ is the vertical effective stress.

### 2.3 One Dimensional Compression Test

The constrained modulus of a cemented specimen cannot be measured directly by penetration tests because of the cementation damage induced during penetration. Therefore, the constrained modulus of a cemented specimen is evaluated from one-dimensional compression tests in an oedometer cell of 74 mm in diameter and 45 mm in height. An un-cemented specimen is pluviated in the cell, and vertical stress is incrementally increased by 25 kPa up to 500 kPa to obtain the $e-\sigma_v'$ relation. Cemented specimens
are prepared by pluviating the sand-gypsum mixture in the cell and, after applying vertical stress (50, 100, 200 kPa), distilled water is supplied through the bottom of the specimen for curing. The specimen is cured for 24 hours, and then the vertical load is incrementally increased to evaluate the constrained modulus, following the same procedure for an un-cemented specimen.

3. Test Results

Fig. 4 shows typical shear wave signatures measured for the specimen of 40% initial relative density. Upper seven signals are the signatures of un-cemented specimen measured during loading stages up to 200 kPa vertical stress, and the last one is the signature measured after the cementation. Despite the relatively far tip-to-tip distance of bender elements, the measured signals are good enough to recognize the first arrival of the shear wave. It is shown that the travel time gradually decreases with the increase of applied vertical effective stress for un-cemented sand. Also observed is the significant decrease in travel time due to cementation.

Fig. 5 is the corrected DMT readings \((P_0, P_1)\) for un-cemented and cemented K-7 sands of about 40% relative density under 100 kPa vertical stress in calibration chamber. Test result shows almost constant \(P_0\) and \(P_1\) values from 30 to 70 cm depth. With increasing the gypsum content, \(P_0\) and \(P_1\) pressures are observed to increase.

Fig. 6 represents the \(e-\sigma'\) relation obtained from one dimensional compression tests for un-cemented and cemented sands. While the void ratio of un-cemented sand decreases monotonically with increasing stress level, cemented sands show clear yielding points, at which the settlement abruptly
increases due to the destruction of cementation bonds. The yield stress of cemented sand increases with increasing the cementation level. The effect of cementation appears more important for stress level below apparent yielding stress that lies on the normal compression line for the de-structured soil (Huang and Airey, 1993, 1998). Therefore, the constrained modulus of cemented sand is determined from the linear section between initial state and yielding point of the e-σv’ curve.

4. Analysis and Discussion

4.1 Sensitivity to Cementation

In general, factors affecting the behavior of granular soil have different degrees of influence on the in-situ and laboratory test results. As the relative density increases, the modulus of un-cemented sand increases more or less linearly while the cone resistance increases exponentially (Jamiolkowski et al., 1988). In addition, the modulus - penetration resistance ratio (M/qc or E/qc) is much higher for OC sand than for NC sand, due to greater effect of stress history on deformation modulus than penetration resistance.

Fig. 7 shows that the cementation also has different degree of influence on Gmax, M and Eo. The cementation effect on each measurement is represented by a ratio between the test result of cemented sand and that of un-cemented one under same relative density and vertical effective stress. It can be observed that there is a significant increase in the measured property with increase in the gypsum content. Regardless of the gypsum content, the rate of increase is the largest for Gmax and the smallest for Eo. For 5~10% gypsum content, Eo of cemented sand is about 2.9~6.3 times larger than that of un-cemented sand; about 9.9~15.3 times increase in M; and about 16.3~65.3 times increase in Gmax. This result means that the cementation of sand is not fully reflected in the penetration test results because the penetration of in situ equipment induces damage of the cementation bonds. Fig. 7 also shows that the cemented/un-cemented ratios of M and Eo increase almost linearly with increasing gypsum content, whereas, the ratio of Gmax increases exponentially. Therefore, it is concluded that Gmax is more sensitive to the cementation than M and Eo.

4.2 Estimation of Constrained Modulus from DMT

In general, the constrained modulus of un-cemented soil can be predicted using Marchetti (1980), M0=EoRM, in which RM is a correction factor related with KD and ID. It has been known that predicted M0 is reasonably accurate, compared with measured constrained modulus. In this study, M0 of cemented sand is evaluated using Marchetti
(1980), although the applicability of correction factor $R_M$ for cemented sand has not been verified.

Fig. 8 is the comparison between $M$ measured from oedometer test and $M_D$ evaluated using Marchetti (1980). It is shown that $M_D/M$ ratios of un-cemented K-7 sand are distributed in the range of $0.7 \sim 1.2$ (0.8 in average). This result agrees with Jamiolkowski et al. (1988) and Bellotti et al. (1997), in which the slightly smaller $M_D$ than the measured $M$ was observed from calibration chamber studies. However, $M_D$ of cemented sands in Fig. 8 is underestimated by about $30 \sim 78\%$, compared with the $M$ value. This is because, although the horizontal stress index $K_D$ increases due to the cementation, increased $K_D$ does not provide a reasonable $R_M$ value that successfully reflects the cementation effect. This result demonstrates that DMT indices do not successfully evaluate the deformation properties of cemented sand due to the damage of cementation bonds during penetration. Other interesting observation in Fig. 8 is that the $M_D/M$ of cemented sands almost linearly increases with the increasing $K_D(\sigma_v'/p_a)^{0.5}$ and the $M_D/M-K_D(\sigma_v'/p_a)^{0.5}$ relation is similar, regardless of the cementation level.

Fig. 9 shows the effect of relative density and gypsum content on $M_D/M$. As shown in Fig. 9 (a), with increasing gypsum content, the magnitude of $M_D$ of cemented sand approaches to that of measured $M$. It means that the damage of cementation due to penetration of DMT blade is less significant at higher level of cementation. It is observed from Fig. 9 (b) that $M_D/M$ ratio of cemented sand steadily increases as the relative density increases. Therefore, it may be suggested that the effect of cementation on $M_D/M$ is more significant than that of relative density.

4.3 Relation Between $G_{\max}$ and DMT Indices

With the development of seismic dilatometer (SDMT), which is the combination of a standard DMT with a seismic module measuring shear wave velocity, Marchetti et al. (2008) presented the $G_{\max}/M_D-K_D$ relations for various natural soils. As shown in Fig. 10, $G_{\max}/M_D$ of un-cemented

![Image](image-url)
K-7 sand decreases with increasing K_D and the relationship proposed by Marchetti (2008) locates near the upper limit of database. It is also observed that the magnitude of G_{\text{max}}/M_D of cemented K-7 sand is considerably larger than that of un-cemented sand although the trend of decreasing G_{\text{max}}/M_D with K_D is quite similar for both un-cemented and cemented sands. This is because G_{\text{max}} is more sensitive to the cementation than M_D. Therefore, it is expected that the G_{\text{max}}/M_D ratio of sand can be used to determine if the deposit is cemented or not.

It is interesting to note that the 5% and 7% cemented sands have almost identical value of G_{\text{max}}/M_D in the range of 9.1\sim 14.3 whereas G_{\text{max}}/M_D of 10% cemented sand is almost 1.4\sim 2.0 times larger than those of 5% and 7% cemented sands. This is due to the exponential increase in G_{\text{max}} with cementation level up to 10% gypsum content. As mentioned by Chang and Woods (1992), at low level of cementation, most of cement particles are coated on the sand surface and only contribute to the small portion of increase in stiffness. However, the contribution of cementing agent to the contact bonds grows more and more as the content of cementing agent increases.

4.4 Relation between G_{\text{max}} and Constrained Modulus

Fig. 11 presents the variation of G_{\text{max}}/M ratio of un-cemented and cemented K-7 sands in terms of normalized horizontal stress index, K_D(\sigma_v'/p_a)^{0.5}. With the increasing K_D(\sigma_v'/p_a)^{0.5}, the G_{\text{max}}/M of un-cemented sand slightly decreases. This is because, although the increase in relative density induces the increase in both measurements, M value of un-cemented sand is more heavily affected by relative density than G_{\text{max}}. Whereas, G_{\text{max}}/M of cemented sand has a slightly increasing trend with increasing K_D(\sigma_v'/p_a)^{0.5} and its value becomes larger as the cementation level increases. Above observations indirectly confirm that, for cemented sand, both of relative density and gypsum content have more significant effect on G_{\text{max}} than M.

5. Summary and Conclusions

This study is concerned with the evaluation of deformation modulus of cemented sand and the effect of cementation on deformation modulus of sand. Three types of deformation moduli (G_{\text{max}}, M, E_D) are evaluated using laboratory and in-situ tests for both un-cemented and artificially gypsum-cemented specimens.

Test results show that cementation has considerable influence on deformation characteristics of sand. The small-strain shear modulus is most sensitive to the cementation among deformation moduli considered in this study. Whereas, the smallest effect of cementation on dilatometer modulus represents the damage of cementation bonds due to the penetration of DMT blade. The G_{\text{max}} is a constant shear modulus at shear strain less than 10^{-3}\%., while the constrained modulus M is intermediate strain modulus of soil. In addition, DMT estimates the deformation characteristics of soil at intermediate strain level (Ishihara, 2001; Mayne, 2001). Therefore, it is obvious that the sensitivity of cementation to various soil properties depends on the strain level at which the properties are measured because the extent of cementation damage depends on the strain level induced due to the penetration. At smaller strain level, the measured soil property is more sensitive to cementation.

The prediction of constrained modulus using DMT results using previously suggested method is not relevant if it does not consider the cementation of soil sediments. It is observed that, for cemented sands, M_D evaluated using
Marchetti (1980) is in the range of 30~75% of measured M value. Since the effect of cementation depends on the strain level, the ratio between non-destructive and destructive measurements, such as G\text{max}/M\text{D}, can be used to determine whether the deposit is cemented or not. In this study, it is shown that G\text{max}/M\text{D} of cemented sand is significantly larger than that of un-cemented sand at the same K\text{D}. It is also found that both cementation and relative density have a larger influence on G\text{max} than M for cemented sand.

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Ground Settlement during Urban Tunneling in Groundwater Drawdown Environment - Case History and Numerical Analysis

1. Introduction

When tunneling under the groundwater table, the tunneling inevitably causes groundwater inflow into the excavated area to some extent, which in turn leads to groundwater drawdown. The effect of tunneling-induced groundwater drawdown can significantly increase ground settlements with larger settlement affected area when compared to a tunneling condition with no ground water drawdown. Also presented are the results of three-dimensional stress-pore pressure coupled finite element analysis on a tunneling situation similar to the case history. Based on the results the effects of groundwater drawdown on the ground settlement, pore water pressure, and lining stresses are discussed. Practical implications of the findings from this study are discussed.

