BEHAVIOR OF VACUUM CONSOLIDATION WITH AND WITHOUT SURCHARGE LOAD

September 2015

Department of Science and Advance Technology
Graduate School of Science and Engineering
Saga University

Steeva Gaily Rondonuwu
BEHAVIOR OF VACUUM CONSOLIDATION
WITH AND WITHOUT SURCHARGE LOAD

By

Steeva Gaily Rondonuwu

A thesis submitted in partial fulfilment of the requirements
for the degree of Doctor of Engineering in
Geotechnical and Geoenvironmental Engineering

Dissertation Advisor : Prof. Chai Jinchun
Examination Committee : Prof. Chai Jinchun
Prof. Ishibashi Koji
Prof. Hino Takenori
DR. Suetsuge Daisuke
External Examiner : Prof. Bergado Dennes T.

Nationality : Indonesian
Previous Degree : Master of Agriculture in Dept.
of Agriculture Saga University,
Japan

Bachelor in Civil Engineering
Sam Ratulangi University (UNSRAT)
Manado, Indonesia

Scholarship Donor : Department of Higher Education Ministry
of Research, Technology and Higher
Education (DIKTI-INDONESIA)

Department of Science and Advance Technology
Graduate School of Science and Engineering
Saga University
Saga, Japan
2015
ACKNOWLEDGEMENT

I give thanks to Jesus Christ my Lord, for He is good, for His steadfast love endures forever. Only by His grace this study has been accomplished.

I want to express my deepest thanks to my advisor, Prof. Jinchun Chai for his sincere guidance, constructive suggestions, encouragements and constant support throughout the duration of this study. He provided not only all the requirements for the accomplishment of my study but also gave the valuable advices in the personal concerns. Without his help in both academic and personal concerns, this dissertation work could not be completed.

I am also very grateful to Prof. Ishibashi Koji, Prof. Hino Takenori, DR. Daisuke Suetsuge for giving advices and suggestions throughout the entire course of study and serving as members of the examination committee. Sincere thanks and appreciation are due to Prof. Bergado Dennes T. at Asian Institute of Technology (AIT), Bangkok, Thailand for his help and suggestions as well as serving as external examiner. Thanks are also extended the laboratory staff, Mr. Saito Akinori, for helping and kind assistance in conducting the laboratory tests.

Sincere appreciation is expressed to the Indonesian Government through Indonesian Directorate General of Higher Education (DIKTI) for providing a Scholarship so that made possible to pursue my doctoral study at Saga University, Saga, Japan. Sincere thanks are extended to my colleagues, DR. Kartika S., Mr. Sudeepong A., Mr. Xu Fang, Mr. Shailesh S., Mr. Zhou Yang, Mr. Nachanok C., and all Indonesian students in Saga for their kindness help. A word of thanks goes to the secretary in the research team, Ms. Kanada Yasuko for her kindness help.

I would like to express my love and sincere gratitude to my parents, brothers and sisters for their support, love and constant pray. A special note of appreciation also goes to my long life partner, Davy Kawengian, for his consistent patience, understanding and encouragement which made this study more than just successful. Finally, I dedicate this piece of work to my beloved children, Daniel, Gervic and Devanov who have being a truly encouragement in my life.

Steeva Gaily Rondonuwu

September 2015
ABSTRACT

Recently preloading soft clayey deposit by surcharge load, vacuum pressure and its combination have been extensively used. The advantages of soft ground improvement using vacuum pressure alone or combined with surcharge load are increasing the effective stress, easy for installation, no need heavy machinery and environmentally friendly (no chemical addition). A vacuum pressure is an isotropic consolidation pressure increment, it will induce settlement and inward lateral displacement of the treated ground. While, surcharge load induces settlement and outward lateral displacement due to the shear stress applied. Understanding the deformation characteristics and the influencing factors are important for evaluating and minimizing preloading induced lateral displacement of a ground.

Further, triaxial vacuum consolidation test were conducted using undisturbed Ariake clay samples under different confining stress, active or at-rest earth pressure under the field condition. By comparing the laboratory measured strains with that measured from a field vacuum consolidation project, the field stress state under vacuum consolidation has been identified. It has been found that the effective confinement due to gravity force from the surrounding soils to the vacuum treated area is close to the value of active earth pressure for a zone about 5 m depth from the ground surface, and below it, the effective confinement is between active and at-rest states. Also, a theoretical matching equation has been derived to convert consolidation time in laboratory to the field case. It has been shown that with the matching equation, the results of laboratory vacuum consolidation test under triaxial condition can be used to predict the field settlement (compression) curves, and lateral displacement profile at the edge of the treated area.

In case of combined surcharge load and vacuum pressure, the magnitude of surcharge loading, ratio of surcharge loading to the vacuum pressure are determined on the preliminary design. One way to control the lateral displacement is by reducing the surcharge loading rate (SLR). The effect of SLR on a lateral displacement was investigated using radial drainage laboratory consolidation tests (oedometer and triaxial), which simulated the mini-unit cell of a prefabricated vertical drain (PVD) improvement.
Based on the test results, a method for determining the optimum value of SLR being resulting in minimum lateral displacement has been established, which used consolidation properties of a deposit and PVD, and the geometric and spacing effect of PVD.
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>CHAPTER</th>
<th>TITLE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Title page</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Acknowledgements</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Abstract</td>
<td>i</td>
</tr>
<tr>
<td></td>
<td>Table of contents</td>
<td>iii</td>
</tr>
<tr>
<td></td>
<td>List of figures</td>
<td>vii</td>
</tr>
<tr>
<td></td>
<td>List of tables</td>
<td>xi</td>
</tr>
<tr>
<td></td>
<td>List of notations</td>
<td>xii</td>
</tr>
<tr>
<td>1</td>
<td>Introduction</td>
<td>1</td>
</tr>
<tr>
<td>1.1</td>
<td>Background</td>
<td>1</td>
</tr>
<tr>
<td>1.2</td>
<td>Objectives and scopes of study</td>
<td>2</td>
</tr>
<tr>
<td>1.3</td>
<td>Organization of the thesis</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>Literature review</td>
<td>4</td>
</tr>
<tr>
<td>2.1</td>
<td>Introduction</td>
<td>4</td>
</tr>
<tr>
<td>2.2</td>
<td>Factors influencing lateral displacement</td>
<td>4</td>
</tr>
<tr>
<td>2.2.1</td>
<td>Soil Properties</td>
<td>5</td>
</tr>
<tr>
<td>2.2.1.1</td>
<td>Deformation properties</td>
<td>5</td>
</tr>
<tr>
<td>2.2.1.2</td>
<td>Consolidation properties</td>
<td>5</td>
</tr>
<tr>
<td>2.2.1.3</td>
<td>Undrained shear strength ($s_u$)</td>
<td>5</td>
</tr>
<tr>
<td>2.2.1.4</td>
<td>Stress state</td>
<td>5</td>
</tr>
<tr>
<td>2.2.2</td>
<td>Loading condition</td>
<td>6</td>
</tr>
<tr>
<td>2.2.2.1</td>
<td>Surcharge loading rate (SLR)</td>
<td>6</td>
</tr>
<tr>
<td>2.2.2.2</td>
<td>Ratio of surcharge loading to vacuum pressure</td>
<td>7</td>
</tr>
<tr>
<td>2.2.3</td>
<td>Summary and comments</td>
<td>7</td>
</tr>
<tr>
<td>2.3</td>
<td>Ground improvement with prefabricated vertical drain (PVD)</td>
<td>7</td>
</tr>
<tr>
<td>2.3.1</td>
<td>Introduction</td>
<td>7</td>
</tr>
<tr>
<td>2.3.2</td>
<td>Consolidation theory of subsoil with vertical drain</td>
<td>8</td>
</tr>
<tr>
<td>2.4</td>
<td>Ground deformation characteristics</td>
<td>11</td>
</tr>
<tr>
<td>2.4.1</td>
<td>Preloading by surcharge load</td>
<td>11</td>
</tr>
<tr>
<td>2.4.1.1</td>
<td>General consideration</td>
<td>11</td>
</tr>
</tbody>
</table>
2.4.1.2 Methods for predicting the ground deformation under surcharge load

2.4.2. Preloading by vacuum pressure
   2.4.2.1 General consideration
   2.4.2.2 Methods for predicting ground deformations under vacuum pressure

2.4.3. Preloading by surcharge load and vacuum pressure

2.4.4. Summary and comments

3 Field stress state induced by vacuum consolidation
   3.1 Introduction
   3.2 Basic consideration
   3.3 Radial drainage laboratory triaxial consolidation test
      3.3.1 Test method
      3.3.2 Sampling site and soil properties
      3.3.3 Equipment
      3.3.4 Cases tested
      3.3.5 Test procedures
         3.3.5.1 Sample preparation
         3.3.5.2 Pre-consolidation
         3.3.5.3 Changing of the confining pressure during consolidation process
      3.3.6 Test results
         3.3.6.1 Settlement
         3.3.6.2 Excess pore pressure
         3.3.6.3 Horizontal strain
         3.3.6.4 Stress ratio \( K \) of Triaxial test
         3.3.6.5 \( K-\varepsilon_h \) relationship
      3.3.7 Summary and comments
   3.4 Vacuum consolidation project in Saga site
   3.5 Deformation conditions in laboratory test and field case
      the sampling site
3.6 Predicting field deformation by laboratory test results
  3.6.1 Matching the field and laboratory consolidation times
  3.6.2 Parameters
  3.6.3 Comparison of vertical and horizontal strains
  3.6.4 Predicting field compression curves

3.7 Summary and comments

4 Surcharge loading rate for minimizing lateral displacement with vacuum pressure
  4.1 Introduction
    4.1.1 Background
    4.1.2 Basic consideration
  4.2 Radial drainage laboratory oedometer consolidation test
    4.2.1 Sampling site and soil properties
    4.2.2 Equipment
    4.2.3 Cases tested
    4.2.4 Test Procedures
      4.2.4.1 Sample preparation
      4.2.4.2 Pre-consolidation
      4.2.4.3 Consolidation test
    4.2.5 Test results
      4.2.5.1 Introduction
      4.2.5.2 Settlement
      4.2.5.3 Lateral earth pressure (LEP) increment
      4.2.5.4 Excess pore water pressure
      4.2.5.5 Coefficient of lateral earth pressure acting on the wall ($K_w$)
    4.2.6 Summary and comments
  4.3 Radial drainage laboratory triaxial consolidation test
    4.3.1 Equipment and material used
    4.3.2 Cases tested
    4.3.3 Test procedures
4.3.3.1 Sample preparation 79
4.3.3.2 Pre-consolidation 79
4.3.3.3 Consolidation under combined loads 79
4.3.4 Test results 80
  4.3.4.1 Settlement 80
  4.3.4.2 Excess pore pressure 81
  4.3.4.3 Horizontal strain 83
  4.3.4.4 Stress ratio ($K$) 85
  4.3.4.5 $K$ - $\varepsilon_h$ relationship 89
4.3.5 Summary and comments 90
4.4 Design method for minimizing lateral displacement 91
  4.4.1 Main factors influencing the lateral displacement 91
    4.4.1.1 Effect of loading ratio ($LR$) 91
    4.4.1.2 Effect of surcharge loading rate ($SLR$) 93
  4.4.2 Introducing a dimensionless parameter $\alpha$ 93
  4.4.3 Design curves 94
    4.4.3.1 $\alpha$ - $K_w$ relationship 94
    4.4.3.2 $\alpha$ - $K$ relationship 96
  4.4.4 $\alpha$ versus horizontal strain ($\varepsilon_h$) 97
  4.4.5 Summary 98
4.5 Summary 99
5 Conclusions 101
  5.1 Introduction 101
  5.2 Main factors affecting lateral displacement 101
  5.3 Proposed loading method for minimizing lateral displacement 102
  5.4 Field stress state under vacuum consolidation 103
  5.5 Predicting vacuum consolidation induced ground deformation 103
References 104
# List of Figures