Abstract

This paper discusses ground settlements during tunneling in groundwater drawdown environment in urban tunneling situations. The dynamics of the effect of groundwater drawdown on the ground settlement development are first illustrated by analyzing the measured data. It is shown that the tunneling-induced groundwater drawdown can significantly increase ground settlements with larger settlement affected area when compared to a tunneling condition with no ground water drawdown. Also presented are the results of three-dimensional stress-pore pressure coupled finite element analysis on a tunneling situation similar to the case history. Based on the results the effects of groundwater drawdown on the ground settlement, pore water pressure, and lining stresses are discussed. Practical implications of the findings from this study are discussed.

Keywords: Finite element method, Groundwater drawdown, Ground settlement, Stress-pore pressure coupled analysis, Tunneling
groundwater drawdown is to increase ground settlements due to increases in effective stresses in the ground in addition to the settlement due to unloading effect caused by the excavation (Yoo 2005, Yoo and Kim 2006), the degree of which depends on a number of factors such as hydraulic and geotechnical factors. In urban tunneling situations excessive settlements may occur in such tunneling environments, which in turn result in damages to nearby structures/utilities. One of the major case histories illustrating damage due to excessive ground settlements associated with tunneling-induced groundwater drawdown is perhaps the Romeriksporten tunnel in which the highspeed railway tunnel construction caused more than 1 m of ground subsidence due to groundwater drawdown, raising significant technical and political issues pertaining to the effect of tunneling on surrounding environment (NSREA 1995).

The magnitude and distribution of ground settlements for a given tunneling depend largely on ground properties, tunnel excavation geometry including depth and shape, support measures, groundwater control measures, and more importantly construction procedures. On account of the increased public concern on the effects of tunneling on surrounding environments, the prediction of potential effects of tunneling, in terms of surface settlements and groundwater drawdown, on surrounding environment has become an essential part of the planning, design, and construction of a tunnel in urban environments. Of the aforementioned influencing factors the appropriateness of the groundwater control measures may perhaps be the controlling factor for ground settlements in tunneling with groundwater drawdown environment.

There have been numerous incidents in which tunneling induced groundwater drawdown caused excessive settlements, particularly in urban areas. Fundamental mechanisms involved in the settlement development during such tunneling situations have not yet been fully understood. In this paper, the dynamics of the effect of groundwater drawdown on the ground settlement development are first illustrated based on the measured data. The results of three-dimensional (3D) finite element analyses are then presented so that the governing factors and the ground settlement can be related.

2. Case History

2.1 Site Condition

The tunnel section under consideration was constructed in a typical ground condition encountered in the vicinity of Seoul. The ground is a multi-layered ground including a fill, alluvium, and a weathered zone. Figure 1 illustrates the stratigraphy. The top layer is 3.5 m thick miscellaneous fill material of gravelly silty sand with SPT blow count (N) ranging from 15 to 30. Underlying the fill layer is a 1.5 to 6 m thick alluvial deposit of clayey sand having N from 30 to 35 followed by a 1.5 to 5 m thick decomposed granitic soil layer (N=10~50). The decomposed soil layer is underlain by a weathered granitic rock layer having N greater than 50. The weathered granitic rock layer is followed by a soft to hard granitic rock layer with a maximum RQD of 40. The hydraulic conductivities of the soil layers are on the order of 2~4×10^{-6} m/s. Due to the close proximity to Han River the groundwater table is relatively high at ground level (GL) -4~6 m, located in the alluvial layer. Figure 1 shows a typical tunnel cross section together with the ground profile.

2.2 Tunnel Design and Construction

A typical tunnel cross section is shown in Figure 2 with the support pattern adopted. As seen in this figure, the tunnel has a width and height of approximately 10.5 m and 8.7 m, respectively, with a cover depth ranging approximately from 20 to 30 m. The primary support system consisted of a 0.2 m thick steel fibre reinforced...
shotcrete (SFRS) layer with 4 m long system rock bolts at 1.0 and 1.2 m, respectively, longitudinal and transverse spacing. The pipe umbrella technique using 800 mm diameter grout injected 12 m long steel pipes was additionally implemented to promote the face stability through improving the load carrying capacity of the ground ahead of the face. Also adopted was a trumpet shaped micro cement injection (MSG) pre-grouting around the tunnel periphery to create a 5 m thick watertight shell for sections in which the weathered soil layer extended to the tunnel crown level. For the tunnel sections under consideration, two types of excavation methods, the bench-cut and the ring-cut, were adopted depending on the extent of the weathered soil with respect to the tunnel heading.

2.3 Measured Data

Figure 3 presents the measured longitudinal settlement profiles along the tunnel center line at various stages of tunneling for a 250 m long tunnel section. As shown in Figure 4, the progressive development of the settlement is well illustrated with the tunnel advancement. For example, the tunnel advancement gradually increased the longitudinal settlements both ahead and behind the top heading with a maximum settlement of approximately 160 mm occurring at STA22km880 upon completion of the tunnel excavation. Also noted in this figure is that for a given top heading location, the distance to which the settlement started to occur ahead of the top heading was approximately four to five tunnel diameter (D), which suggests the settlement susceptible zone extended up to $4 \sim 5D$ ahead of the top heading. Such a trend is well in accordance with the three dimensional numerical investigation reported by Yoo and Kim (2008) for an urban tunneling situation involving tunnel excavation through water bearing ground directly under a commercial building.

The settlement history plots for the three monitoring points installed at STA22K860, 870, and 880 are shown in Figure 4 from which the trend of progressive development of settlements with the top heading location can be identified. As shown it can be seen that for each monitoring point, the settlement started to develop approximately five tunnel diameters (D) ahead of the top heading and stabilized when the top heading advanced $5 \sim 6D$ from the monitoring point, which suggests a larger settlement affected zone than typical tunneling cases.

Also shown in Figure 5 is the progressive change in the groundwater level measured by a piezometer installed at station 22km780. Since the piezometer had been installed
prior to the tunneling commencement, the data can be considered to represent the general trend of the effect of tunneling on the groundwater regime. As shown, it can be observed that the tunneling started to influence the groundwater regime near the monitoring point by decreasing the initial groundwater level of GL -4 m to GL-12 m when the tunnel heading reached 3D away from the monitoring point. Upon arrival of the top heading at the monitoring point, a total of 16 m of drawdown was completed. The groundwater level dropped further with the tunnel advancement and stabilized at GL -29 m, resulting in a total groundwater drawdown of 25 m when the tunnel heading advanced approximately 2D beyond the monitoring section.

2.4 Observations

In general for tunneling cases with no significant groundwater drawdown, ground surface settlement at a monitoring point tends to start when the tunnel heading is two to three tunnel diameters (D) away from the monitoring point. The settlement also ceases to increase when the tunnel heading moves two to three tunnel diameters ahead of the monitoring point, which suggests the settlement zone being $2 \sim 3D$ ahead and behind the tunnel heading.

As illustrated in the previous section, however, when the tunneling has a significant impact on the groundwater drawdown, the settlements at the monitoring points started when the tunnel heading was at 5D away from the monitoring points and stabilized with the tunneling heading advancement of approximately 5D. Such observations suggest a considerably larger settlement influence zone and are in accordance with the settlement development characteristics discussed by Yoo (2005). The strong connection between the settlement development and the groundwater drawdown has been confirmed by the piezometric data presented earlier. In short, the settlement data together with the groundwater monitoring results provided evidences that the groundwater drawdown indeed played a significant role in the settlement development.

3. Three-Dimensional Stress-Pore Pressure Coupled Finite Element Analysis

In this section the results of a series of stress pore pressure coupled analyses on a tunneling case in which tunneling induces groundwater drawdown are presented to illustrate the importance of adopting the stress-pore pressure coupled formulation for settlement prediction in tunneling cases with groundwater drawdown. The analyses were conducted using Abaqus (2007).

3.1 Fundamentals of Stress-pore Pressure Coupled Formulation

In urban tunneling condition where prediction of settlement is of utter most importance a numerical model that is capable of describing the mechanical and hydrological interaction between the tunneling and the ground including groundwater should be adopted. Such an interaction can only be adequately taken into consideration when the adopted numerical scheme is based on the stress-pore pressure coupled formulation. In the coupled formulation a porous medium is modeled approximately by attaching the finite element mesh to the solid phase. A continuity equation, equating the rate of increase in liquid mass stored at a point to the rate of mass of liquid flowing into the point within the time increment, is written in a variational form as a basis for finite element approximation as Eq. (1):

$$\int \sigma : \delta a dV = \int t \cdot \delta v dS + \int \dot{f} \cdot \delta v dV$$  \hspace{1cm} (1)
where $dv$ is a virtual velocity field, $\mathbf{v}$ is the virtual rate of deformation, $\mathbf{\sigma}$ is the true (Cauchy) stress, $t$ is surface traction per unit area, and $\mathbf{f}$ body forces per unit volume. For coupled analysis, $\mathbf{f}$ includes the weight of the wetting liquid;

$$\mathbf{f} = (sn + n_i)\rho_w g$$

where $\rho_w$ is the density of the wetting liquid and $g$ is the gravitational acceleration, which we assume to be constant and in a constant direction. Considering this loading explicitly so that any other gravitational term in $\mathbf{f}$ is associated only with the weight of the dry porous medium, equation (1) can be rewritten as Eq. (3);

$$\int_V \sigma : \mathbf{\varepsilon} \mathbf{d}V = \int_S t \cdot \mathbf{d}S + \int_V \mathbf{f} \cdot \mathbf{d}V + \int_V (sn + n_i)\rho_w g \cdot \mathbf{\varepsilon} \mathbf{d}V$$

where $\mathbf{f}$ is every body force except the weight of the wetting liquid. The liquid flow is described by introducing Darcy’s law or, alternatively, Forchheimer’s law.

The continuity equation is satisfied approximately in the finite element model by using excess wetting liquid pressure as the nodal variable (degree of freedom 8), interpolated over the elements. The equation is integrated in time by using the backward Euler approximation. The total derivative of this integrated variational statement of continuity with respect to the nodal variables is required for the Newton iterations used to solve the nonlinear, coupled, equilibrium and continuity equations.