<table>
<thead>
<tr>
<th>FIGURE NO.</th>
<th>TITLE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Illustration of stress state and deformation pattern of soil slices in the ground under vacuum consolidation</td>
<td>6</td>
</tr>
<tr>
<td>2.2</td>
<td>Illustration of preloading using PVD</td>
<td>8</td>
</tr>
<tr>
<td>2.3</td>
<td>Unit cell of vertical drain application</td>
<td>8</td>
</tr>
<tr>
<td>2.4</td>
<td>Vertical stress profile: (a) initial condition, (b) surcharge load and (c) vacuum pressure</td>
<td>10</td>
</tr>
<tr>
<td>2.5</td>
<td>Ground deformation induced by surcharge load</td>
<td>11</td>
</tr>
<tr>
<td>2.6</td>
<td>Ground deformation induced by vacuum pressure</td>
<td>13</td>
</tr>
<tr>
<td>2.7</td>
<td>Predicted field deformation induced by vacuum consolidation in Saga site</td>
<td>17</td>
</tr>
<tr>
<td>2.8</td>
<td>Ground deformation induced by surcharge by combination vacuum pressure and surcharge load</td>
<td>19</td>
</tr>
<tr>
<td>2.9</td>
<td>Effect of loading rate under combination of surcharge load and vacuum pressure</td>
<td>20</td>
</tr>
<tr>
<td>3.1</td>
<td>Illustration of ground improved by vacuum consolidation</td>
<td>23</td>
</tr>
<tr>
<td>3.2</td>
<td>Illustration of changing stress conditions</td>
<td>24</td>
</tr>
<tr>
<td>3.3</td>
<td>Locations of test and sampling sites</td>
<td>25</td>
</tr>
<tr>
<td>3.4</td>
<td>Soil profiles and some index and mechanical properties of sample on vacuum consolidation project</td>
<td>25</td>
</tr>
<tr>
<td>3.5</td>
<td>Soil properties of Ariake clay on sampling site</td>
<td>26</td>
</tr>
<tr>
<td>3.6</td>
<td>Laboratory tri-axial test device</td>
<td>29</td>
</tr>
<tr>
<td>3.7</td>
<td>Settlement-time relationship (1.5 m depth)</td>
<td>32</td>
</tr>
<tr>
<td>3.8</td>
<td>Settlement-time relationship (3.5 m depth)</td>
<td>32</td>
</tr>
<tr>
<td>3.9</td>
<td>Settlement-time relationship (7.5 m depth)</td>
<td>33</td>
</tr>
<tr>
<td>3.10</td>
<td>Settlement-time relationship (9.5 m depth)</td>
<td>33</td>
</tr>
<tr>
<td>3.11</td>
<td>Variation of pore water pressure (1.5 m depth)</td>
<td>34</td>
</tr>
<tr>
<td>3.12</td>
<td>Variation of pore water pressure (7.5 m depth)</td>
<td>35</td>
</tr>
<tr>
<td>3.13</td>
<td>Shape of specimen at end of consolidation (1.5 and 3.5 m depth)</td>
<td>36</td>
</tr>
</tbody>
</table>
3.14 Horizontal strain-depth relationship under $K_0$ and $K_a$ condition
3.15 Time - $K$ variation (1.5 m depth)
3.16 Time - $K$ variation (3.5 m depth)
3.17 Horizontal strain $\varepsilon_h$ - $K$ relationship
3.18 Locations of test sections
3.19 Plan layout of instrumentation points
3.20 Measured vacuum pressure under the air tightening sheet
3.21 Measured excess pore pressures in the subsoil
3.22 Measured settlement - time curves
3.23 Model for calculating compressions of each subsoil layer
3.24 Comparison of vertical strains
3.25 Comparison of horizontal strains
3.26 Comparison of compression curves of soil layer of 0 - 4.1 m depth
3.27 Comparison of compression curves of soil layer of 4.1- 10.3 m depth
4.1 Illustration of the pressure change on the wall of a consolidation ring
4.2 Sampling site location
4.3 Laboratory odometer test device
4.4 Cylinder device for making hole at the centre of specimen
4.5 Illustration of applied the combination of surcharge load and vacuum pressure
4.6 Time- settlement curves ($LR = 1.0; d_w = 8 \text{ mm}$)
4.7 Time- settlement curves ($LR = 1.0; d_w = 4 \text{ mm}$)
4.8 Time- settlement curves ($SLR = 10 \text{ kPa/ 30 min}$)
4.9 Time- settlement curves ($SLR = 10 \text{ kPa/ 60 min}$)
4.10 Effect of surcharge loading rate to the total lateral earth pressure ($LR =1.0; d_w = 8 \text{ mm}$)
4.11 Effect of surcharge loading rate to the total lateral earth
Variation of total lateral earth pressure ($\sigma'_v0 = 20\, kPa$) 65
Variation of total lateral earth pressure ($\sigma'_v0 = 40\, kPa$) 65
Variation of total lateral earth pressure ($\sigma'_v0 = 80\, kPa$) 66
Variation of total lateral earth pressure ($SLR = 10\, kPa/30\, \text{min}$) 66
Variation of total lateral earth pressure ($SLR = 10kPa/60\, \text{min}$) 67
Excess pore pressure variation ($LR = 1.0; d_w = 8\, \text{mm}$) 68
Excess pore pressure variation ($LR = 2.0; d_w = 8\, \text{mm}$) 68
Excess pore pressure variation ($LR = 1.0; d_w = 4\, \text{mm}$) 69
Excess pore pressure variation ($\sigma'_v0 = 20\, kPa$) 69
Excess pore pressure variation ($\sigma'_v0 = 40\, kPa$) 70
Excess pore pressure variation ($\sigma'_v0 = 80\, kPa$) 71
Excess pore pressure variation ($SLR = 10\, \text{kPa/30\, min}$) 71
Excess pore pressure variation ($SLR = 10\, \text{kPa/60\, min}$) 72
Variation of $K_w$ values ($LR =1.0; d_w = 8\, \text{mm}$) 72
Variation of $K_w$ values ($\sigma'_v0 = 20\, kPa$) 74
Variation of $K_w$ values ($\sigma'_v0 = 40\, kPa$) 75
Variation of $K_w$ values ($\sigma'_v0 = 80\, kPa$) 75
Variation of $K_w$ values ($SLR = 10\, \text{kPa/30\, min}$) 76
Settlement-time relationship ($\sigma'_v0 = 20\, kPa$) 80
Settlement-time relationship ($SLR = 10\, \text{kPa/30\, min}$) 81
Variation of pore water pressure ($\sigma'_v0 = 20\, kPa$) 82
Variation of pore water pressure ($\sigma'_v0 = 60\, kPa$) 82
Variation of pore water pressure ($SLR = 10\, \text{kPa/30\, min}$) 83
Shape of specimen at end of consolidation ($\sigma'_v0 = 20\, kPa$) 84
Shape of specimen at end of consolidation ($\sigma'_v0 = 60\, kPa$) 84
Horizontal strain ($\varepsilon_h - SLR$ relationship under triaxial test with different initial vertical effective stress ($\sigma'_v0$) 85
Time - $K$ variation ($\sigma'_v0 = 20\, kPa$) 87
Time - $K$ variation ($\sigma'_v0 = 60\, kPa$) 88
Time - $K$ variation ($SLR = 10\, \text{kPa/30\, min}$) 89
| 4.41 | Horizontal strain $\varepsilon_h$ - $K$ relationship | 90 |
| 4.42 | $SLR$- $K_w$ relationship | 92 |
| 4.43 | $SLR$- $K$ relationship | 92 |
| 4.44 | $K_w$ - $\alpha$ relationship on odometer consolidation test $\sigma'_{i0} = 0$ | 95 |
| 4.45 | $K_w$ - $\alpha$ relationship on odometer consolidation test $\sigma'_{i0} > 0$ | 95 |
| 4.46 | $K$ - $\alpha$ relationship on tri-axial consolidation test | 96 |
| 4.47 | $\varepsilon_h$ - $\alpha$ relationship on tri-axial consolidation test | 97 |
List of Tables

<table>
<thead>
<tr>
<th>TABLE NO.</th>
<th>TITLE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>Ariake clay properties</td>
<td>26</td>
</tr>
<tr>
<td>3.2</td>
<td>Test cases for undisturbed Ariake clay samples</td>
<td>29</td>
</tr>
<tr>
<td>3.3</td>
<td>Values of parameter related to PVD consolidation</td>
<td>45</td>
</tr>
<tr>
<td>4.1</td>
<td>Basic soil properties</td>
<td>55</td>
</tr>
<tr>
<td>4.2</td>
<td>Cases tested using the modified odometer</td>
<td>57</td>
</tr>
<tr>
<td>4.3</td>
<td>Test cases for reconstituted Ariake clay samples</td>
<td>78</td>
</tr>
</tbody>
</table>
List of Notations

\begin{itemize}
  \item \textit{B} \quad \text{half width area}
  \item \textit{c}_{c} \quad \text{Index compression of the soil}
  \item \textit{c}_{h} \quad \text{coefficient of consolidation in the horizontal direction}
  \item \textit{c}_{v} \quad \text{coefficient of consolidation in the vertical direction}
  \item \textit{c}' \quad \text{effective cohesion}
  \item \textit{D}_{e} \quad \text{diameter of unit cell}
  \item \textit{D}_{L} \quad \text{the diameter of the soil sample used on laboratory triaxial consolidation test}
  \item \textit{d}_{s} \quad \text{diameter of the smear zone}
  \item \textit{d}_{w} \quad \text{diameter of the drain}
  \item \textit{e} \quad \text{void ratio}
  \item \textit{F(n)} \quad \text{a factor considering the effect of drain spacing}
  \item \textit{H} \quad \text{depth of PVD improve soil}
  \item \textit{k} \quad \text{hydraulic conductivity}
  \item \textit{k}_{h} \quad \text{horizontal hydraulic conductivity of natural soil}
  \item \textit{k}_{s} \quad \text{horizontal hydraulic conductivity of a smear zone}
  \item \textit{K} \quad \text{ratio of horizontal effective stress to the value in the vertical direction}
  \item \textit{K}_{a} \quad \text{coefficient of active lateral earth pressure}
  \item \textit{K}_{ao} \quad \text{coefficient lateral earth pressure between at-rest and active}
  \item \textit{K}_{w} \quad \text{coefficient of earth pressure on the wall of the consolidation ring}
  \item \textit{K}_{0} \quad \text{coefficient of at-rest lateral earth pressure}
  \item \textit{l} \quad \text{drainage length of a PVD}
  \item \textit{LR} \quad \text{ratio of loading}
  \item \textit{n} \quad \text{spacing ratio}
  \item \textit{NLD} \quad \text{normalized lateral displacement}
  \item \textit{p}_{a} \quad \text{atmospheric pressure}
  \item \textit{p}_{n} \quad \text{index pressure}
  \item \textit{p}_{em} \quad \text{embankment pressure}
  \item \textit{p}_{vac} \quad \text{vacuum pressure applied to a deposit}
  \item \textit{p}' \quad \text{mean effective stress}
  \item \textit{q}_{w} \quad \text{discharge capacity of a PVD}
\end{itemize}
$r$ radial
$r_a$ radial distance of pore water pressure measurement
$r_e$ radius of a unit cell of PVD improvement
$r_s$ radius of smear zone
$r_w$ equivalent radius of the vertical drain
$RLS$ a parameter
$s$ ratio of radius of smear zone to the equivalent radius of the vertical drain
$S$ settlement
$S_f$ maximum settlement
$SLR$ surcharge loading rate
$s_{su}$ undrained shear strength
$t$ time
$t_F$ time of field consolidation
$t_L$ time of laboratory consolidation
$T_h$ time factor for horizontal drainage
$T_r$ a time factor dimensionless
$u_a$ pore water pressure measurement on the radial distance $r_a$
$u_e$ measured excess pore water pressure
$U_h$ average degree of consolidation in horizontal direction
$w_p$ plastic limit
$wl$ liquid limit
$y_m$ maximum lateral displacement
$z$ depth from the ground surface
$z_c$ depth of the tension crack
$z_l$ depth of the soil with no lateral displacement
$z_w$ depth of ground water level
$\alpha$ a parameter which function of $SLR$ and rate of consolidation
$\alpha_l$ a dimensionless parameter
$\alpha_{1\text{min}}$ a factor with minimum value
$\beta$ a dimensionless multiple
$\gamma_t$ total unit weight of soil
\( \gamma_w \) unit weight of water
\( \gamma' \) effective unit weight of the soil
\( \delta_h \) lateral displacement
\( \delta_{nm} \) the net maximum lateral displacement
\( \Delta \sigma_s \) applied surcharge load increment
\( \Delta \sigma_{vac} \) applied vacuum pressure increment
\( \varepsilon_h \) horizontal strain
\( \varepsilon_{vol} \) volumetric strain
\( \varepsilon_{vv} \) vertical strain
\( \lambda \) virgin compression index
\( \sigma'_h \) horizontal effective stress
\( \sigma'_{ho} \) initial horizontal effective stress
\( \sigma'_v \) vertical effective stress
\( \sigma'_{hw} \) horizontal stress (effective or total) acting on the wall of consolidation ring
\( \sigma_{ep} \) measured total horizontal earth pressure on the wall
\( \sigma'_{vo} \) initial vertical effective stress
\( \mu \) a parameter considering the effect of PVD spacing, smear and well resistance
\( \mu_F \) \( \mu \) value of field consolidation
\( \mu_L \) \( \mu \) value of laboratory consolidation
\( \phi' \) internal friction angle of the soil
CHAPTER 1
INTRODUCTION

1.1 Background

Soft clayey soils are widely deposited in many places around the world, especially in coast areas and river deltas. Due to the low strength, high compressibility and low permeability of soft clayey, it is not suitable for most civil engineering constructions. Therefore, the soft ground improvement has to be done before using the land for building or transportation system construction.

There are ground improvement methods such as adding chemicals, thermal method, reinforcements, mixing cement as well as using stone column or granular piles. Recently, the preloading method by surcharge load and vacuum pressure, or combination of both of them assisted by vertical drain has been conducted in many mega projects, for example: Changi Airport in Singapore, Kansai International Airport in Japan, Second Bangkok International Airport in Thailand etc. The advantages of using vacuum pressure with vertical drain alone or combined with surcharge load are, shortening the consolidation period, increasing the effective stress, reducing construction time and it is environmentally friendly (Chai et. al 2006).

Application of a vacuum preloading will induce inward lateral displacement (toward the centre of the improved area) (Chai et al. 2005a; 2009) and it will induce tension crack on the vicinity area. The vacuum pressure induced inward lateral displacement will change the earth pressure applied to the treated area from the surrounding area. Understanding the stress state of a deposit under vacuum pressure is important for predicting vacuum pressure induced ground deformation (Imai 2005; Chai et al. 2005; Robinson et al. 2012).

Preloading using surcharge load assist by vertical drain can increase the strength of a deposit. However applied surcharge load will induce shear stress in the ground and cause outward lateral displacement (moved away from the centre of the area improved). Besides, to create a large preloading pressure, a high embankment is needed. In addition some methods are needed to minimize the lateral displacement. Combination of surcharge load and vacuum pressure may be a good option to minimize the lateral displacement (Indraratna et al. 2011; Rujiatkamjorn et al. 2009; Chai et al. 2005).
1.2 **Objectives and scopes of study**

The first objective of this study is to understand the stress state of soft clay deposits under vacuum pressure and to develop a method for predicting the field deformation by laboratory test results. The second one is under combination of vacuum pressure and surcharge load, to establish a method for design optimum surcharge loading rate for minimizing lateral displacement of the ground.

Those objectives can be achieved by means:

1) Laboratory consolidation test under oedometer condition using vacuum and/ or vacuum plus surcharge load.

2) Laboratory consolidation test under triaxial condition with vacuum and/ or vacuum plus surcharge load.

3) Analysing and comparing the laboratory and field test results.

The lateral displacement of a ground is influenced by the loading condition (magnitude of loading and surcharge loading rate), as well as the consolidation, strength, and deformation characteristics of the soil deposit (Ong and Chai 2012; Mesri and Khan 2012). However, there is no systematic study, about the degree of influence of these factors reported in the literature. Therefore, the main influencing factors (e.g. ratio of surcharge loading to the vacuum pressure and surcharge loading rate and initial effective stress state on ground lateral displacement will be investigated systematically in this study by laboratory consolidation tests.

1.3 **Organization of the thesis**

This dissertation consists of five chapters. Following the first Chapter of introduction, Chapter two contains the literature review which covers the ground deformation characteristics relate to vertical drain assisted preloading using vacuum pressure and the combination with surcharge load. Also, it describes the influencing factors for lateral displacement and the related theories.

Chapter three presents the radial drainage laboratory triaxial consolidation test under vacuum pressure, using undisturbed Ariake clay sample and the comparison of the test results with a field vacuum consolidation project to evaluate the stress state of a deposit under a vacuum pressure. Further a method for predicting the field deformation by the laboratory test results is proposed.
Chapter four provides the radial drainage laboratory consolidation tests under oedometer and triaxial condition with combined vacuum pressure and surcharge load and their results. Further, a method to design a preloading using combination of vacuum pressure and surcharge loads that result in minimum lateral displacement of a deposit is proposed.

Finally Chapter five consists of the conclusions of the research.
CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

Preloading vertical drain improved deposit with vacuum pressure alone or with the combination of vacuum pressure and surcharge load has become popular recently. Soft ground improvement using vacuum pressure has been used in many geotechnical works (Bergado et al. 1997; 1998; 2002; Shang et al. 1998; Yan and Chu 2003; 2005; Chai et al. 2005a; 2006). The advantages of vacuum pressure with vertical drain are: shorten the consolidation period; easy for installation; does not require heavy machinery; and does not need chemical addition. Therefore, it is environmentally friendly. Vacuum pressure can induce settlement and inward lateral displacement (toward the centre of improvement area) during the consolidation process. Calculating or predicting the deformation induced by vacuum consolidation is needed for geotechnical design.

Embankment preloading induces settlement and its shear stress induces outward lateral displacement (move away from the centre). The combination of vacuum pressure and surcharge load can increase the preloading pressure, shorten the construction time in case of road construction project (Bergado et al. 1998; Tran et al. 2004; Chai et al. 2006; Mesri and Khan 2012) and minimize the lateral displacement.

The lateral displacement of the ground can cause significant damage for existing structures. Therefore, controlling and minimizing preloading induced lateral displacement is an important issue for design preloading project with prefabricated vertical drain (PVD). This chapter will review the literatures on: 1) Factors influencing lateral displacement; 2) Consolidation theory with PVD; 3) Ground deformation characteristics induced by embankment load, vacuum pressure and their combination.

2.2 Factors influencing lateral displacement

The factors influencing lateral displacement of a deposit can be categorized on two groups: (1) soil properties and (2) loading condition.
2.2.1  Soil Properties

2.2.1.1 Deformation properties

For both embankment loading induced settlement and outward lateral displacement and vacuum pressure induced settlement and inward lateral displacement, the compression index \( C_c \) of a deposits is an important influencing parameter. The larger the value of \( C_c \), the larger the lateral displacement under the same loading condition.

2.2.1.2 Consolidation properties

The outward lateral displacement induced by surcharge load is mainly due to the shear deformation. However, the vacuum pressure induced lateral displacement occurs during consolidation process. The faster the rate of consolidation the quicker the strength gained of the deposit, and lesser the amount of the lateral displacement.

2.2.1.3 Undrained shear strength \( (s_u) \)

The undrained shear strength \( (s_u) \) strongly influences the deformation under embankment fill or vacuum pressure (Mesri and Khan 2012). The higher the \( s_u \) value, the smaller the shear stress induced lateral displacement.

2.2.1.4 Stress state

The lateral and vertical deformation of soils depends on stress state in the ground (Chai et al. 2005; Mesri and Khan 2012; Robinson et al. 2012). Chai et al. (2005a) introduced a parameter of stress ratio \( (K) \) to judge whether a vacuum pressure can induce inward lateral displacement of a deposit, as expressed by Eq. (2.1),

\[
K = \frac{\Delta \sigma_{vac}}{\Delta \sigma_{vac} + \sigma'_{vo}}
\]

(2.1)

where \( \Delta \sigma_{vac} \) is vacuum pressure applied and \( \sigma'_{vo} \) is initial vertical effective stress. If \( K \leq K_o \) (at-rest earth pressure coefficient), there will be no inward lateral displacement and vice versa.
Chai et al. (2005a) discussed the different zone of the ground with different lateral displacement induced by vacuum consolidation (Fig. 2.1). From the figure, at the ground surface, inward lateral displacement induced by vacuum pressure may cause tension crack with a depth of $z_c$. For a soil element located a depth less than $z_c$, the stress state can be approximated by that shown in Fig. 2.1(a). However below $z_c$ and above $z_l$ (where there is no inward lateral displacement) the coefficient of lateral earth pressure is between $K_a$ (active earth pressure) and $K_o$.

Robinson et al. (2012) stated that there is more lateral inward compression close to the ground surface and it decreases with depth. Below the tension crack zone the earth pressure in the soil will be between the at-rest state ($K_o$ condition) and the active state ($K_a$ condition).

### 2.2.2 Loading condition

#### 2.2.2.1 Surcharge loading rate (SLR)

Surcharge loading rate (SLR) is a parameter to indicate how fast the surcharge load is applied. The outward lateral displacement under a surcharge load is mainly
induced by shear stress, and the rate of surcharge load has an effect on the shear deformation. Therefore by controlling the SLR, one can control the lateral displacement.

### 2.2.2.2 Ratio of surcharge loading to vacuum pressure

Loading ratio ($RL$) is defined as the ratio of total surcharge load applied divided by vacuum pressure applied. The larger the total surcharge load the larger the loading ratio as well as the tendency of outward lateral displacement (Ong and Chai 2012).

### 2.2.3 Summary and comments

The factors influencing lateral displacement can be categorized as soil properties and loading conditions. The soil properties include (1) compression index ($C_c$); (2) coefficient of consolidation ($c_v$ and $c_h$); (3) undrained shear strength ($s_u$) and (4) stress state. While the factor influencing lateral displacement due to loading condition which include surcharge loading rate (SLR) and ratio of surcharge loading to the vacuum pressure ($LR$). All of these factors will influence the lateral deformation of a deposit under the vacuum pressure and with the combination of surcharge load. Thus, to predict the lateral deformation, all these factors need to be considered directly or indirectly.

### 2.3 Ground improvement with prefabricated vertical drain (PVD)

#### 2.3.1 Introduction

In 1930’s, the application of vertical drain (Fig. 2.2) was developed by using the sand drain in California. In Sweden, during the same decade, Kjellman introduced the prototype of a prefabricated drain made from cardboard (Jamiolkowski et al., 1983). Since then, the development has continued and now there are more than hundred different types of drains available in the market. Nowadays, the prefabricated vertical drains (PVDs) are most widely used.

The principal of vertical drain are, to shorten the drainage path and therefore the consolidation period. The installation of vertical drain may create disturbance to the soil nearby, in which the hydraulic conductivity may be reduced. Such disturbance zone due to the vertical drain installation is called smear zone (Fig. 2.3).
2.3.2 Consolidation theory of subsoil with vertical drain

In the study of vertical drain two cases were considered by Barron (1948), namely (a) free strain case, and (b) equal strain case. The free strain case assumes that the ground surface is of a flexible nature, where there will be equal distribution of surface load. On the other hand, the equal strain hypothesis says the ground surface is rigid, and thus the surface settlement will be the same all over. In term of average degree of consolidation, the results of the analysis are not much different, but the equal
strain theory is simpler. Therefore, the equal strain solution has been widely used. The average degree of consolidation in horizontal direction \((U_h)\) under the ideal condition (no smear effect and well resistance) can be calculated as in Eq. (2.2),

\[
U_h = 1 - \exp\left(\frac{-8T_h}{F(n)}\right)
\]  

(2.2)

where \(T_h\) = time factor for horizontal drainage, \(F(n)\) = a factor considering the effect of drain spacing and they can be expressed as in Eq. (2.3) and (2.4),

\[
T_h = \frac{c_h t}{D_e e^2}
\]

(2.3)

\[
F(n) = \left[\frac{n^2}{1-n}\right]\ln(n) - \frac{3}{4} + \frac{1}{n^2}
\]

(2.4)

where \(c_h\) = coefficient of consolidation of clayey deposit in horizontal direction, \(n = D_e / d_w\) is the spacing ratio, \(D_e\) = diameter of a unit cell (a cylinder with a PVD in the centre) and \(d_w\) = equivalent diameter of a PVD.