### 3.2 Tunneling Case Considered

To illustrate the effect of groundwater drawdown on the tunneling-induced ground settlement, a tunneling condition similar to the case history mentioned in the previous section was considered. It was assumed that the primary excavation method is the ring-cut excavation method. The effect of groundwater drawdown level on the tunneling performance was then examined. The groundwater drawdown was controlled by assigning different permeability values for the circumferential pregrouting around the tunnel. Three cases were considered; 1) no grouting (CASE A); 2) grouting with permeability of $k_g = 8 \times 10^{-6} \text{ m/s}$ (CASE B); and 3) $k_g = 8 \times 10^{-7} \text{ m/s}$ (CASE C)

### 3.3 Finite Element Modeling

Figure 6 shows the finite element model adopted in this study, consisting of approximately 80,000 nodes and 70,000 elements. The finite-element mesh extends to the solid rock layer and laterally to a distance of 100 m, or 10D, from the tunnel centerline. In prescribing displacement boundary conditions, displacements perpendicular to the lateral boundaries were restrained whereas pin supports were applied to the bottom boundary.

With regard to the hydraulic boundary conditions and with reference to Figure 6, the no-flow condition was assigned to the vertical boundaries perpendicular to the lateral boundaries. A constant head boundary condition was prescribed to the lateral boundaries.
so that pore pressures are maintained constant throughout the analysis at the initial groundwater level of GL -4 m. The locations of the lateral and bottom boundaries were selected so that the presence of the artificial boundaries does not significantly influence the stress-strain-pore pressure field in the domain.

In discretizing the model, the ground layers and the shotcrete lining were modeled using the 8-node brick element with reduced integration. For the ground under the groundwater table, full saturation was assumed and the stress-pore pressure coupled elements (C3D8RP) were used while for those above the groundwater table, the stress-only elements (C3D8R) were used. System rock bolts and fore poling which are usually adopted as part of the support were not modeled for simplicity.

In terms of the constitutive modeling, the soil and rock layers were modeled as elasto-plastic material conforming to the Mohr-Coulomb failure criterion together with the non-associated flow rule proposed by Davis(1968), while the shotcrete lining was assumed to behave in a linear elastic manner. Table 1 summarizes the geotechnical properties of the materials.

In the analysis, the initial geostatic stress and pore water pressure conditions were first established by prescribing appropriate boundary conditions. The actual excavation sequence for the ring-cut excavation method was then closely simulated by removing the corresponding excavation area. Full excavation of the tunnel through the entire domain was modeled, and then the analysis was continued until a steady state in terms of pore pressure was achieved.

### 3.4 Results and Discussion

Figure 7 shows the progressive development of the groundwater drawdown zone with the tunnel advancement for CASE A in which no circumferential pregrouting was considered. Note that the face distance from the portal to the tunnel heading is denoted as FD in this figure. As shown, as the tunnel advances, the drawdown zone, elliptical in shape, progressively expands in front and lateral directions. For example, when the tunnel heading advances 6 m, the drawdown zone extends approximately 2D and 3D, respectively, in front and lateral directions. With the further advancement to FD=38 m, the drawdown zone increases to 4D and 5.5D, respectively, in front and lateral directions. As ground settlements are closely related to the groundwater drawdown, the extent of the settlement influence zone can be expected in terms of this relation.

Figure 8 presents ground settlement history plots for the three cases monitored at a point 4D from the portal.

<table>
<thead>
<tr>
<th>Material</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$c'$ (kPa)</th>
<th>$\phi'$ (deg)</th>
<th>$E$ (MPa)</th>
<th>$\nu$</th>
<th>$K$ (cm/sec)</th>
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</thead>
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<tr>
<td>Fill</td>
<td>18</td>
<td>0</td>
<td>27</td>
<td>5</td>
<td>0.40</td>
<td>$3.8 \times 10^{-4}$</td>
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<tr>
<td>Alluvial</td>
<td>20</td>
<td>15</td>
<td>30</td>
<td>10</td>
<td>0.40</td>
<td>$3.8 \times 10^{-4}$</td>
</tr>
<tr>
<td>Weathered soil</td>
<td>25</td>
<td>15</td>
<td>30</td>
<td>20</td>
<td>0.33</td>
<td>$2.4 \times 10^{-4}$</td>
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<tr>
<td>Weathered Rock</td>
<td>25</td>
<td>60</td>
<td>35</td>
<td>120</td>
<td>0.30</td>
<td>$8.8 \times 10^{-5}$</td>
</tr>
<tr>
<td>Hard rock</td>
<td>26</td>
<td>100</td>
<td>35</td>
<td>200</td>
<td>0.25</td>
<td>$5.0 \times 10^{-5}$</td>
</tr>
</tbody>
</table>

Note: $\gamma$ = unit weight, $c'$=cohesion, $\phi'$=internal friction angle, $E$=Young’s modulus, $\nu$=Poisson’s ratio, $K$=coefficient permeability.

Fig. 7. Progressive development of groundwater drawdown zone.
As one would expect, the greater the groundwater drawdown $H_D$, the larger is the settlement. For example, the settlement increases by 35% when the drawdown level increases from 6 m to 13 m, demonstrating that the groundwater water drawdown in fact increases the ground surface settlement.

In Figure 9, the changes in pore pressure at the monitoring point located 15 m below the ground surface are illustrated for three different drawdown cases. As shown, the general shape of the curves tends to follow that of the settlement history curves shown in Figure 8. The initial pore water pressure of 120 kPa dropped to approximately 10 kPa for the case of $H_D = 13$ m. Also noticed in this figure is that more than 50% of the pore pressure decrease is completed before the tunnel heading passes the monitoring section. Such a trend sends a strong message that the postgrouting is no alternative to pre-grouting in view of groundwater control.

The tunneling induced groundwater drawdown may eventually reach a steady state condition upon completion of excavation. The hydraulic pressure on the permanent lining decreases with increasing the drawdown for a given condition. The shotcrete lining, which is installed during tunneling, is subject to groundwater seepage force during tunneling and therefore its stresses are somewhat governed by the transient nature of stresses. In Figure 10, axial stresses in the shotcrete lining are shown for the different cases. Unlike the settlement, the lining stresses are almost the same level regardless of the drawdown level. Unless the groundwater table is lowered prior to excavation, no significant variation in lining stresses can be expected for different levels of groundwater drawdown during tunneling.

4. Summary and Conclusions

This paper concerns a case history and numerical analysis on ground settlements during tunneling in groundwater drawdown environment in urban tunneling situations. The
dynamics of the effect of groundwater drawdown on the ground settlement development were illustrated by analyzing the measured data. It is shown that the tunneling-induced groundwater drawdown can significantly increase ground settlements with larger settlement affected area when compared to a tunneling condition with no ground water drawdown. Also presented are the results of three-dimensional stress-pore pressure coupled finite element analysis on a tunneling situation similar to the case history. Based on the results the effect of groundwater drawdown on the ground settlement, pore water pressure, and lining stresses are discussed. The results indicated that the groundwater drawdown that may occur during tunneling intensifies the ground surface settlement, and some correlation between the level of drawdown and the magnitude of settlement may be established with further research. Also revealed is that the shotcrete lining stresses do not significantly vary with the drawdown level due to the transient nature of seepage forces caused by the groundwater drawdown during tunneling.

Acknowledgements

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References


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Numerical Evaluation on Thermal Performance and Sectional Efficiency of Closed-Loop Vertical Ground Heat Exchanger

수치해석을 통한 수직 밀폐형 지중열교환기 단면의 열전달 효율 분석

Gil, Hujeong*1  길 후 정  Lee, Kangja*1  이 강 자

Lee, Chulho*1  이철호  Choi, Hangseok*1  최 항 석

요  지

본 연구에서는 현재까지 경험적으로 설계・시공되는 지중열교환기의 열효율을 수치해석을 통해 분석하였다. 지중 열교환기에 사용되는 HDPE (High Density Polyethylene) 파이프의 열전달 거리에 따른 열효율 분석을 위해 일련의 수치해석을 실시하였다. 유한요소해석 프로그램(ANSYS)을 이용하여 유입 파이프와 유출 파이프 사이의 이격거리 영향을 검토하였고, 유입 파이프와 유출 파이프 사이에 단열을 위해 격자형 단면을 제안하였으며, 이에 대한 2차원 해석을 실시한 결과, 난방방식 유입 파이프와 유출파이프 사이의 열간섭이 전체 시스템 효율에 영향을 줄 수 있음을 확인하였다. 즉, 유입 파이프와 유출 파이프의 이격 거리가 클수록 유입 파이프가 유출 파이프에 미치는 열간섭 현상이 적어질 수 있었고, 나아가 두 파이프 사이에 단열 격자의 존재가 열간섭이 더욱 감소하였다. 또한 난류 유동, 유체의 흐름, 열전달 해석이 가능한 3차원 유한 체적 해석 프로그램(FLUENT)을 이용하여 유입 파이프와 유출 파이프의 거리에 따른 영향, 천공벽과 HDPE 파이프 사이의 채움재 특성에 따른 영향, 격자형 파이프의 효율 분석, 그리고 순환수의 순환속도 영향 등에 대한 3차원 해석을 실시하였다. 해석결과 유입 파이프와 유출 파이프 간의 상호 열간섭이 지중 열교환기 전체 시스템에 매우 중요한 영향을 미치고, 이로부터 기존의 독립된 2가닥 HDPE 파이프 보다 본 연구에서 제안된 격자형 일체 파이프 단면이 단면 열효율을 증대시킬 수 있음을 알 수 있다.

Abstract

This paper presents a series of numerical simulations on the thermal performance and sectional efficiency of a closed-loop vertical ground heat exchanger (U-loop) in a geothermal heat pump system (GHP). A 2-D finite element analysis adopting ANSYS was employed to evaluate the temperature distribution over the cross section of the U-loop system involving HDPE pipe/grout/soil to compare the sectional efficiency between the conventional U-loop and a new latticed HDPE pipe system, which is equipped with a thermally insulating lattice in order to reduce thermal interference from the inlet pipe to outlet pipe. In addition, a 3-D finite volume analysis (FLUENT) was used in simulating the operation of the closed-loop vertical ground heat exchanger with the consideration of the effect of grout’s thermal properties, the rate of circulation pump, the distance between the inlet and outlet pipes, and the effectiveness of the latticed HDPE pipe system. It was observed that the thermal interference between the two strands of U-loop is of importance in determining the efficiency of the ground heat exchanger. Therefore, it is highly recommended to modify the configuration of the conventional U-loop system by equipping the thermal insulating lattice between the two pipe strands.

Keywords : Ground heat exchanger, HDPE pipe, Latticed pipe, Sectional efficiency

*1 Graduate Student, School of Civil, Environmental and Architectural Engrg., Korea Univ.
*2 Member, Graduate Student, School of Civil, Environmental and Architectural Engrg., Korea Univ.
*3 Member, Associate Prof., School of Civil, Environmental and Architectural Engrg., Korea Univ., bchoi2@korea.ac.kr, Corresponding Author
1. Introduction

Under the present international energy situation, it is necessary to develop new and/or renewable energy sources, which will substitute for fossil energies. Among new and renewable energy sources, the geothermal energy that is produced by the internal heat of the earth is not only inexhaustible but also eco-friendly. The most representative application of geothermal energy is the geothermal heat pump (GHP) system using the closed-loop vertical ground heat exchanger (Philippacopoulos and Berndt, 2001; ASHRAE 2000).

The geothermal heat pump (GHP) system is a heating and/or cooling system that uses the earth’s ability to store heat in the ground or water thermal masses. This system operates utilizing the stability of underground temperatures. The ground has a very stable temperature (between 10 and 16°C) throughout the year depending upon location’s annual climate. The ground temperature is warmer than the air above during winter and cooler than the air in summer. The GHP system uses such available heat in winter (heating) and puts heat back into the ground in summer (cooling). There are four common types of closed-loop system; vertical, horizontal, slinky and pond. Especially, closed-loop vertical ground heat exchanger is typically used when there is a limited area of land available (IGSHPA, 2007).

The closed-loop vertical ground heat exchanger is composed of pipes that run vertically in the ground. A hole with a diameter of around 15 cm is bored in the ground, typically, 50 to 200 m deep. Pipe pairs in the hole are joined with a U-shaped cross connector at the bottom of the hole. The borehole is commonly filled with bentonite grout or cementious grout surrounding the pipe to provide a good thermal connection to the surrounding soil or rock to maximize the heat transfer. Grout also protects the ground water from contamination, and prevents ground water from flooding the property. The high-density polyethylene (HDPE) pipe is typically used for circulating pipe which demands durability, corrosion resistance and high thermal conductivity. A Schematic of installation of the closed-loop vertical ground heat exchanger is illustrated in Figure 1.

In this paper, a series of numerical analyses on the thermal performance and sectional efficiency of a closed-loop vertical ground heat exchanger (U-loop) in the GHP system involving ground soils and grouts was accomplished. A 2-D numerical analysis using ANSYS was employed to evaluate the temperature distribution over the cross section of the U-loop system involving HDPE pipe/grout/soil. Based on the temperature distribution, the sectional efficiency of a conventional two strands U-loop and a new latticed HDPE pipe system, which is equipped with a thermal insulation zone (lattice) in order to reduce thermal interference from the inlet pipe to the outlet pipe, were compared with each other. In addition, a 3-D finite volume analysis program FLUENT was adopted to analyze the coupled model of heat transfer and fluid flow through the HDPE pipe. FLUENT is a Computational Fluid Dynamics (CFD) program for predicting and modeling fluid flow, heat and mass transfer, chemical reactions, and related phenomena by solving numerically the set of governing mathematical equations. It provides complete mesh flexibility, including the ability to solve flow problem using unstructured meshes that can be generated about complex geometries with relative ease (FLUENT, 2008). This program was used to simulate the operation of the closed-loop vertical ground heat exchanger with the consideration of the effect of distance between the inlet and outlet pipes, thermal properties of grout, effectiveness of the latticed HDPE pipe system and rate of circulation pump.
2. Analysis Methods

2.1 2-D Heat Transfer

A two-dimensional cross section for the GHP system was chosen to numerically evaluate the sectional efficiency using ANSYS. Figure 2 (a) shows the cross section of a conventional U-loop, and Figure 2 (b) shows a newly developed latticed pipe system. The ratios of s/D being considered in the current study are presented in Table 1. The symbol s indicates the shortest distance between the inlet and outlet pipes. The symbol D represents the borehole diameter.

An analysis for the latticed pipe section, which is devised to fill thermally insulating materials such as air in the empty lattice between the inlet and outlet pipes, was performed. Heat transfer analysis was carried out under the conditions of the ratio of s/D equal to 0.18 in both the cooling mode and heating mode. Figure 3 shows numerical meshes used in the numerical simulation for the conventional U-loop and the latticed pipe system.

In order to analyze the thermal behavior of two kinds of pipe systems in cooling and heating modes of the GHP system, fixed temperatures of the inlet and outlet pipes were considered as boundary conditions. The steady-state analysis was performed with those boundary conditions. Thermal conductivities (k) of 1.0, 2.5, 0.4 and 0.025 W/m·K

<table>
<thead>
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<th>D (mm)</th>
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<tbody>
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<td>2.7</td>
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<td>Cross section geometry, s/D</td>
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<td>0.18</td>
<td>0.33</td>
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(a) Conventional U-loop

(b) Latticed pipe system

Fig. 2. Cross section (A-A) of ground heat exchanger

(a) Conventional U-loop

(b) Latticed pipe system

Fig. 3. Two dimensional numerical mesh configuration
for the grout, the soil formation, the pipe, and the air, respectively, were used as input values in the numerical analysis. The temperatures of fluid flowing into the inlet pipe are maintained as 25°C in the cooling mode and 2°C in the heating mode. The initial temperature of both grout and soil formation are assumed to be 12°C.

2.2 3-D Heat Transfer Considering Fluid Flow through Pipe

A series of 3-D heat transfer analyses using FLUENT were performed in order to assess the heat transfer efficiency of GHP. First, the effect of the distance between the inlet and outlet pipes has been evaluated. The heat transfer analysis was performed for the three ratios of s/D, that is, 0.03, 0.18, and 0.32, which were exactly same with the previous 2-D heat transfer analysis. This 3-D numerical analysis can simulate a real condition of fluid circulation through the ground heat exchanger. The model geometry was as follows: a parallelepiped soil column with the height of 100 m and with the cross section of 1 m × 1 m, and a borehole with a diameter of 15 cm, in which two strands of HDPE pipe with a diameter of 3 cm are installed. The 3-D model profile and mesh configuration are rendered in Figure 4. The temperature of the fluid, which circulates through the pipe, was assumed to be 25°C (298K), and the temperatures of the soil formation and grout were equally assumed to be 12°C (285K). It was assumed that a cementitious grout was used for backfilling the borehole. The densities of the soil, grout, and fluid were assumed as 1820, 3640, and 998.2 kg/m³, respectively. Thermal capacities of the soil, grout, and fluid were fixed as 440, 840, and 4182 J/kg·K, respectively. Thermal conductivities of the soil, grout,
and fluid were selected as 2.5, 2.02, and 0.6 W/m·K, respectively, from geologic literatures. Table 2 summarizes the thermal properties of soil, grout and fluid, which were inputs for the numerical analysis.

It is commonly assumed that an increase in the thermal conductivity of backfilling grouts may lead to the enhancement of system efficiency during operating the GHP system (Allen, 2000, Choi et al, 2008). There is a pitfall in reasoning this hypothesis with no consideration of sectional configuration of the GHP system. Therefore, two types of grout materials (i.e., bentonite grout and cementitious grout) were considered to evaluate the effect of grout properties on the efficiency of a closed-loop vertical ground heat exchanger. As can be shown in Table 3, the cementitious grout possesses the thermal conductivity much higher than that of the bentonite grout.

A 3-D numerical analysis for the latticed pipe section was conducted to assess the thermal interference between the inlet and outlet pipes. For this analysis, it is necessary to input additional material properties for air as a heat insulator. The density, thermal capacity, thermal conductivity, and viscosity of air were assumed as 1.225 kg/m³, 1006.43 J/kg·K, 0.01 W/m·K, and 0.0000179 kg/m·s, respectively.

In addition, the effect of rate of circulating pump on the thermal performance of the GHP system has been studied. Three pumping rates, i.e., 0.1 m/s, 0.6 m/s, 6 m/s were considered in the numerical analysis. Especially, the rate of 0.6 m/s is practically recommended by Han et al (2005).

### Table 2. Thermal properties of soil, grout and fluid

<table>
<thead>
<tr>
<th>Properties</th>
<th>Soil</th>
<th>Grout</th>
<th>Fluid</th>
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<tr>
<td>Density (kg/m³)</td>
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<td>Thermal capacity (J/kg·K)</td>
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### Table 3. Properties of grout materials

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<th>Cementitious grout</th>
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<td>Thermal conductivity (W/m·K)</td>
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### 3. Results

#### 3.1 2-D Heat Transfer

Temperature distribution in the circumference of the outlet pipe was recorded from the left side of the outlet pipe in a counterclockwise direction in order to evaluate the thermal interference. In the cooling mode, it can be seen in Figure 5 that the thermal interference between the inlet and outlet pipes is reduced considerably by increasing the ratio of s/D. This tendency can be observed in the heating mode shown in Figure 6 as well. It can be inferred that the thermal interference between the inlet and outlet pipes is considerably reduced by increasing the ratio of s/D.

In order to assess the sectional efficiency of the latticed pipe section that is devised to fill a heat insulator such
as air in the lattice between two pipes for insulating space between the inlet and outlet pipes, the latticed pipe was compared with the conventional U-loop in the case that the ratio of s/D equals 0.18. Figure 7 and 8 show the temperature distribution along the circumference of the outlet pipe in cooling and heating modes. In the cooling mode with a fixed inlet pipe temperature of 25°C, the average outlet temperature of the conventional pipe is 17.9°C and the average outlet temperature of the latticed pipe is 288.7 K. In the heating mode with a fixed inlet pipe temperature of 2°C, the average outlet temperatures of the conventional and latticed pipes are 7.5°C and 9.2°C, respectively. The latticed pipe shows much less thermal interference from the inlet to outlet than does the conventional U-loop.

3.2 3-D Heat Transfer Considering Fluid Flow through Pipe

Figure 9 presents the distribution of temperature along the radial direction of the borehole at 10 m in depth from the surface. When the inlet temperature of fluid at the ground surface is fixed at 25°C with the ratio of s/D equal to 0.03, a difference between temperatures of the inlet and outlet fluids is around 2°C. However, in the case of s/D equal to 0.33, the inlet temperature is 24.6°C and the outlet temperature is 18.6°C with a temperature difference equal to around 6°C. The larger the ratio of s/D is, the lower the thermal interference is. For this reason, an increase in the distance s/D between the inlet and outlet pipes seems to enhance the thermal efficiency of the GHP system.
Figure 10 presents the effect of grout’s thermal properties at the depth of 50 m from the ground surface. In the case of bentonite grout, the inlet temperature is 19.9°C and the outlet temperature is 17.0°C. In the case of cement grout that possesses higher thermal conductivity, the inlet temperature is 19.6°C and the outlet temperature is 17.3°C. This result is opposed to the common belief that an increase in the thermal conductivity of backfilling grouts leads to the enhancement of system efficiency of the GHP system. Because the thermal conductivity of cement grout is much higher than the bentonite grout, the cement grout is more vulnerable to thermal interference between the inlet and outlet pipes without improvement of sectional efficiency. Consequently, in enhancing the thermal efficiency of the GHP system, one should consider not only the use of thermally-enhanced grout but also the sectional efficiency of the GHP system.