Hansbo (1981) has modified the equation developed by Barron (1948) for considering smear well resistance and smear effect. Hansbo’s (1981) solution can be expressed as in Eq. (2.5):

\[
U_h = 1 - \exp\left(\frac{8T_h}{\mu}\right)
\]

(2.5)

where \(\mu\) = a parameter considering the effect of PVD spacing, smear and well resistance and it can be expressed as in Eq. (2.6),

\[
\mu = \ln\left(\frac{D_e}{d_s} + \frac{k_h}{k_s}\right) \ln\left(\frac{d_s}{d_w}\right) - \frac{3}{4} + \pi \cdot \frac{2H^2 k_h}{3 q_w}
\]

(2.6)

where \(d_s\) = diameter of smear zone, \(k_h, k_s\) = horizontal hydraulic conductivities of natural soil and smear zone, respectively, \(H\) = drainage length of PVD and \(q_w\) = discharge capacity of a PVD.
Fig. 2.4 Vertical stress profile: (a) initial condition, (b) surcharge load and (c) vacuum pressure (After Elgamal and Adalier 1996)
2.4 Ground deformation characteristics

2.4.1 Preloading by surcharge load

2.4.1.1 General Consideration

Preloading by using surcharge loading assisted by PVD is a common method to improve the strength and stiffness of the soft clay deposits. The placement of the embankment soil on the ground will increase the total stress and generate the excess pore pressure (Fig. 2.4b), and followed by the increasing of the effective stress. The construction of embankment, imposes immediate shear stress and induce the outward (away from the centre of soft ground improved) lateral displacement. Further the application of embankment load can cause the instability problem especially when the construction spread is high.

The characteristics of ground deformation induced by surcharge load are illustrated in Fig. 2.5. The surcharge load results in settlements as well as the outward lateral displacement (away from the centre of the loading area) of the ground (Shang et al. 1998; Chai et al. 2005a). The lateral displacement is mainly caused by the shear stress induced by the surcharge load and normally it is an immediate process.

![Fig. 2.5 Ground deformation induced by surcharge load](image-url)
2.4.1.2 Methods for predicting the ground deformation under surcharge load.

Loganathan (1980) discussed the method to predict the deformation under embankment loading based on volumetric strain. For the undrained condition there is no volume change, so that the volume change due to downward vertical settlement is equal to the volume change of outward lateral displacement.

Tavenas (1979) reported that the lateral displacement induced by embankment load is a function of settlement. During the construction or consolidation process, the maximum lateral displacement ($y_m$), depends on the settlement under the centre of the embankment. So the lateral displacement developed during construction and consolidation can be expressed as a function of settlement $y_m = f(s)$ of the ground. In case of the embankment constructed gradually, the lateral displacement increased slowly in the initial construction stage (ground in over consolidated state) and became larger at the end of embankment load application (ground becomes a normally consolidated state) (Tavenas et al. 1979).

Tavenas (1980) argued that the magnitude of the lateral displacement induced by embankment loading mainly influenced by the factor of safety ($FS$) of the ground.

Chai et al. (2013) discussed the effect of partial drainage during an embankment construction period which can increase the effective stress and strength of the subsoil, and therefore can reduce the amount of lateral displacement induced by shear stresses due to embankment loading.

2.4.2 Preloading by vacuum pressure

2.4.2.1 General consideration

Commonly, vacuum pressure application is assisted by PVD to accelerate the consolidation process of the soft deposit. The application of vacuum pressure to the soil mass generates a negative pore water pressure. Then, the effective stress of the soil is increased while the total stress remains unchanged (Fig. 2.4c). The characteristics of ground deformation induced by vacuum pressure are illustrated in Fig. 2.6. The vacuum pressure is an isotropic incremental consolidation pressure, which will result in settlement and inward lateral displacement of a ground (towards the centre of the loading area) (Chai et al. 2005a; Indraratna et al. 2007; Mesri and Khan 2012). This condition will take place slowly (time dependent), following the consolidation process.
This inward lateral displacement sometimes may cause surface cracks nearby the treatment area (Shang et al. 1998; Chu et al. 2000; Chai et al. 2005a).

2.4.2.2 Methods for predicting ground deformations under vacuum pressure

(1) Magnitude of settlement

Chai et al. (2005a; 2006; 2010) discussed that vacuum consolidation induces the same or less settlement than that from surcharge load with the same magnitude.

Mesri and Khan (2012), argued that there is no difference in magnitude and rate of settlement for a vacuum load and an equivalent surcharge load.

(2) Method for predicting lateral displacement

(a) Imai’s (2005) method

Imai (2005) proposed a method to predict the lateral displacement and the strains induced by vacuum consolidation. He introduced an isotropic index parameter (I) which is the function of lateral earth pressure coefficient ($K_0$), initial stress and the vacuum pressure ($\Delta\sigma_{vac}$) applied. For calculating I as in Eq. (2.7),

$$I = \frac{\Delta\sigma_{vac}}{\Delta\sigma_{tv}} = 1 - (K_0 - K_n) \frac{\sigma_{tv0}}{\Delta\sigma_{vac}}$$  \hspace{1cm} (2.7)
where: $\sigma'_{v0}$ is an initial vertical effective stress, $\Delta \sigma'_{h}$ is an incremental horizontal effective stress and $\Delta \sigma'_{v}$ is an incremental vertical effective stress.

It assumes:

- a. Vacuum pressure application is constant through whole deposit
- b. The soil adjacent to the improved zone is in active lateral earth pressure condition
- c. The initial vertical effective stress is the effective overburden pressure.

Hence, the initial horizontal effective stress ($\sigma'_{h0}$) can be calculated from the effective vertical stress ($\sigma'_{v0}$) and by Eq. (2.8),

$$
\sigma'_{h0} = K_0 \sigma'_{v0}
$$

During the sustainable application of vacuum pressure ($\Delta \sigma_{vac}$), the ($\Delta \sigma_{vac}$) is added to both horizontal effective stress and vertical effective stress and the horizontal effective earth pressure will be reduced from at-rest to the active state. Hence, $\sigma'_{v} = \sigma'_{v0} + \Delta \sigma_{vac}$ and $\sigma'_{h} = K_a \sigma'_{v0} + \Delta \sigma_{vac}$. Finally the incremental effective stress can be obtained using Eq. (2.9) and (2.10),

$$
\Delta \sigma'_{v} = \Delta \sigma_{vac}
$$

$$
\Delta \sigma'_{h} = \Delta \sigma_{vac} - (K_0 - K_a) \sigma'_{v0}
$$

The deformation is calculated using elastic theory. Further the isotropic index parameter (I) can be used for judging the lateral displacement, when $I = K_0$ there will be no inward lateral displacement. The depth satisfies $I = K_0$ can be found by the following equation (Eq. (2.11)),

$$
\frac{\sigma'_{v0}}{|\Delta \sigma_{vac}|} = \frac{1-K_0}{K_0-K_a}
$$

If the depth of improved soil with PVD is less than the depth for satisfying Eq. (2.11), then index parameter can be obtained from Eq. (2.12),

$$
I = 1 - (1 - K_0) \frac{z}{H}
$$
where \( z \) = depth from the ground surface, and \( H \) = the depth of PVD improved soil. However this method is not considering the tension crack caused by a vacuum pressure.

(b) Chai et al.’s (2005a) method

Chai et al. (2005a) proposed a method to predict the deformation of PVD improved ground induced by vacuum pressure. Based on the odometer test results, the proposed condition for inward lateral displacement to occur can be expressed as follows (Eq. (2.13)):

\[
\Delta \sigma_{\text{vac}} > \frac{K_0 \sigma_{\text{at}_0}}{1-K_0}
\]  

(2.13)

For field case, Chai et al. (2005a) argues that the inward lateral displacement induced by vacuum pressure may cause tension cracks with a depth of \( z_c \). During the consolidation process the earth pressure will be less than that of at-rest state. Then a coefficient of earth pressure \( (K_{a0}) \), between \( K_0 \) and \( K_a \) has been introduced as:

\[
K_{a0} = \beta K_a + (1 - \beta) K_0
\]  

(2.14)

where \( \beta \) is an empirical factor and in a range of 0.67 to 1 (Chai et al. 2005a). Denote the ground water level as \( z_w \), from Rankine earth pressure theory, the depth of the tension crack \( (z_c) \) can be expressed as in Eq. (2.15) to Eq. (2.17)

\[
z_c = \frac{2c'}{\gamma_t/\gamma_a'} \quad \text{for } z_c < z_w
\]  

(2.15)

\[
z_c = \frac{1}{\gamma_t-\gamma_w} \left( \frac{2c'}{\sqrt{K_a}} - \gamma_w y_z \right), \quad \text{for } z_c > z_w
\]  

(2.16)

\[
K_a = \tan^2 \left( 45^\circ - \frac{\phi'}{z} \right)
\]  

(2.17)

where \( \gamma_t \) = total unit weight of soil; \( \gamma_w \) = unit weight of water; \( c' \) and \( \phi' \) = effective cohesion and the internal friction angle of the soil, respectively. There is a depth, \( z_l \), where there is no lateral displacement occurs, and the value of \( z_l \), can be determined by the following equation,
\[ |\Delta \sigma_{vac}| = \frac{K_0 \sigma_{vo} - \sigma_{av}}{1 - K_0} \]  \hspace{1cm} (2.18)

where \( \sigma_{av}' \) is calculated using Eq. (2.19),
\[
\sigma_{av}' = \begin{cases} 
0 & \text{for } z < z_c \\
K_a z' \gamma' & \text{for } z > z_c 
\end{cases}
\]  \hspace{1cm} (2.19)

and \( \gamma' \) = effective unit weight of soil.

For the vertical strain \( (\varepsilon_{vv}) \) the one dimensional consolidation caused by vacuum consolidation is expressed by Eq. (2.20),
\[
\varepsilon_{vv} = \alpha_1 \frac{\lambda}{1+e} \ln \left( 1 + \frac{\Delta \sigma_{vac}}{\sigma_{vo}} \right) 
\]  \hspace{1cm} (2.20)

where \( e \) = void ratio, \( \lambda \) = the virgin compression index in an \( e\text{-ln}p' \) plot \( (p' = \text{mean effective stress}) \), \( \alpha_1 \) = a factor with minimum value \( (\alpha_{1min}) \) at the ground surface and it will be unity when \( z > z_l \). The \( \alpha_1 \) value is expressed by Eq. (2.21),
\[
\alpha_1 = \alpha_{1min} + \frac{1 - \alpha_{1min}}{\Delta \sigma_{vac}} \left( \frac{K_0 \sigma_{vo} - \sigma_{av}'}{1 - K_0} \right) f o r \Delta \sigma_{vac} > \frac{K_0 \sigma_{vo} - \sigma_{av}}{1 - K_0} 
\]  \hspace{1cm} (2.21)

where \( \alpha_{1min} = 0.80 \) for axisymmetric and \( \alpha_{1min} = 0.85 \) for plane strain condition. While, for simplicity it is assumed that the volumetric strain \( (\varepsilon_{vol}) \) under vacuum consolidation is the same as 1D consolidation, and expressed by Eq. (2.22),
\[
\varepsilon_{vol} = \frac{\lambda}{1+e} \ln \left( 1 + \frac{\Delta \sigma_{vac}}{\sigma_{vo}} \right) 
\]  \hspace{1cm} (2.22)

By knowing the vertical strain \( (\varepsilon_{vv}) \) and volumetric strain \( (\varepsilon_{vol}) \), the inward horizontal strain \( (\varepsilon_h) \) can be expressed as Eq. (2.23) and (2.24)
\[
\varepsilon_h = \frac{1}{2} (\varepsilon_{vol} - \varepsilon_{vv}) \text{ for axisymmetric} 
\]  \hspace{1cm} (2.23)
\[ \varepsilon_h = (\varepsilon_{pot} - \varepsilon_{vv}) \quad \text{for plane strain} \quad (2.24) \]

Once \( \varepsilon_h \) is known, the lateral displacement \( (\delta_h) \) can be simply approximated using Eq. (2.25),

\[ \delta_h = B \cdot \varepsilon_h \quad (2.25) \]

where \( B \) = the half width of the area treated by vacuum consolidation.