Figure 11 shows a comparison of the conventional U-loop pipe and latticed pipe. In the case of the inlet pipe, the temperature of both pipe systems is equal to 24°C. However, in the outlet pipe, the temperature of the conventional U-loop pipe is 22.6°C, but the outlet temperature in the latticed pipe is 18.6°C. The latticed pipe has lower thermal interference than does the conventional pipe.

The effect of rate of circulating pump is compared in Figure 12 for the three cases, i.e., 0.1 m/s, 0.6 m/s and 6.0 m/s. Even though the result indicates that a slower pumping rate results in more efficient heat transfer to the soil formation, the optimal pumping rate should be determined on the basis of fluid dynamic factors and maintenance cost.

4. Conclusions

This research was conducted to evaluate numerically the thermal performance of the closed-loop vertical ground heat exchanger in a geothermal heat pump system. The following results were found in this research.

(1) In the case of the conventional HDPE pipe, it is concluded that the shorter the distance between the inlet and outlet pipes is, the higher the thermal interference is. The thermal interference reduces system efficiency of the geothermal heat pump system and leads to wasteful design.

(2) When interpreting a heat transfer phenomenon in relation to grout materials, it is found that cement grout, which has relatively higher thermal conductivity than does bentonite grout, produces higher thermal interference between the inlet and outlet pipes. In order to enhance the thermal efficiency of the GHP system therefore, the engineers should consider not only the use of thermally-enhanced grout but also the sectional efficiency of the GHP system.

(3) In the case of a new developed latticed pipe, it is confirmed that thermal interference can be reduced considerably by equipping an insulation zone between the inlet and outlet pipes, in which a thermal insulator such as air can be filled.
Acknowledgement

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Thermal Stability in Underground Structure with Ground Freezing

지반동결에 따른 지하구조물의 열적 안정성

Shin, Eun-Chul
Kang, Jeong-Gu
Park, Jeong-Jun

요  지

최근 우리나라 산업화 및 기술의 현대화로 국토의 유효면적이 인구에 비해 부족하기 때문에 대규모 공유수면을 매립하여 지하구조물 설치를 위한 연약지반의 동결, LNG와 같은 저온 액체를 저장하기 위한 지하저장탱크 건설 및 주변지반의 동결 등 지반동결 공법에 관심이 집중되고 있다. 본 연구에서는 실트질 홀과 모래질 홀으로 구성된 지반에서 지반동결공법 적용시 지하구조물인 LNG 저장탱크의 안정성 평가하기 위하여 실험적실험을 통한 지속시간과 온도 변화에 따른 동상압력을 예측하였다. 즉, ○○ LNG 인수기지에 위치한 TK-A, B, C, D 지하저장탱크에 대하여 실험적실험, 현장계측결과를 바탕으로 수치해석을 수행하여 동상압력과 동결 영향범위 예측 및 안정성을 평가하였다.

Abstract

The in-ground LNG storage tank, and buildings can be constructed using artificial freezing method on the reclaimed land to control the uplift pressure caused by seepage forces. In this paper, the stability analysis for LNG storage tank constructed on silty and sandy soil is presented. Upon freezing a saturated soil in an open-system from the top, a considerable freezing pressure was developed. The freezing pressure arising within the soil samples and the temperature of the samples inside were monitored with elapsed time. This paper presents the result of the stability analysis for LNG storage tank located in the ○○ LNG Receiving Terminal to evaluate the stability of LNG Tanks TK-A, B, C, D type. The influence range of freezing pressure was considered using freezing laboratory test and numerical analysis. The field measurements and heat analysis of the foundation for LNG tank were also conducted.

Keywords : Artificial freezing method, Freezing laboratory test, Freezing pressure, Freezing zone, Heat transfer

1. Introduction

The first LNG storage tank in Korea was completed at Pyeongtaek terminal in 1986. Since then, LNG consumption has steadily increased and total storage capacity has risen in line with the rising usage. The liquefied natural gas was stored in state of -162 degree. Therefore, LNG storage tank requires high structure stability like expansion under the extremely low temperature.

Various models have been developed to describe frost heave (Kujala, 1997; Konard and Mogenstern, 1982). Presently, the model most frequently used to describe the
mechanism of frost heave is the secondary frost heave theory proposed by Miller (1972), since it can explain the formation of intermittent layers of ice lenses. The model emphasizes the importance of ice growing in the frozen fringe, which was assumed to consist of a network of pore ice extending from the base of the warmest ice lens to a position at or near the 0°C isotherm. However, the configuration of ice in a porous medium depends greatly upon the conditions of freezing and the characteristics of the medium.

Taber (1929), Kinoshita and Ono (1962), Miller et al. (1960) experimentally measured both the amount of heave and the heaving pressure. More significant, however, it was observed that the pressure does not only develop in an ice-water system, but is common to all growing crystals. Their experiment differs from the previous laboratory experiments which showed that a center zone of sample was restrained from heaving, while the perimeter heaved freely. Friction forces along the loaded area added to the heaving pressure and that heaving pressure increases with decreasing pore size and that the pore size characteristics are related to soil parameters such as permeability and capillarity.

For a saturated silty soil, the effects of freezing depend on the rate at which the temperature is lowered. The major concerns to the engineer are the damaging effects of ice lens growth at the freezing front, heave of the pavement surface, followed by thaw weakening of the foundation soils in the spring. The pavement is most susceptible to breakup during thawing of the foundation soils. Differential heave of foundations results in the distortion of structures. To establish a reliable criterion, i.e., one by which it can be stated that a soil is definitely frost susceptible or is definitely not frost susceptible, boundary conditions based on all the significant soil parameters must be established. The most significant of these parameters will have the greatest effect on the classification.

Reclaimed soils can be subjected to large uplift force resulting from freezing of the surrounding soils. Pressures are transmitted to the structure by vertical forces acting on the underside of a structure, by lateral forces acting behind walls, or by transfer of uplift forces through soil frozen to the sides of a structure. Many investigators have been concerned with the pressure developed by soil as it freezes. This paper presented a soil freezing test to estimate an expansion pressure. Silty soil and sandy soil were used in laboratory freezing test subjected to thermal gradients under open-systems. It presents the result of case study on seepage analysis and countermeasure against increasing the seepage volume of in-ground LNG storage tank excavation work. The result of Temp/w analysis to estimate freezing zone from the sidewall and bottom slab of in-ground LNG tank is also presented.

2. Review for Design of LNG Storage Tank

2.1 Types of LNG Storage Tank

Since Pyeongtaek thermal power plant began to use natural gas in 1986, the annual consumption of natural gas rapidly increased and reached 12.7 million tons in 1999. When the natural gas is cooled to a temperature of approximately -162°C at atmospheric pressure, it condenses to a liquid called liquefied natural gas (LNG). LNG has a special character such as being odorless, colorless, non-corrosive, and non-toxic. So, LNG storage tank, tanker ship, transfer pipelines require the special storage and transportation systems and technology. The presently operating LNG terminals are Pyongtaek and Incheon Terminals. A total of 19 above-ground LNG storage tanks (100 thousand kℓ grade) are currently in operation with a send-out capacity of 4,360 tons/hour. To meet the growing domestic demand of LNG supply, the Incheon Receiving Terminal is being expanded (ten in-ground tanks) and a third LNG terminal is being constructed at Tongyeong.

In general, above-ground LNG storage type was constructed at a cost 1.3~1.5 times as low as that of in-ground type. However, in-ground type has an advantage in some cases. It is relatively safe due to no leakage to the ground. It does not need chemical material for fire extinguishing and it has high earthquake resistance. And it has not a blot on the landscape compared with above-ground tank type. Table 1 and Table 2 show the
characteristic of above-ground type and in-ground type of LNG tank. It describes capacity, construction method, advantages and disadvantages of each type, and stability. The technical construction method was developed, based on deep slurry walls considering high ground water level. This made possible the construction of in-ground type LNG tanks with the capacity of 200,000 kℓ that came into operation at Incheon LNG Terminal.

2.2 Design of In-ground LNG Storage Tank

LNG storage tank, TK-A and TK-B were constructed by in-ground type with storage capacity of 200,000 kℓ. Based on detail investigation for construction design, it was designed with an inside diameter of 72 m, and excavation depth of EL -49.2 m. The foundation type of tank was designed with mat foundation supported by weathered rock. Fig. 1 shows the tank configuration in detail design.

It was designed after construction of bottom slab of which thickness is about 9 m. The gravel layer of 1.2
2.3 Alternation Method of Foundation

Originally, TK-A and TK-B were designed with mat foundation, but it was changed into pile foundation as an alternation method because an operation method of LNG tank was changed as a rapid high water level type using freezing method. Rapid high water level type can reduce the duration of water pumping after storage of LNG and reduce a weight of LNG using uplift pressure of ground water in early period.

The steel casing method was realized by installation of steel casing pile with diameter of 711.2 mm which was intruded into soft rock and then of which top was covered by cap. Table 3 describes a characteristic of casing pile which was installed into ground. Fig. 2 shows a plan view of casing piles layout on the base.

Table 3. Characteristic of casing pile installed

<table>
<thead>
<tr>
<th>Number of Pile</th>
<th>1072 EA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td>Structural Steel, SKK 400</td>
</tr>
<tr>
<td>Allowable stress</td>
<td>1400 kgf/m²</td>
</tr>
<tr>
<td>Diameter/Thickness</td>
<td>711.2 mm/14 mm</td>
</tr>
<tr>
<td>Length of Pile</td>
<td>12,600 mm</td>
</tr>
</tbody>
</table>

3. Laboratory Test for Freezing Pressure

3.1 Physical Properties of Soil Used

The general geotechnical properties of silty soil and sandy soil from ○ ○ LNG Receiving Terminal are classified as ML and SP, respectively, by the Unified Soil Classification System. The percent passings of U.S. sieve No. 200 for these soils are 75.2%, and 7.8%. The physical properties of these soils are tabulated in Table 4. The resulting grain size distribution curves are shown in Fig. 3.

The reclaimed soils reported here include two samples of the silty soil and sandy soil which obtained from the soil strata of the ○ ○ LNG Receiving Terminal construction site. Based on the grain size distribution, the percent of fine particle (<0.02 mm) was approximately 32.5%. Therefore, the silty soil was frost susceptible soil because the amount greater than 0.02 mm is more than 15% from U.S. Army Corps of Engineers frost-susceptibility criteria.

Table 4. Physical properties of soils

<table>
<thead>
<tr>
<th>Physical properties</th>
<th>Silty soil</th>
<th>Sandy soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>( w_{li} (%) )</td>
<td>26.4</td>
<td>17.18</td>
</tr>
<tr>
<td>( C_s )</td>
<td>2.67</td>
<td>2.65</td>
</tr>
<tr>
<td>( LL % / P.I. )</td>
<td>24.3/NP</td>
<td>- / NP</td>
</tr>
<tr>
<td>( \gamma_{dry} )  (kN/m³)</td>
<td>16.8</td>
<td>19.1</td>
</tr>
<tr>
<td>( w_{sat} )  (%)</td>
<td>16.3</td>
<td>11.1</td>
</tr>
<tr>
<td>( k ) (cm/sec)</td>
<td>( 1.7 \times 10^{-5} )</td>
<td>( 2.1 \times 10^{-4} )</td>
</tr>
<tr>
<td>Passing No. 200 (%)</td>
<td>75.2</td>
<td>7.8</td>
</tr>
<tr>
<td>USCS</td>
<td>ML</td>
<td>SP</td>
</tr>
</tbody>
</table>

Fig. 2. Plan view of LNG tank foundation installed with pile

Fig. 3. Grain size distribution of soil used for the present study
3.2 Experimental Apparatus

Improvements on the freezing test equipment incorporate the recommendations of ASTM (1996). This equipment includes a rubber-membrane-lined, multi-ring freezing cell to minimize the side friction effect, liquid-cooled cold plates for precise top and bottom temperature control. The data acquisition and control system are furnished for automated temperature control and data processing. The test uses two freezing-thawing cycles to account for changes in susceptibility to frost heave caused by prior freezing-thawing cycle. The warm and cold side temperatures were controlled by two independent brine circulation systems which have an accuracy of ±0.1°C. Three identical samples were tested at the same test condition, each 100 mm in diameter and 250 mm in height. Samples were compacted to the equivalent field density and moisture conditions.

The freezing cell is an apparatus that was modified for the purpose of examining the frost heave process in a freezing soil system. The freezing cell permits water intake from the bottom through the base plate.

Insulation was placed along the sides of the freezing cell in an attempt to create a condition of unidirectional heat flow. Unfortunately, the insulation was not sufficient to completely eliminate the effect of outside temperature fluctuations; however, it did reduce the effects to a tolerable level. Thermocouples were installed in the samples at the depth of 50, 100, 150, 200 mm. A schematic diagram of frost heaving pressure apparatus and the test setup is presented in Fig. 4.

3.3 Laboratory Test Results

In general, the freezing and heaving forces increase as the frost line penetrates the unfrozen soil. It is difficult to predict the magnitude of these mobilized heave expansion pressures because many variables are involved. These variables include soil type and heterogeneity, variation of soil temperature with time, rate of freezing, availability of water, foundation surface type, overburden pressure, and foundation loads (Hoekstra, et al., 1965). Therefore, the reclaimed soil expands due to freezing of pore water and formation of ice lenses from water sucked from the unfrozen soil.

The frost action is used to describe the detrimental process of freezing resulting from the formation of ice lenses at the freezing plane in soil during the freezing period followed by thaw weakening of decrease in bearing strength when seasonally frozen soil thaws. The general unsteady heat flow near the ground surface coupled with conditions of crystal ice nucleation and growth is the necessary condition for formation of alternating bands of soil and ice. In addition, it is essential that the rate of heat extraction exceeds the rate of heat supply to the freezing front. If the pressure that develops upon freezing a soil is the pressure required of ice to transfer through the pores, then this pressure should originate at the freezing front.

The freezing pressures were measured with elapsed time for a given soil specimen. As can be seen from Fig. 5, each soil has a unique characteristic value. The rate of freezing pressure development curves shows its specific characteristic pressures for each given soil types. Fig. 5 shows the change of freezing pressure determined by the load cell with the passage of time and phase down temperature. The maximum freezing pressure varies from 340 kN/m² to 427 kN/m² for silty soil. However, the maximum freezing pressure of sandy soil was approximately 40 kN/m². In the design of ○○ LNG tank, the maximum freezing pressure applied to the LNG tank was 100 kN/m².
The type of soil deposited around ○○ LNG tank as shown in Fig. 1 is sand. Therefore, by comparing the freezing pressure shown in Fig. 5 (b) and design freezing pressure for ○○ LNG tank, it is turned out that the stability of LNG tank in terms of freezing pressure is pretty stable.

Water contained in the voids of a moist or saturated sand of gravel freezes in situ when the temperature drops below the freezing point. If the temperature drops gradually, a large part of the frozen water accumulates in the form of layers of ice oriented parallel to the surface exposed to the freezing temperature. As a consequence, the frozen silt of silty sand consists of a series of layers of frozen soil separated from each other by layers of ice.

In sandy soils, because of the size of the particles, gravity forces are most likely to predominate both in the mineral and liquid phases. Hence, the surface forces are, by comparison with the gravity forces, so small that their effect may be neglected. In consequence, water in the soil voids will freeze as the ice front propagates into the void spaces, resulting therefore in little movement of water.

Silty soil was volumetric expansion of pore water and ice segregation constitutes two separate mechanisms in the soil-freezing phenomenon. However, in fine-grained soils, because one mechanism dominates the production of the overall freezing mechanism, it is possible to predict freezing pressure in laboratory test. Several situations can be considered for a silty soil with a zone of frozen soil, an active ice lens, a freezing fringe, and an underlying zone of unfrozen soil. The fringe is a region of impeded flow caused by partial filling of soil pores by ice. The soil skeleton, within the fringe, will expand when pressure in the ice exceeds the overburden pressure plus any pressure required to initiate separation of the soil skeleton. With sufficient ice pressure, the soil skeleton separates and a new ice lens forms (Shin and Park, 2003).

Consequently, freezing pressure was relieved by heaving of the soil in the direction of least resistance. It was shown that the freezing expansion pressures developed upon freezing a soil apparently dominate for an initial setting temperature and with elapsed time.

4. Stability Analysis for Ground Settlement

4.1 Installing of Settlement Gauge

In case of TK-A, vertical settlement gauges (A, B, and C) were installed to measure a differential settlement at the center and edge of the tank. Displacement measurement gauges (A1, A2~D1, and D2) were also furnished to check the vertical and horizontal displacement of the bottom and side wall. The position of each case as vertical settlement gauge and displacement relative measurement gauge can be seen in Fig. 6.

4.2 Analysis for Measurement of Settlement

In case of TK-A, the result of maximum settlement is a bit high, however it is lower than that of allowable settlement on the bottom of tank. TK-B and TK-C also
show similar displacement and same direction as TK-A. The displacements intend to increase gradually so that it needs to maintain through long time. On the other hand, the settlement level of case TK-D is relatively high and gradually increases as shown in Fig. 7. Table 5 shows a measurement result for vertical settlement and horizontal
displacement at each tank since the tank has been operated. The settlement levels are somewhat related to the operation of LNG storage tank and it shows a gradual increment with the elapsed time. Hence, the safety of LNG tank should be carefully monitored.

4.3 Numerical Analysis for Freezing Zone Around LNG Tank

The analysis for temperature based on construction and ground condition in TK-A, TK-B, TK-C, TK-D was realized to calculate freezing zone at side wall and bottom slab of tank by TEMP/W (Ver. 5.14). The TEMP/W two-dimensional finite element computer program has been used extensively by many practicing ground freezing consultants for design of artificial ground freezing projects. In the past it was common to apply the thermal boundary condition for a freeze pipe in LNG tank by assuming a fixed temperature decay function for early stages of freezing followed by an assumption that the pipe surface is as cold, or nearly cold as the brine for the remainder of the freezing period (GEO-SLOPE, 1996).

Recently, TEMP/W has been modified so that the user is able to apply a convective heat transfer analysis for the pipe-ground interface in LNG tank. The amount of convective heat transferred between the ground and the chilled brine is dependent on the ground temperature relative to the brine temperature and it subsequently determines what the new ground temperature will be. Based on this approach, the ground will cool at variable rate and to a minimum value that is determined by the brine flow parameters and the difference between brine and ground temperature.

Technically, the convective heat transfer is occurring between the internal pipe wall and the fluid but it is acceptable to combine the conductive heat transfer across the steel pipe wall with the convective component to arrive at a combined convective heat transfer coefficient. Regardless of the nature of the convective heat transfer process, the appropriate rate equation is:

\[ q = h(T_s - T_f) \]  

where \( q \) is the unit heat flux \((W/m^2)\), \( h \) is the combined convective heat transfer coefficient \((W/m^2\cdot°C)\), \( T_s \) is the pipe external surface temperature \(°C\), and \( T_f \) is the fluid temperature.

It is critical that an appropriate reference manual on heat transfer be consulted prior to determining the convective heat transfer coefficient for each specific situation. In general, the heat transfer coefficient between the pipe’s internal surface and the fluid is calculated from:

\[ h_i = \frac{N_u K}{D_h} \]

Where \( N_u \) is the Nusselt number (a function of fluid temperature, viscosity, velocity, pipe friction), \( K \) is the thermal conductivity of the fluid at the fluid temperature \((W/m^2\cdot°C)\), and \( D_h \) is the effective hydraulic diameter of the pipe \((m)\).

The objective of this paper is to outline the correct modeling technique for determining the heat transfer between the ground and a buried freeze pipe in LNG tank that contains a flowing coolant at a known temperature.

The concrete of LNG tank (wall or bottom slab) can be pressured by freezing zone and material of soil during operation because the temperature of LNG is extremely low temperature like -162. Table 6 indicates a thermal constants of concrete related with temperature rising.

<table>
<thead>
<tr>
<th>Table 6. Thermal constants for concrete by temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heat conductivity ((kcal/mh°C))</td>
</tr>
<tr>
<td>-----------------------------------------------</td>
</tr>
<tr>
<td>2.00</td>
</tr>
</tbody>
</table>
number is constant at 4.10. This would apply to most ground freezing scenarios. The hydraulic diameter is one half of the difference between the internal diameter of the external freeze pipe and the external diameter of the polyethylene supply pipes. In other words, it is the thickness of the annular gap in the freeze pipe.