(3) Predicting field deformation by laboratory test results

A radial drainage laboratory triaxial consolidation test under vacuum pressure has been conducted for predicting the field deformations induced by vacuum consolidation in Saga, Japan (Tanabashi et al. 2004).

![Graph showing predicted field deformation](image)

**Fig. 2.7 Predicted field deformation induced by vacuum consolidation in Saga site**

*(After Tanabashi et al. 2004)*

However the predicted settlements are smaller than the field measurements (Fig. 2.7). The possible reasons are due to, the pre-consolidation was conducted under isotropic
condition and the effect of earth pressure change during vacuum consolidation was not considered.

### 2.4.3 Preloading by combined surcharge load and vacuum pressure

The advantages of combining surcharge load and vacuum pressure are increase the overall effective load and reduce the construction times in the case of road construction (Chai et al. 2006). The maximum achievable vacuum pressure in the field is about 60 to 80 kPa (Bergado et al. 1998; Tang and Shang 2000; Chai et al. 2012), which is approximately equivalent to 3 or 4 m heights of embankment fill. A vacuum pressure is an isotropic consolidation pressure increment and it tends to induces inward lateral displacement. Therefore, application of a vacuum pressure tends to reduce outward lateral displacement of the ground. While an embankment load will generally cause outward lateral displacement of a deposit. Conceptually, the lateral displacement of a ground may be a “superposition” of surcharge load induced outward lateral displacement and inward lateral displacement due to vacuum pressure. Therefore, lateral displacement induced by combination of surcharge load and vacuum pressure shall be larger (outward) than the one induced by vacuum pressure only, while it should be smaller than the lateral displacement induced by surcharge load only. The possibility of lateral deformation of the ground under the combination of surcharge load and vacuum pressure can be located somewhere between the vacuum pressure induced inward and embankment load induced outward lateral displacement as illustrated on Fig. 2.7 (Ong 2011).

Tran et al. (2004) and Chai et al. (2005b) mentioned that the combination of surcharge load and vacuum pressure is possible to minimize the lateral displacement of the ground.

Chai et al. (2011) reported that under the combination of surcharge load and vacuum pressure, the lateral displacement mainly influenced by the ratio of the magnitude of the surcharge load to the vacuum pressure (Chai et al. 2005b), as well as the rate of application of surcharge load.

Chai et al. (2013), introduced a term of normalized maximum lateral displacement (NLD) which is the ratio of the net maximum lateral displacement (δ_{nm})
to the maximum settlement \(S_f\) at the centre line of an embankment. It is expressed as in Eq. (2.26),

\[
NLD = \frac{\delta_{nm}}{S_f}
\]

Next, an index pressure \(p_n\) corresponding to the end of the embankment construction was introduced, which is defined as the embankment pressure \(p_{em}\) reduced by the magnitudes of the partially consolidated embankment pressure and the partial consolidated vacuum pressure. This index pressure \(p_n\) can be expressed in Eq. (2.27)

\[
p_n = p_{em} - (|p_{vac}| + p_{em})U
\]

where \(p_{vac}\) is the vacuum pressure applied to the deposit and \(U\) is the average degree of consolidation of PVD improved zone at the end of the embankment construction. Finally the ratio of \(p_n\) to \(s_u\) (where \(s_u\) is undrained shear strength of the soil) is designated as the ratio of the index pressure to the representative shear strength (RLS), as shown by Eq. (2.28),
RLS = \frac{p_n}{s_u} \tag{2.28}

Based on the results of several field cases, it has been proposed that \( NLD \) is a function of \( RLS \), and can be expressed by Eq. (2.29),

\[ NLD = 0.09 \, RLS + 0.12 \tag{2.29} \]

When the NLD and \( s_f \) are known then the lateral displacement can be predicted.

![Diagram](image.png)

**Fig. 2.9 Effect of loading rate under combination of surcharge load and vacuum pressure (after Ong 2011)**

Ong (2011) studied the effect of loading rate under combination of surcharge load and vacuum pressure by laboratory consolidation test. The results are shown in Fig. 2.9. It is found that by reducing the rate of surcharge load, the outward lateral displacement can be reduced. However, there is no design method available for minimizing the lateral displacement.
2.4.4 Summary and comments

This section presents the summary about the ground deformation characteristics under 1) surcharge load, 2) vacuum pressure and 3) combination of vacuum pressure and surcharge load. A surcharge load alone induces settlement and outward lateral displacement, while a vacuum pressure induces settlement and inward lateral displacement.

The inward lateral displacement is due to the “suction” of vacuum pressure and it is associated with the consolidation process. This inward lateral displacement will result in change of earth pressure of the improved zone. To predict a vacuum pressure induced ground deformation, understanding the earth pressure acting on the improved zone is important.

Ideally, it is possible to minimize the lateral ground displacement by combining surcharge load with vacuum pressure (Bergado et al. 1998; Tran et al. 2004; Chai et al. 2006). In an urban environment, controlling or minimizing the geotechnical engineering activity induced lateral displacement of a ground is important, sometimes may be a crucial design consideration. However, there is no rigorous solution or laboratory and/or field test results to guide the design. The outward lateral displacement is mainly caused by the shear stress induced by the surcharge load and normally it is an immediate process. Since the mechanisms of the outward lateral displacement due to an embankment load and inward lateral displacements due to vacuum pressure are different, a combination of surcharge load and vacuum pressure may reduce or minimize the lateral displacement of a deposit but may not be able to maintain a zero lateral displacement condition through the whole depth of the deposit.

The ratio of the surcharge load to the vacuum pressure, and rate of surcharge load are key parameter that influencing the lateral displacement (Chai et al. 2013; Ong and Chai 2011; Indraratna et al. 2007).
CHAPTER 3
FIELD STRESS STATE INDUCED BY VACUUM CONSOLIDATION

3.1 Introduction

A vacuum pressure is an isotropic consolidation pressure increment, which induces settlement and inward (towards the centre of improved soil) lateral displacement (Chai et al. 2005a; 2005b; Imai 2005). Due to the inward lateral displacement, the earth pressure on the treated area will be changed. The results of vacuum consolidation under odometer condition show that horizontal effective confinement from the consolidation ring reduced during the vacuum consolidation process, and even causing separation of the soil sample and the ring when the initial vertical effective stress in a soil sample is low (Chai et al. 2005; 2009). However, oedometer condition is different from the field condition. To calculate/predict vacuum consolidation induced ground deformation, understanding the effective stress state in a soil deposit is essential, but there is very limited laboratory or field data on this aspect.

A series of radial drainage vacuum consolidation tests under triaxial stress condition with a drain at the center of the specimen was conducted using the undisturbed Ariake clay samples. The effect of the confining cell pressure on the deformation characteristics of the soil sample was investigated using the laboratory test results. Total of 8 (eight) cases were tested. The laboratory test is described sequentially, as included: (1) the material and sampling site, (2) the equipment, (3) the test method and procedures, and (4) the test results. The test results are: settlement, excess pore pressure and horizontal strain.

A vacuum consolidation project conducted in Saga, Japan (Chai et al. 2006), is briefly described. The laboratory test results are compared with the field measured ones.

Finally a method has been established for predicting the field deformation using laboratory test results. Further possible earth pressure in the deposits under vacuum consolidation has been evaluated.

3.2 Basic consideration

The stress and deformation status of a deposit before and after vacuum consolidation is illustrated by Fig. 3.1. Before treated by vacuum pressure, zone A was
in at-rest condition and there was no lateral displacement occurs. When a vacuum pressure is applied, the treated zone will compress gradually during the consolidation period due to the suction, and an inward lateral displacement will occur. Due to the inward lateral displacement, the at-rest earth pressure \((K_0)\) will be reduced. However, there is no evident yet for understanding how and how much the earth pressure will be reduced due to a vacuum pressure.

![Diagram showing ground improved by vacuum consolidation](image)

**Fig. 3.1 Illustration of ground improved by vacuum consolidation**

3.3 Radial drainage laboratory triaxial consolidation test

3.3.1 Test method

In case of a field vacuum consolidation project, the vacuum pressure will induce inward lateral displacement of the treated soil, which may reduce the effective earth pressure from surrounding soil. To simulate this kind of effect, the tests of reducing the earth pressure at-rest \((K_0)\) to active \((K_a)\) condition were conducted. Stresses applied for \(K_0\) and \(K_a\) confinements are illustrated in Fig. 3.2 (a) and (b), respectively. To simulate the effect of PVD, a mini-drain was inserted at the center of the specimen.
3.3.2 Sampling site and soil properties

The objective of this study is to identify the stress state of a deposit under a vacuum pressure by comparing the field measured and laboratory test results. However, the undisturbed soil sample from the vacuum consolidation project location was not available in this study. Due to the financial and other restrictions, it is not practical to get the undisturbed soil sample from a field vacuum consolidation test site. In Saga Plain the soft clayey deposits is relative uniform. The undisturbed Ariake clay samples nearby the vacuum consolidation site project were available. The locations of the field vacuum consolidation test and the site of the undisturbed Ariake clay samples are illustrated in Fig. 3.3. The soil profile and some properties are shown in Figs. 3.4 and 3.5.
Comparing the soil profile at the vacuum site project (Fig. 3.4) and sample from the sampling site (Fig. 3.5), for soil strata from the ground surface soil down to 11.0 m depth, the water content ($w_n$) was more than 100%, the void ratio was about 3.0, and the values of compression index ($C_c$) were about 1.6 to 2.1 at the vacuum consolidation site and 1.2 to 2.2 at the sampling site.