The convective heat transfer coefficient can be combined with the pipe wall thermal conductivity to arrive at an overall heat transfer coefficient that accounts for conduction in the pipe. A heat transfer reference manual will give

![Fig. 8. Result of numerical analysis for the determination of freezing zone around LNG tank](image)

(a) TK-A  (b) TK-B  (c) TK-C  (d) TK-D

Fig. 8. Result of numerical analysis for the determination of freezing zone around LNG tank

![Fig. 9. The range of temperature around LNG tank](image)

(a) At side wall  (b) At bottom slab

Fig. 9. The range of temperature around LNG tank
various methods of arriving at an overall heat transfer coefficient for different pipe geometries. Once this is determined, it can be substituted directly for the \( h \) value in Eq. (1) above.

Fig. 8 and Fig. 9 indicate the area of freezing by numerical analysis. Using this figure, it is possible to calculate the freezing zone at the side wall and bottom slab by temperature. Based on the results of numerical analysis, the distance of freezing zone was extended about \( 2 \text{ m} \sim 2.2 \text{ m} \) outward from external face of side wall and the lowest temperature is \(-5^\circ\text{C}\). The range of freezing temperature is about \( 0.85 \sim 1.0 \text{ m} \) downward from the bottom slab for in-ground LNG storage tank and the lowest temperature right below the bottom slab is about \(-14.5^\circ\text{C}\). The range of freezing temperature around in-ground LNG tank shown in Figs. 8 and 9 is comparable to the results shown in Fig. 5.

5. Conclusions

This paper presented the result of the thermal stability for underground structures located in ○○ LNG Terminal to evaluate the safety of LNG tanks for TK-A, B, C, D type. The influence range of freezing pressure was considered using freezing laboratory test, measurement of settlement on the bottom slab below LNG tank, and calculating freezing zone at side wall and bottom slab of tank by TEMP/W.

The maximum freezing pressure varies from 340 kN/m\(^2\) to 427 kN/m\(^2\) for silty soil. However, the maximum freezing pressure of sandy soil was approximately 40 kN/m\(^2\), which is much lower than design freezing pressure of 100 kN/m\(^2\) for in-ground LNG tank in ○○ LNG Terminal. Therefore, it is turned out that the stability of LNG tank in terms of freezing pressure is pretty stable.

Based on the numerical analysis, the distance of freezing zone was extended about \( 2 \text{ m} \sim 2.2 \text{ m} \) outward from external face of side wall and also about \( 0.85 \sim 1.0 \text{ m} \) downward from the bottom slab for in-ground LNG storage tank, and the temperature related to the freezing pressure was not so serious.

The settlement levels are somewhat related to the operation of LNG storage tank and it shows a gradual increment with the elapsed time. Hence, the safety of LNG tank should be carefully monitored.

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Evaluation of Liquefaction Potential with Simplified Method and Effective-Stress Site Response Analysis

간편예측법과 유효응력 지반응답해석을 통한 액상화 예측

Park, Duhee1박두희
Kwak, Dong-Yeop2곽동엽

요  지

액상화 발생 여부 평가는 지진동에 대한 구조물의 성능목표를 달성하기 위한 내진설계의 주요 요소이다. 액상화 평가는 지진하중을 나타내는 등가전단응력과 지반의 저항응력의 관계를 이용하는 간편예측법이 널리 사용된다. 하지만, 간편예측법의 정확성과 국내 적용가능성에 대해서는 정확하게 규명된 바 없다. 본 연구에서는 간편예측법의 적용성을 평가하기 위하여 포항과 광양 두 개의 부지에서 간편예측법과 유효응력 비선형 지반응답해석을 재현주기 1000년과 2400년 지진에 대하여 수행하여 결과를 비교하였다. 비교결과, 두 방법에는 상당한 차이가 있는 것으로 나타났다. 간편예측법은 유효응력 지반응답해석에 비하여 액상화 발생 가능성을 높게 예측하는 것으로 나타났으며 두 예측기법간의 차이는 특히 재현주기 1000년 지진의 경우에 큰 것으로 나타났다. 또한, 예측된 지층 내 과잉간극수압의 분포형상에도 상당한 차이가 있는 것으로 계산되었다. 간편예측법은 액상화 발생가능성의 초기 예측에는 사용될 수 있으나, 보다 정확한 판단을 위해서는 고급해석이 필요하다고 판단된다.

Abstract

Evaluation of liquefaction susceptibility is an important part of the seismic design of structures, which is. The simplified method, which relates the equivalent shear stress due to the earthquake and resisting shear stress, is widely used in practice. However, the accuracy and applicability of the simplified method to domestic sites are not well known or documented. This paper evaluated the applicability of the simplified procedure by comparing the results of the simplified method and the effective-stress nonlinear site response analysis at Pohang and Kwangyang using ground motions representative of earthquake with return periods of 1000 and 2400 years. Comparison showed that the predictions of the liquefaction susceptibility are significantly different. The simplified method constantly predicted higher susceptibility to liquefaction compared to effective-stress site response analysis results. The discrepancy between the estimated susceptibility was especially large for motions representative of seismic hazard with return period of 1000 years. In addition, the estimated variations of the pore pressures within the soil column were distinctively different. The comparisons demonstrated that the simplified method can be used as a preliminary evaluation procedure, but a more advanced model should be used for an accurate evaluation of the liquefaction susceptibility.

Keywords: Effective-stress site response analysis, Excess pore water pressure, Ground motion, Liquefaction, Simplified method

1 Member, Assistant Prof., Dept. of Civil Engrg., Hanyang Univ., dpark@hanyang.ac.kr, Corresponding Author
2 Member, Graduate Student, Dept. of Civil Engrg., Hanyang Univ.
1. Introduction

Various structures such as bridges and port facilities are often built over very loose sandy soil, making them vulnerable to major damage under liquefaction during an earthquake. It is important to accurately estimate liquefaction susceptibility and to provide adequate remediation plans if necessary.

It is a common practice to evaluate the liquefaction potential with the simplified method proposed by Seed and Idriss (1971). The procedure relates the equivalent cyclic shear stress caused by the earthquake loading to the resistance determined by the standard penetration test (SPT) in evaluation of the liquefaction susceptibility. The equivalent shear stress is highly dependent on the frequency content and duration of the ground motion. However, such characteristics are not modeled in the simplified method. For a more accurate prediction of the liquefaction potential, an effective-stress site response analysis is performed. This study evaluated liquefaction susceptibility at Pohang and Kwangyang, which are located in the southern coast of Korea, using both the simplified method and the effective-stress analysis. The results of the analyses are compared and discussed.

2. Simplified Liquefaction Potential Evaluation Procedure

Various methods have been developed to evaluate the liquefaction potential. The most widely used method in engineering practice is the simplified procedure proposed by Seed and Idriss (1971). The method, often termed as the “cyclic stress method,” compares the earthquake induced loading, represented in terms of cyclic stress ratio (CSR), to the liquefaction resistance, represented in terms of the cyclic resisting ratio (CRR). CSR is calculated by the following equation:

\[
\text{CSR} = \frac{\tau_d}{\sigma_v'} = 0.65 \left( \frac{a_{\text{depth}}}{g} \right) \left( \frac{\sigma_v}{\sigma_v'} \right)
\]

where \( \tau_d \) = shear stress, \( a_{\text{depth}} \) = maximum acceleration at the depth at which liquefaction susceptibility is evaluated, \( \sigma_v \) = total vertical stress, \( \sigma_v' \) = effective vertical stress. \( a_{\text{depth}} \) in the equation is either determined by a total-stress site response analysis or by calculating peak acceleration at the surface and then multiplying it to a reduction factor (e.g. depth dependent reduction factor developed by Seed and Idriss (1971)).

CRR represents the minimum CSR required to cause liquefaction of a given soil parameter. The most widely used parameter is the SPT resistance, \( (N_{1})_{60} \), but the cone tip resistance from cone penetration test and shear wave velocity have also been used. An example plot of CRR in terms of \( (N_{1})_{60} \) is shown in Fig. 1, which was proposed by Seed et al. (1985). If the CSR is above the CRR line, then the soil is susceptible to liquefaction. If it is below the CRR line, then the liquefaction is not likely to occur. CRR is also a function of the fine content, since the fine content controls the permeability of soils and hence, the amount of excess pore pressure build-up. The curves in Fig. 1 represent the liquefaction susceptibility under a magnitude 7.5 earthquake. Therefore, it is often written as CRR\(_{7.5}\).

Once CSR and CRR\(_{7.5}\) are determined, the factor of safety (FS) against liquefaction can be calculated by the following equation:

\[
FS = \left( \frac{\text{CRR}_{7.5}}{\text{CSR}} \right) \text{MSF}
\]
where MSF is magnitude scaling factor. The FS is dependent on the magnitude of the earthquake because the duration, energy, and frequency content of the ground motion are determined by the magnitude. Since a typical CRR plot is developed for a magnitude 7.5 event (e.g. Fig. 1), MSF is 1 for a magnitude = 7.5 earthquake. If the magnitude is less than 7.5, then MSF becomes larger than 1.

Various values of MSF have been proposed based on empirical data. Discussion on the differences between the values of proposed MSF is out of the scope of this paper. Korean design code (MOCT, 2005) proposes use of MSF proposed by Idriss (1995), which is defined as follows:

\[
MSF = 10^{2.24} / M_{w}^{2.56}
\]  

(3)

Since the FS is highly dependent on the magnitude of the earthquake, it is crucial that the magnitude be reliably estimated. The representative magnitude should be selected based on probabilistic seismic hazard analysis and deaggregation. The seismic hazard analysis gives an estimate of the aggregate risk from all possible earthquake scenarios in terms of a ground motion parameter, but it does not give any information on the likelihood and contribution of each earthquake event. Deaggregation is an inverse process of probabilistic seismic hazard analysis and shows the contribution of each earthquake scenario to the seismic hazard. The most representative earthquake scenario is, therefore, an event that has the highest contribution to the calculated seismic hazard. In Korea, M = 6.5 is used in the design code (MOCT, 1997) without a clear reasoning. More research is warranted for a more reliable estimate of the representative magnitude.