Fig. 3.4 Soil profile and some index and mechanical properties of sample on vacuum consolidation project (Tanabashi et al. 2004)
### Table 3.1 Ariake clay properties

<table>
<thead>
<tr>
<th>Depth GL -m</th>
<th>Density $\rho_s$ (g/cm³)</th>
<th>Water content $W$ (%)</th>
<th>Void ratio $e$</th>
<th>Degree of saturation $Sr$ (%)</th>
<th>Unconfined compressive strength $q_u$ (kN/m²)</th>
<th>Grain size distribution (%)</th>
<th>Compression Index $C_c$</th>
<th>Soil Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.00-1.85</td>
<td>2.622</td>
<td>128.2</td>
<td>3.45</td>
<td>98.8</td>
<td>76.82</td>
<td>55.3</td>
<td>44.7</td>
<td>Clay</td>
</tr>
<tr>
<td>2.00-2.85</td>
<td>2.622</td>
<td>128.2</td>
<td>3.45</td>
<td>98.8</td>
<td>76.82</td>
<td>55.3</td>
<td>44.7</td>
<td>Clay</td>
</tr>
<tr>
<td>3.00-3.85</td>
<td>2.618</td>
<td>129.7</td>
<td>3.59</td>
<td>98.9</td>
<td>77.40</td>
<td>55.7</td>
<td>44.3</td>
<td>2.227</td>
</tr>
<tr>
<td>4.50-5.35</td>
<td>2.623</td>
<td>127.4</td>
<td>3.38</td>
<td>98.8</td>
<td>76.53</td>
<td>60</td>
<td>40</td>
<td>2.157</td>
</tr>
<tr>
<td>6.50-7.35</td>
<td>2.623</td>
<td>127.4</td>
<td>3.38</td>
<td>98.8</td>
<td>76.53</td>
<td>60</td>
<td>40</td>
<td>2.157</td>
</tr>
<tr>
<td>8.50-9.35</td>
<td>2.615</td>
<td>123.3</td>
<td>3.07</td>
<td>98.9</td>
<td>75.09</td>
<td>61.3</td>
<td>38.7</td>
<td>1.591</td>
</tr>
<tr>
<td>10.00-10.85</td>
<td>2.615</td>
<td>123.3</td>
<td>3.07</td>
<td>98.9</td>
<td>75.09</td>
<td>61.3</td>
<td>38.7</td>
<td>1.591</td>
</tr>
<tr>
<td>11.00-11.85</td>
<td>2.657</td>
<td>99.4</td>
<td>2.74</td>
<td>98.8</td>
<td>66.98</td>
<td>61.5</td>
<td>38.5</td>
<td>1.739</td>
</tr>
<tr>
<td>12.00-12.85</td>
<td>2.705</td>
<td>91.5</td>
<td>2.67</td>
<td>98.6</td>
<td>64.26</td>
<td>53.3</td>
<td>46.7</td>
<td>1.034</td>
</tr>
<tr>
<td>12.90-13.75</td>
<td>2.601</td>
<td>68.8</td>
<td>1.95</td>
<td>97.5</td>
<td>56.08</td>
<td>56.6</td>
<td>43.4</td>
<td>0.959</td>
</tr>
<tr>
<td>15.00-15.60</td>
<td>2.663</td>
<td>21.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20.00-21.00</td>
<td>2.664</td>
<td>31.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>22.00-22.90</td>
<td>2.421</td>
<td>89.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>26.00-27.00</td>
<td>2.42</td>
<td>78.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 3.5 Soil properties of Ariake clay on sampling site
Although there were some discrepancies, generally the two profiles are similar. Therefore, laboratory triaxial vacuum consolidation from the sampling site was conducted.

3.3.3 Equipment

The equipment used in laboratory triaxial consolidation test was modified from an ordinary triaxial compression test device. The sketch and photo of the device are shown in Fig. 3.6 (a) and (b) respectively. There are two main modifications:

1. Instead of using a conventional solid cylindrical specimen, an annular shaped soil specimen with a drain inserted in the centre was used to simulate a unit cell of prefabricated vertical drain (PVD) improved soil layer in the field. The nominal outer diameter of the specimen was 50 mm, inner diameter of 8 mm, and height of 100 mm.
2. Adding a system for applying vacuum pressure to the centre-drain inserted into the specimen.

The whole parts of the equipment are described based on the sketch in Fig. 3.6(a) as follow:

1. Deviation surcharge stress was applied from the top of the device using dead load through the axial rod.
2. Settlement gauge was placed on the top of device for measuring the axial deformation.
3. The air pressure was applied from the top intake to generate the confine pressure through the water in the specimen.
4. A vertical drain made by a steel spring and wrapped by filter paper was inserted into the central hole of the sample as a vertical drain. The spring is made of 0.6 mm in diameter steel wire with a pitch between two adjacent coils of about 2.7 mm. The outer diameter of the spring is about 8 mm. The stiffness of the spring is very low, and its influence on the vertical stress can be ignored.
5. The top pedestal was modified so that a central drain can be connected to the vacuum pressure source.
(a). Sketch of triaxial device

(b) Photo of triaxial device

Fig. 3.6 Laboratory tri-axial test device
A small porous stone disk was inserted into the bottom of pedestal at a distance of about 17 mm from the centre for pore pressures measurements.

All transducers were connected to the data logger and computer. Then the measurements of the settlement and pore water pressure were recorded. Only the radial drainage was allowed during consolidation. After the consolidation test the diameter of the specimen was measured for calculating the horizontal strain.

3.3.4 Cases tested

Table 3.2 Test cases for undisturbed Ariake clay samples

<table>
<thead>
<tr>
<th>Test no</th>
<th>Depth (m)</th>
<th>Initial stress</th>
<th>Reducing loading $\sigma_h$ (kPa)</th>
<th>Final $\sigma_h$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\sigma_{\sigma_0}$</td>
<td>$\sigma_{\sigma_0}$</td>
<td>$\rho_{\sigma_0}$</td>
</tr>
<tr>
<td>T-2Ko</td>
<td>1.5</td>
<td>14.72</td>
<td>8.8</td>
<td>-70</td>
</tr>
<tr>
<td>T-2Ka</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T-4Ko</td>
<td>3.5</td>
<td>24.53</td>
<td>14.7</td>
<td>-70</td>
</tr>
<tr>
<td>T-4Ka</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T-8Ko</td>
<td>7.5</td>
<td>44.15</td>
<td>26.5</td>
<td>-70</td>
</tr>
<tr>
<td>T-8Ka</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T-10Ko</td>
<td>9.5</td>
<td>53.96</td>
<td>32.4</td>
<td>-70</td>
</tr>
<tr>
<td>T-10Ka</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: Before test all samples were released from the 7 mm in diameter of metal tube, cut into 12 cm in thickness, covered by wrapping paper, waxed by paraffin and submerged into the water.

The undisturbed Ariake clay was used. Eight (8) of the test cases are listed in Table 3.2. Both tests with the confining pressure corresponding to the at-rest $(K_0)$ and active $(K_a)$ earth pressure were conducted.

3.3.5 Test Procedures

3.3.5.1 Sample preparation

(1) Specimen preparation

(a) A cylinder soil with 50 mm in diameter and 100 mm in height was prepared by cutting the undisturbed soil sample.

(b) Then the specimen was fixed on a frame (guide) and an 8 mm in diameter
central hole was made using a drilling tool manually.

(2) **Saturating the specimen**

Soil specimen with the centre drain installed was put into a rubber membrane sleeve with a diameter of 50 mm. Then the specimen was placed into a dessicator to which vacuum pressure can be applied, and de-aired water was put around the specimen till the water level reached the height of the specimen. A vacuum pressure of about 20 kPa was applied to the dessicator for about 1 hour to eliminate possible trapped air bubbles between the rubber membrane, the drain, and the soil specimen.

### 3.3.5.2 Pre-consolidation

The specimen was set into the triaxial cell, and a cell pressure of about 5 kPa was applied, and de-aired water was flowed from the bottom to the top of the specimen to remove any air bubbles in the drainage system. Then anisotropic consolidation stresses were applied to consolidate the specimen for 1 day. For the cases tested, a value of the coefficient of at-rest earth pressure, $K_0$, was assumed to be 0.6 (slightly over consolidated soil) in this step. The isotropic part of the consolidation stresses was applied using air-pressure and the deviational part by dead load. To avoid excessive shear deformation in the pre-consolidation process, the pre-consolidation pressures were applied in a stepwise manner with a vertical stress increment of 10 kPa and horizontal stress increment of 6 kPa and a time interval between two loading steps of 2 hours.

### 3.3.5.3 Changing of the confining pressure during consolidation process

Considering that the vacuum pressure induced inward lateral displacement of the deposit will change the effective earth pressure to the vacuum consolidation treated zone, the test of reducing the confining pressure from $K_a$ to $K_a$ condition was conducted. One hour after the start of the test, the confining pressure was gradually reducing from $K_0, \sigma'_v0$ to $K_a, \sigma'_v0$ as illustrated in Fig. 3.2.

The confining pressure was reduced through the water pressure. However the vertical stress was kept unchanged by adding the additional dead load. The consolidation test generally lasted for about 2 to 4 days. The vertical settlement, excess pore water pressure at the bottom of the specimen and at a radial distance, $r_a = 17$ mm
(Fig. 3.6(a)), were measured and recorded through a data logger and a computer. During the pre-consolidation, the amount of water drained out from the specimen was measured and used for calculating the diameter of the specimen before the vacuum consolidation. Further, the final average diameter of the specimen was measured after the consolidation test, and the value is used for calculating the horizontal strain of the specimen caused by the consolidation.

3.3.6 Test results
3.3.6.1 Settlement

The measured settlements of the undisturbed samples that retrieved from 1.5 to 9.5 m depth from the ground surface are shown on Fig. 3.7 to Fig. 3.10. It can be seen that $K_a$ confining condition resulted in larger settlement than that of the corresponding $K_0$ confinement. Reducing the confining pressure from $K_0$ to $K_a$ condition increased the settlement.

![Settlement-time relationship (1.5 m depth)](image)

Fig. 3.7 Settlement-time relationship (1.5 m depth)
Fig. 3.8 Settlement-time relationship (3.5 m depth)

Case
- 3.5mKo T-4Ko
- 3.5mKa T-4Ka

Fig. 3.9 Settlement-time relationship (7.5 m depth)

Case
- 7.5mKo T-8mKo
- 7.5mKa T-8mKa

\[ \sigma_{\text{vac}} = -70 \text{ kPa} \]
\[ \sigma'_{\text{v0}} = 44.15 \text{ kPa} \]
Under a $K_a$ condition, deviation stress in the sample was increased because the vertical stress ($\sigma'_v$) unchanged and the horizontal stress ($\sigma'_h$) was decreased, and result in a larger shear deformation. The results in these four figures indicate that the deeper the soil sample located, the smaller the distinction of a measured settlement between $K_a$ and $K_0$ confinement. The reason is at a deeper location, the soil sample is denser and less compressible.

### 3.3.6.2 Excess pore pressure

The variation of measured excess pore pressure ($u$) induced by vacuum pressure for soil samples of 1.5 and 7.5 m depth are shown in Figs. 3.11 and 3.12. For the case of $K_a$ confinement condition, values of $u$ are higher (less negative value) than that of the $K_0$ confinement condition. Reducing confining pressure can induce additional shear stress in the specimen, and it induced some more pore pressure increment. The applied vacuum pressure was -70 kPa, but the final measured values were about -44 to -48 kPa for 1.5 m depth (Fig. 3.11) and -35 to -40 for 7.5 m depth samples (Fig. 3.12). The possible reason considered is that under the laboratory condition, with a vacuum
Fig. 3.11 Variation of pore water pressure (1.5 m depth)

Fig. 3.12 Variation of pore water pressure (7.5 m depth)
A pressure of about -70 kPa, the soil adjacent to the mini-drain in the centre of the specimen can become unsaturated, and in the unsaturated zone, convex (toward the drain) meniscuses will be formed, which can reduce the efficiency of the vacuum pressure transferring into the soil sample. The deeper the soil sample, the denser the soil and lower the permeability, therefore the less propagated of the vacuum pressure in to the soil.

3.3.6.3 Horizontal strain

(1) Introduction

The diameter of specimen was measured before and after the consolidation test. The measurement was taken with following steps:

(a) Before installation of the specimen, the initial shape was measured. The measurements include: initial height \( h_0 \) and initial diameter \( d_0 \).

(b) During the pre-consolidation, the settlement \( s_{pc} \) and the amount of water drained out \( \Delta v \) were measured.

Since the specimen was full saturated, then the volume change \( v_{pc} \) of a specimen at the end of pre-consolidation can be calculated. Once the settlement and soil volume at the end of pre-consolidation are known, then the diameter after pre-consolidation \( d_{pc} \) can be calculated. Next, the \( d_{pc} \) is considered as the initial diameter before consolidation. After the consolidation test, the final diameter of the specimen was measured. Thus, the horizontal strain after test can be calculated.

Sketch of final shape of the soil cylinder after vacuum consolidation are shown on Fig. 3.13. The \( K_0 \) confinement condition resulted in smaller settlement than \( K_a \) confinement condition. Therefore, the outward lateral displacement was larger for \( K_a \) confinement condition. All measured horizontal strains are presented on Fig. 3.14. It is clear that \( K_a \) confinement condition resulted in larger outward lateral displacement than that of \( K_0 \) confinement condition.
Fig. 3.13 Shape of the specimen at end of consolidation (1.5 and 3.5 m depth)

Note: Diameter of the specimen was measured at every 5 mm height and on two perpendicular directions, and average value was used in calculating the horizontal strain.