3. Site Description

The sites selected in this study are Pohang and Kwangyang, which are both located at southern part of Korea. The stratigraphy and the \((N_1)_{60}\) profiles of selected sites are shown in Fig. 2. Pohang site is approximately 27 m in thickness and is composed of layers of coarse sand, silty sand, sandy gravel, and silty clay. Kwangyang site is composed of filled sand layer underlain by very soft clay. The soil column is approximately 29 m in thickness. The shear wave velocity \((V_s)\) profiles were not available at the site. Therefore, the following empirical equation proposed by Imai and Tonouchi (1982) was used to estimate the \(V_s\):

\[
V_s = 97.0(N_1)_{60}^{0.314}
\]  

(4)

The shear wave velocity of the bedrock was assumed to be 760 m/s.

4. Liquefaction Potential Evaluation

4.1 Liquefaction Potential Evaluation Via Simplified Method

The liquefaction potentials of selected sites were evaluated using the simplified approach proposed by Seed and Idriss (1971) and using the curves shown in Fig. 1. Total-stress nonlinear site response analyses were performed using one-dimensional code DEEPSOIL (Hashash and Park, 2001) to obtain peak acceleration profile within the soil column. The constitutive model incorporated in DEEPSOIL is the modified hyperbolic model developed by Matasovic (1993), which defines the backbone curve as follows:

\[
\gamma = \frac{G_m \gamma}{1 + \beta (\frac{G_m}{\gamma})^\gamma} = \frac{G_m \gamma}{1 + \beta (\frac{\gamma}{\gamma'})^\gamma}  \]  

(5)
where $\tau = $ shear stress, $\gamma = $ shear strain; $G_{ma} = $ initial shear modulus; $\tau_{mo} = $ shear strength, $\gamma_r = $ reference shear strain, $\beta$ & $s$ = material constants that adjust the shape of the backbone curve. The parameters selected for the constitutive model are listed in Table 1. Sandy gravel and sand parameters were selected to match upper and mean sand curves by Seed and Idriss (1970), and clay parameters were selected to match PI = 15 curves by Vucetic and Dobry (1991). The selected dynamic curves are shown in Fig. 3. Two input ground motions (Fig. 4) widely used in Korea were used, which were the recorded motion during Ofunato Earthquake and the synthetic motion generated using SIMQKE (Gasparini and Vanmarcke, 1976).

In applying the simplified method, the depth reduction factor was not used since the peak acceleration profile calculated by the site response analysis was applied directly. The MSF was applied by using a design magnitude of 6.5 and applying the Idriss (1995) equation. The FS was not calculated at the silty clay and clay layers.

The calculated FS profiles are shown in Fig. 5. At both sites, the differences between results using the Ofunato and synthetic motions were negligible, while the return period had a higher influence on the FS. At Pohang site, the FS was greater than 1 at layers above the depth of 15 m. However, at the coarse sand layer (19 - 27 m), the calculated FS was below 1 for both 1000 and 2400 years return period motions, and was therefore evaluated as being susceptible to liquefaction. The Kwangyang site is composed of a thick deposit of very loose fill sand. Therefore, it was evaluated as being highly susceptible to liquefaction. The return period did not have a noticeable difference in the calculated FS, since FS was well below 1 even for the return period = 1000 years earthquake.

4.2 Liquefaction Potential Evaluation Via Effective Stress Site Response Analysis

To validate whether the simplified method provides an
acceptable degree of accuracy, the liquefaction potential was also evaluated with the effective-stress site response analysis. DEEPSOIL (Hashash and Park, 2001) was again used in performing the effective-stress analysis. Two excess pore pressure generation models for sands and clays, respectively, are incorporated in DEEPSOIL. The equations for calculating the pore pressure in sand and clay models, respectively, are given in Eqs. (6) and (7):

\[
\begin{align*}
\hat{u}^* &= \frac{u}{\sigma^*_v} = p \cdot f \cdot F \cdot N \cdot \left( \gamma - \gamma_{op} \right)^r \quad - \text{Sand model (6)} \\
\hat{u}^* &= AN^{2(v-\gamma_{op})} + BN^{2(u-\gamma_{op})} + CN^{2(u-\gamma_{op})} + D \quad - \text{Clay model (7)}
\end{align*}
\]

where \( \hat{u}^* = \) normalized pore water pressure ratio (NPWP) after cycle \( N \), \( \gamma_{op} = \) threshold strain under which excess pore pressure does not develop, \( f \) & \( p \) & \( F \) & \( s \) and \( A \) & \( B \) & \( C \) & \( D \) & \( r \) = curve fitting parameters for sands and clays, respectively. Note that both models are functions of shear strain amplitude and number of cycles. The sand model, Eq. (6), was developed by Dobry (1985), while the clay model, Eq. (7), was developed by Matasovic (1993). In both models, the excess pore pressure is dependent on the number of cycles and shear strain amplitude.

Build-up of excess pore pressure causes a decrease in the effective stress and stiffness of the soil. The excess pore pressure model has to be linked with the constitutive model for a hydro-mechanical coupled analysis. In DEEPSOIL, the following shear stress-strain-fluid model developed by Matasovic (1993) is incorporated:

\[
\tau^* = \frac{\sqrt{1-u^* \times G_{so}^*}}{1 + \beta \left( \sqrt{1-u^* \times G_{so}^*} \times \tau_{so}^* \right)}
\]

where \( \nu = \) material constant. In the model, both shear strength and shear stiffness decrease with increase in pore
water pressure.

Since site-specific laboratory tests were not performed, the parameters for the pore pressure model had to be assumed. Table 2 lists the pore pressure model parameters selected in this study. The parameters assigned for sandy gravel layers are representative of the loose fill at Treasure Island, while the properties for sand layers are obtained from Owi site (Matasovic, 1993). Liquefaction occurred at both Treasure Island and Owi. Therefore, the parameters for both sandy gravel and sand layers are representative of very loose condition at which high excess pore pressure can develop. The parameters for the clay layers are representative of soft normally consolidated clays (Matasovic, 1993).

Fig. 6 shows the calculated normalized excess pore pressure using the sand and clay models when applying five cycles of shear strain. Note that sand results in highest pore pressure generation, followed by the sandy gravel. The clay shows very limited pore pressure development.

Fig. 7 shows the calculated NPWP profiles calculated by the effective-stress site response analyses using the Ofunato motion. If the NPWP is close to 1, it means that the liquefaction has occurred. At Pohang, the maximum calculated NPWPs for return period = 1000 and 2400 years earthquakes were 0.2 and 0.6 respectively, and well below 1. The liquefaction is expected not to occur, while it was predicted to occur using the simplified method. The variation of the NPWP and FS within the soil column

![Fig. 6. Calculated normalized excess pore pressures of sand, sandy gravel, and clay pore pressure models](image)

![Fig. 7. Calculated excess pore water pressure via effective-stress site response analyses using Ofunato motion](image)

**Table 2. Parameters of the excess pore water pressure generation models**

<table>
<thead>
<tr>
<th></th>
<th>Parameters for sand model</th>
<th>Parameters for clay model</th>
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<tbody>
<tr>
<td></td>
<td>$f$</td>
<td>$\rho$</td>
</tr>
<tr>
<td>Sandy gravel</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Sand</td>
<td>1.0</td>
<td>1.1</td>
</tr>
</tbody>
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is also remarkably different. While the largest pore pressure is calculated at a depth of 15 m near the boundary between silty clay and silty sand, the lowest FS is calculated at depths higher than 20 m, below the silty clay layer. This showed that peak acceleration and SPT resistance are not sufficient in evaluating the liquefaction susceptibility, since the excess pore pressure is highly dependent on the amplitude of the shear strain. In addition, use of total stress site response analysis in the simplified method has clear limitations in simulating the dynamic soil-fluid interaction.

At Kwangyang, the calculated NPWP is much higher than Pohang. For return period = 1000 years motion, the NPWP was higher than 0.8, and for return period = 2400 years motion, the NPWP was 1. In the simplified approach, liquefaction occurred for both return period motions. The characteristics of calculated NPWP and the FS were, again, significantly different. While high pore pressure was concentrated at depths between 6 - 7 m in the effective-stress analysis, the calculated FS was quite uniform between 0 - 13 m. This showed that an advanced study is needed to determine not only the liquefaction susceptibility but also the distribution of the excess pore pressure within the soil column.

Fig. 8 shows the NPWP profiles using the synthetic motion. At Pohang, the calculated NPWPs using the synthetic and the Ofunato motions were distinctively different. Fig. 9 compares the evolution of the NPWP with time calculated at a depth of 15 m at Pohang site. The synthetic motion caused significantly higher pore pressure than the Ofunato motion, due to the longer duration and richer frequency contents of the ground motion. This comparison demonstrates that the calculated pore pressure is highly dependent on the duration and frequency cha-
racteristics of the ground motion, in addition to the peak ground acceleration. It means that the ground motion time history should be selected more carefully when performing the effective-stress site response analysis. In the simplified method, the differences in the calculated FS between the ground motions were negligible, which means that the simplified method is less sensitive to the characteristics of the ground motion.

6. Conclusion

This paper evaluated the liquefaction susceptibilities of two sites in southern Korea. The methods used in the evaluation were the simplified method proposed by Seed and Idriss (1971) and the effective-stress site response analysis. The simplified method used the factor of safety in evaluation of the liquefaction susceptibility, while the effective-stress analysis used the normalized excess pore pressure ratio. In the simplified method, the cyclic stress ratio within the soil column was determined using the peak acceleration profile calculated by the total-stress nonlinear analysis. The factor of safety against liquefaction was calculated by relating the cyclic stress ratio and the cyclic resisting ratio chart developed by Seed et al. (1985), along with the magnitude scaling factor proposed by Idriss (1995). In performing the effective-stress analysis, two separate excess pore pressure generation models for sand and clay, respectively, were used.

Comparison of two methods of evaluation showed that the predictions of the liquefaction susceptibility were quite different. At Pohang, the simplified method predicted that liquefaction would occur for both return period motions, while the effective-stress analysis indicated that the liquefaction would not occur at the site. The calculated normalized pore pressures were well below 0.5, demonstrating very low liquefaction potential. In addition, the estimated variations of the pore pressure within the soil column showed clear discrepancy between the simplified method and the effective-stress analysis. Similar characteristics were observed at Kwangyang.

The results of the effective-stress analyses were very sensitive to the characteristics of the ground motion, while the simplified method showed lower sensitivity. When using the effective-stress analysis, the representative motion should be selected with care.

The comparisons demonstrated that the simplified method is very conservative, and a more advanced model can provide a more accurate estimate of the liquefaction susceptibility.

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