Fig. 3.14 Horizontal Strain – depth relationship under $K_0$ and $K_a$ condition
3.3.6.4 Stress ratio (K) of Triaxial test

For an ordinary triaxial test, the total confining pressure is a constant, but there is no restriction on horizontal displacement. Thus if the horizontal effective stress (σ′_h) is less than the stress (K_0σ′_v) required to maintain a no horizontal deformation (K_0) condition state, there will be outward lateral deformation. Further, if σ′_h > K_0σ′_v, there will be inward lateral displacement.

A stress ratio (K) is defined as the ratio of horizontal effective stress (σ′_h) to the value in the vertical direction (σ′_v). Therefore, the tendency of lateral displacement can be easily judged using the stress ratio (K) (Eq. (3.1)).

\[
K = \frac{\sigma'_h}{\sigma'_v}
\]  

(3.1)

Figs. 3.15 and 3.16 show the typical time – K relationships. The K_0 confinement condition resulted in higher stress ratio (K) than that of the K_a confinement condition.

![Fig. 3.15 Time – K variation (1.5 m depth)](image)
3.3.6.5 $K - \varepsilon_h$ relationship

The stress ratio, $K$, versus horizontal strain, $\varepsilon_h$, for the undisturbed sample is shown in Fig. 3.17. The value of $K$ is corresponding to the end of consolidation test. The data points of sample are scatter, and it may be due to the spatial variation of soil properties of the sample. There are close relationships between $K$ and $\varepsilon_h$. It is considered that when $K = K_0$ will result in minimum lateral displacement. For the Ariake clay, a $K_0$ value of 0.41 (Chai and Kawaguchi 2011) is adopted. In Fig. 3.17, all values of $K$ are larger than 0.41.
3.3.7 Summary and comments

Investigating on the deformation behaviour and stress state of soft clay deposits under vacuum pressure had been carried out through radial drainage triaxial laboratory consolidation test, using the undisturbed Ariake clays. The total of 8 (eight) cases were conducted.

An ordinary triaxial apparatus was modified to allow the vacuum pressures application. Only radial drainage was allowed during the consolidation process. The measurements were carried on settlement, pore pressure and the lateral displacement (horizontal strain) at the end of the consolidation.

The purpose of this test is to evaluate the effect of confining condition on deformation induced by vacuum pressure. From the result, it is found that reducing the confining pressure will reduce the inward lateral displacement and increase the settlement.
3.4 Vacuum consolidation project in Saga site

A vacuum consolidation with the duration of 9 months (June 2003- March 2004) was carried out at a site in Saga Japan (Chai et al. 2006). The total improved area was 1 km and 16 - 18 m wide and divided into 7 sections (Fig. 3.18). The section 4 was instrumented with a settlement gauge, inclinometer, and pore pressure gauges. It had a length of 187 m. The ground water level at the site was 0.5 - 1.5 m below the ground surface (0.7 m was assumed in subsequent analysis).

![Fig. 3.18 Location of test sections (after Chai et al. 2006)](image)

A sand mat about 1.0 m thick was spread on the ground surface as a drainage layer. Then a 0.5 mm thick polyvinyl chloride (PVC) membrane was placed as air sealing sheet. PVD were installed from the original ground surface down to 10.5 m depth in square pattern at 0.8 m spacing. A prefabricated drain (0.3 m wide and 4.5 m thick) were laid on top of sand mat with 0.8 m horizontal spacing. To avoid air leakage through the top of the unsaturated zone, the edges of the sealing sheet were embedded in a 1.5 m depth trench. During the sand mat construction the settlement was not monitored and the instruments were installed after the sand mat construction. It was assumed that the settlement was finished before commenced the vacuum pressure. Just before the commencement of vacuum consolidation, the measured settlement at S-1 was 112 mm.
Vacuum consolidation was started on November 2003 for section 4. The vacuum pressure measured at the ground surface was around -60 to -70 kPa (Fig. 3.20), and the duration was 120 days. However, at the piezometer locations (in the soils between PVDs) of P-1-1, P-1-2 and P-1-3, the measured excess pore pressure at the time of stopping the vacuum pump were about -35 to -50 kPa as shown in Fig. 3.21.

Fig. 3.19 Plan layout of instrumentation points

Fig. 3.20 Measured vacuum pressure under the air tightening sheet
The measured settlements from the starting of the vacuum consolidation are shown in Fig. 3.22. At the time of stopping the vacuum pump, the measured ground surface settlement was about 0.86 m.
3.5 Deformation conditions in laboratory test and field case

There are very limited reports about comparing vacuum consolidation on laboratory test and field cases. In this study, the laboratory consolidation was tested under triaxial stress condition, while in the field case the vacuum consolidation was close to plane strain condition. Based on the elastic theory with the isotropic consolidation such as vacuum pressure increment, the vertical and horizontal strains induced by triaxial and plane strain conditions are different. However, for a normally consolidated clayey soil the deformation is mainly plastic than elastic. The direction of plastic deformation is controlled by the direction of effective principle stress rather than the stress increment. Vaid and Campanella (1974) reported some laboratory test results comparing the deformation behavior of a clayey under triaxial and plane strain conditions. Under drained compression, the volumetric strains are almost the same for both triaxial and plane strain conditions, while under drained extension, plane strain condition resulted in expansion but triaxial condition resulted in contraction. Sun et al. (2011) reported numerical results for an assumed over-consolidated clay. The volumetric strain under the plane strain condition is between the volumetric strains of the triaxial compression and extension conditions. Since the volumetric strain of a soil is not only induced by the increment of mean effective stress but also the change of deviator (shear) stress, and the shear deformation induced volumetric strain depends on the dilatancy behavior of the soil and there may not be a simple conclusion on this issue. Chai et al. (2005) assumed that for both a plane strain and a triaxial deformation condition, a vacuum pressure induced volumetric strain is the same as that of one-dimensional consolidation with the same vertical consolidation pressure as the vacuum pressure. The vertical and horizontal strains under a plane strain condition are larger than that of a triaxial condition.

For the laboratory consolidation test under triaxial condition, the pre-consolidation vertical stresses (initial effective vertical stresses) applied to the soil specimens before applying the vacuum pressure were evaluated using gravity force of natural soil deposit. The effect of the sand-mat was not considered. Therefore, the adopted pre-consolidation stresses may be lower than the actual field values. And as a result, the measured deformation of the specimens may be larger than the one that considered the effect of the sand-mat. As a tendency, this kind of larger deformation
can somehow compensate the smaller vertical and horizontal strains from the triaxial test condition in the laboratory comparing to that of a closer to plane strain condition in the field. To be simple, the laboratory measured strains are directly compared with the field values as described in the following section.

3.6 Predicting field deformation by laboratory test results

3.6.1 Matching the field and laboratory consolidation times

Using laboratory consolidation curve to estimate the field compression curve of the corresponding soil layer, due to the different geometric conditions, a matching between the laboratory and the field consolidation times is required. By considering that field vacuum consolidation projects usually involving installation of prefabricated vertical drains (PVDs) into the deposit, the Hansbo’s (1981) solution for PVD induced consolidation has been used to derive a match equation. In the Hansbo’s (1981) solution, the average degree of the consolidation \( U_h \) of a unit cell (a PVD and its improvement area) is calculated as in Eq. (3.2),

\[
U_h = 1 - \exp(-8T_h / \mu)
\]  

where

\[
T_h = \frac{c_h t}{D_e^2}
\]

\[
\mu = \ln(D_e / d_s) + (k_h / k_s) \ln(d_s / d_w) - \frac{3}{4} + \frac{2\pi \cdot l^2 \cdot k_h}{3q_w}
\]

where \( c_h \) = the coefficient of consolidation in the horizontal direction of a deposit, \( t \) = time, \( D_e \) = the diameter of a unit cell, \( d_s \) = the diameter of smear zone, \( d_w \) = the equivalent diameter of PVD, \( l \) = drainage length of a PVD, \( k_h, k_s \) = horizontal hydraulic conductivities of natural deposit and the smear zone, respectively, and \( q_w \) = discharge capacity of a PVD. Let’s define the times, \( \mu \) values of laboratory and the field consolidation as \( t_L \) and \( t_F \); \( \mu_L \) and \( \mu_F \), respectively. Then, under the same degree of consolidation, the corresponding field time can be computed by Eq. (3.5),
\[ t_F = \frac{D_e^2 \mu_F}{D_l^2 \mu_L} t_L \] (3.5)

where: \( D_L \) = the diameter of the soil sample used in the laboratory triaxial consolidation test. \( \mu_L \) and \( \mu_F \) can be evaluated by Eq. (3.4) with corresponding parameters.

### 3.6.2 Parameters

The parameter related to geometric of the PVD used for consolidation analysis for laboratory test and field case are listed in Table 3.3. The values for the field case are from Chai et al. (2006), and the values for the laboratory condition are modified from Chai and Rondonuwu (2015). Under the laboratory condition, a thin soil zone adjacent to the drain might become unsaturated and the value of \( k_s \) reduced significantly. For the laboratory odometer test, Chai and Rondonuwu (2015) suggested \( d_s = 2d_w \), but for the laboratory triaxial test reported in this study, \( d_s = 3d_w \), gives a better match by the laboratory test results to the field measured settlement curves. With the values of the parameters in Table 3.3 are used and ignoring the well resistance, Eq. (3.5) resulted in \( t_F = 550t_L \), i.e. multiplying the laboratory time by a factor of 550, the laboratory strain versus time curve can be used for the filed condition. Reversely, the laboratory time will be 1/550 of the field time.

<table>
<thead>
<tr>
<th></th>
<th>( d_w ) (mm)</th>
<th>( d_s ) (m)</th>
<th>( D_e ) or ( D_L ) (m)</th>
<th>( k_s/k_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Field</td>
<td>52.3</td>
<td>0.3</td>
<td>0.9</td>
<td>10</td>
</tr>
<tr>
<td>Laboratory</td>
<td>8.0</td>
<td>0.024</td>
<td>0.05</td>
<td>10</td>
</tr>
</tbody>
</table>

For the field vacuum consolidation project considered in this study, the vacuum pressure was applied for about 102 days, and the corresponding laboratory time will be about 4.5 hours. The undisturbed soil samples were only available at 1.5 m; 3.5 m; 7.5 m and 9.5 m depths as indicated in Fig. 3.23 by open circles.
3.6.3 Comparison of vertical and horizontal strains

Fig. 3.24 shows the comparison of the vertical strains. The field measured strains have been calculated as the final measured compression of the soil layer divided by the thickness of the soil layer, and it was assumed that the compression of the sand-mat and the middle sand layer (4.0 – 4.5 m depth from the ground surface) can be ignored. The laboratory values are corresponding to an elapsed time of 4.5 hours (corresponding to the end of vacuum pressure application in the field). In Fig. 3.24, the depth is referred from the original ground surface. Generally, the field vertical strains are larger than that of the laboratory values. There are some possible reasons, i.e. the evaluated field strains may include the certain compressions of the sand-mat and the sandy silt soil layer; and the soil specimens for the laboratory tests were not retrieved at the site of the field test; etc. Nevertheless, they are comparable, and the direct comparison clearly shows that the laboratory values with $K_a$ confinement are closer to the field vertical strains. For the soil samples at 9.5 m depth, the vertical strain of $K_a$ confinement was slightly larger than that of $K_0$ confinement, and it may be due to the non-uniformity of the undisturbed soil samples used for the laboratory tests.
The comparison of the horizontal strains is shown in Fig. 3.25. The field horizontal strains have been calculated using the measured lateral displacement at the edge of the vacuum pressure treated area along the longer side and the half width (8.8 m) of the treated area. The laboratory values are corresponding to the final values after the tests were dismantled. Therefore, the laboratory values may be larger than that at the laboratory elapsed time of about 4.5 hours. While in case of compressive horizontal strain (most cases of the laboratory tests), after dismantling, the specimen would swell and in the direction of reducing compressive strain. Therefore, it is believed that the laboratory values are still comparable with the field values. Although the data are scattered, generally, laboratory $K_0$ confinement resulted in more inward compression than that of the $K_a$ confinement. For the zone 0 – 5 m depth from the ground surface, the measured data are close to the value from the laboratory $K_a$ confinement, and at deeper location, the measured values are between the values from the laboratory $K_a$ and $K_0$ confinements.
The results in Figs 3.24 and 3.25 show that the vacuum pressure induced inward lateral displacement had an influence on the gravity force induced earth pressure in the ground (cell pressure in the laboratory condition). Although there are some inconsistency on the general tendencies revealed from Figs 3.24 and 3.25, to give more credit to the horizontal strains, it is suggested that under a vacuum pressure, generally at shallow depths (< 5 m), the gravity force induced horizontal earth pressure is close to active earth pressures, and at deeper locations, the gravity force induced horizontal earth pressures are between active and at-rest earth pressures.

### 3.6.4 Predicting field compression curves

The proposed thicknesses of soil layer to be represented by each undisturbed sample are shown in Fig. 3.23. Then, the field compression curves can be constructed using the laboratory measured strains by multiplying them with appropriate thicknesses of soil layer.
For example, the compression curve of the soil layer from the ground surface to 4.1 m depth can be calculated by multiplying the laboratory measured strains of the soil specimens from 1.5 m and 3.5 m depths with the thicknesses of 2.5 m and 1.5 m respectively, and then added the resulting values together which will be the value of the compression of the soil layer at a given time. The calculated curves are compared with the field measured ones in Figs. 3.26 and 3.27.
From both Figs. 3.26 and 3.27, it can be seen that, generally the calculated curves using the laboratory test results are comparable with the field measured ones. This is the same with the comparison of vertical and horizontal strains shown in Figs. 3.24 and 3.25. For the layer 0 - 4.1 m depth from the ground surface, the calculated curve using the laboratory results of $K_a$ confinement is closer to the field measured one, while for the layer from 4.1 m - 10.3 m depth, it is not obvious which results from which confinement condition is better. And somehow both of the curves are comparable with the measured one.

When observing the measured and the calculated curves in detail, it can be seen that at the earlier stage, less than about 50 days (field time), the calculated settlement rates (for 0 - 4.1 m depth, the results from $K_a$ confinement) are higher than that of the measured ones. While for the elapsed time larger than about 50 days, the field settlement rates are higher. At this site there is a sandy silt layer at 4.0 - 4.5 m depth and there might be vacuum leakage through this sandy silt layer. Initially, this kind of leakage might be significant and as a result reduced the settlement rate compared to that with no leakage. With the progress of consolidation, some fine particles might be sucked into the void of the geotextile filter of the PVDs, which caused clogging of the filter and reduced the amount of leakage of vacuum pressure through the sandy silt layer, and resulting in faster vacuum pressure increase in the soil layers (Fig. 3.21) and therefore, faster settlement rates (Figs. 3.26 and 3.27). Another point is that at the time of terminating vacuum pressure, the measured compressions of soil strata are larger than that of calculated using the laboratory test results. There are several possible reasons for this. One is that the final achieved (reached) vacuum pressure in the laboratory and in the field are not exactly the same. And the others are the soil samples used in laboratory test are not retrieved at the exact site where the field vacuum consolidation test was conducted, and in the calculation, the compressions of the sand-mat and the middle thin sandy silt layer are ignored, etc. Nevertheless, the differences are small, and they are not altering the general trend identified from this study.
3.7 Summary and Comments

The radial drainage laboratory triaxial consolidation test has been conducted under vacuum pressure on undisturbed Ariake clay. By reducing the effective confinement on the specimen, it is found that $K_0$ (at-rest) confinement resulted in more outward lateral displacement than a $K_a$ (active earth pressure) confinement. The results were used to predict the deformation under field condition. For a prefabricated vertical drain (PVD) improved deposit, a theoretical equation has been derived to convert consolidation time in laboratory to the field case. Using the equation and the results of laboratory vacuum consolidation test under triaxial condition, the compression versus time curves of the soil strata at the site of the field vacuum consolidation project considered in this study have been constructed and reasonably well with compared the field measured results. This indicates that a vacuum pressure induced deformations in the field can be predicted using the results of laboratory vacuum consolidation test under triaxial condition with proper confinements.

Comparing the laboratory measured vertical and horizontal strains with the deduced ones from the field measured data indicates that for a field vacuum consolidation, the effective confinement due to gravity force from the surrounding soils to the vacuum treated area is close to the value of active earth pressure for a zone about 5 m depth from the ground surface, and below it, the effective confinement due to gravity force is between active and at-rest states.
CHAPTER 4
SURCHARGE LOADING RATE FOR MINIMIZING LATERAL DISPLACEMENT WITH VACUUM PRESSURE

4.1 Introduction

4.1.1 Background

Preloading clayey deposits improved by prefabricated vertical drain (PVD) with combination of vacuum pressure and surcharge load has been widely used in engineering practice. The combined load offers several advantages and one of them is, it can reduce preloading induced lateral displacement of a deposit (e.g., Bergado et al. 1998; Yan and Chu 2005; Kelly and Wong 2009; Indraratna et al. 2011; Long et al. 2013; Chai et al. 2013a; Xu and Chai 2014). Vacuum pressure is an isotropic incremental consolidation pressure and it will result in settlement and inward lateral displacement (toward the centre of the loading area). This inward lateral displacement sometimes may cause surface cracks around the treatment area (Shang et al. 1998; Chu et al. 2000; Chai et al. 2005a). While a surcharge (embankment) load will induce settlement and outside lateral displacement of the ground. Controlling and minimizing lateral displacement in the ground is an important issue for many geotechnical engineering projects. Therefore combining a vacuum pressure with a surcharge load may be a good option for minimizing lateral displacement during preloading of a soft clayey deposit. Based on the radial drainage laboratory consolidation tests (oedometer and triaxial) under vacuum pressure and surcharge load, a method for designing the surcharge loading rate to minimize the lateral displacement of a deposit has been proposed.

The factors influencing lateral displacement of a deposit induced by embankment and/or vacuum loadings are: (1) the magnitude and loading rate of the loads; (2) the ratio \( LR \) of a surcharge load \( \Delta \sigma_s \) to a vacuum pressure \( \Delta \sigma_{vac} \) \( LR = \Delta \sigma_s / \Delta \sigma_{vac} \); (3) the initial stress state in and the undrained shear strength \( s_u \) of the subsoil; and (4) the consolidation and deformation characteristics of the subsoil. Ong and Chai (2011) reported that for a given subsoil condition, \( LR \) and surcharge loading rate \( SLR \) are two main influence factors on the lateral displacement of a deposit under combined loading. The smaller the \( SLR \) and \( LR \) values, the smaller the outward lateral
displacement. For applying the preloading with combined loads to road or railway embankment constructions, in most cases, the achievable magnitude of vacuum pressure is limited, and the final height of an embankment is pre-specified, i.e. $LR$ may be decided prior to construction. While a value of $SLR$ can be controlled during the construction process, to minimize the lateral displacement. However, there has been no such design method available yet.

In this study a series of laboratory consolidation tests under both modified odometer and triaxial tests with radial drainage (drain was located at the center of specimens) were conducted to investigate the tendency of lateral displacement of the samples under the combination of a vacuum pressure and a surcharge load. Based on the test results, a method has been proposed to determine the optimum $SLR$ with which the lateral displacement of a deposit can be minimized in the field condition.

### 4.1.2 Basic consideration

For the field condition involving the combination of surcharge load and vacuum pressure, a stress ratio ($K$) is defined as the ratio of horizontal effective stress ($\sigma'_h$) to the value in the vertical direction ($\sigma'_v$) as in equation (3.1),

$$K = \frac{\sigma'_h}{\sigma'_v}$$  \hspace{1cm} (3.1bis)

When $K > K_0$ ($K_0$ is the coefficient of at–rest earth pressure) a soil sample will be compressed, and when $K < K_0$, it may expand in horizontal direction.

![Fig. 4.1 Illustration of the pressure change on the wall of a consolidation ring](after Chai et al. 2013b)
Under oedometer condition, outward lateral displacement is restricted and only inward lateral displacement is allowed (Fig. 4.1). While horizontal effective stress is changeable. To judge the lateral displacement tendency of a specimen, Chai et al. (2013b) proposed a parameter called coefficient of earth pressure on the wall of the consolidation ring ($K_w$). For detail, it will be explained in the next section.

For an ordinary triaxial test, the total confining pressure is a constant, but there is no restriction on horizontal displacement.

4.2  Radial drainage laboratory oedometer consolidation test

4.2.1  Sampling site and soil properties

Remolded Ariake clay was used for the oedometer tests. The remolded sample was excavated at about 2 m depth below ground surface from Ashikari site, Ogi town, Saga Prefecture Kyushu, Japan (Fig. 4.2). Majority of Ariake clay is marine formation with smectite as main clay mineral (Ohtsubo et al. 1995; Tanaka et.al 2001). Natural Ariake clay deposit has a high compressibility and low strength (Miura et al. 1998).

![Fig. 4.2 Sampling site location](image)
Table 4.1 Basic soil properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Plastic limit</th>
<th>Liquid limit</th>
<th>Compression index</th>
<th>Coefficient of consolidation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$w_p$ (%)</td>
<td>$w_l$ (%)</td>
<td>$C_c$</td>
<td>Vertical, $c_v$</td>
</tr>
<tr>
<td>Value</td>
<td>56.8</td>
<td>120.3</td>
<td>0.821</td>
<td>$5 \times 10^{-3}$</td>
</tr>
</tbody>
</table>

Some physical and mechanical properties of the remolded Ariake clay tested are listed in Table 4.1. To determine its consolidation properties both incremental loading (IL) and constant rate of strain (CRS) consolidation tests with a strain rate of 0.02%/min were conducted. The CRS tests were conducted with vertical and horizontal drainages. A ratio of the coefficients of consolidation in the horizontal direction ($c_h$) to the vertical direction ($c_v$) is 1.63.

### 4.2.2 Equipment

To simulate the prefabricated vertical drain (PVD) induced consolidation in the field, the radial drainage oedometer tests were conducted in this study. The equipment used in the laboratory study was modified from a Maruto Multiple Oedometer Apparatus (manufactured in Tokyo, Japan). The sketch and photo of oedometer device used are shown in Fig. 4.3 (a) and (b), respectively. It consists of oedometer ring 60 mm in diameter and 20 mm in height. At the middle height of the wall of the ring a pore pressure transducer and an earth pressure transducer are inserted into the wall and opposite to each other. A cylinder metal porous stone is inserted in the middle of the specimen as a vertical drain. The diameter of the drain is 4 mm or 8 mm. The vacuum pressure is applied to the specimen through the vertical drain and the surcharge load is applied using air pressure. The settlement gauge is installed on the top of the loading piston. Settlement, pore water pressure and lateral total earth pressure are recorded by a data logger. The specimen is 60 mm in diameter and 20 mm in thickness.
Fig. 4.3 Laboratory oedometer test device
4.2.3 Cases tested

Total of twenty (20) cases were tested, as listed in Table 4.2. The tests were conducted with variation of $LR$ and $SLR$. For all cases, the applied vacuum pressure was -80 kPa and total surcharge load of 80 kPa, but the $SLR$ was different. The $SLR$ was varied from 10 kPa/360 min to 10 kPa/10 min. The value of $LR$ was ranged from 1 to 2.

Table 4.2 Cases tested using the modified oedometer

<table>
<thead>
<tr>
<th>Case</th>
<th>$\sigma'_{v0}$ (kPa)</th>
<th>Drain diameter ($d_w$) (mm)</th>
<th>$\Delta\sigma_i$ (kPa)</th>
<th>$\Delta\sigma_{vac}$ (kPa)</th>
<th>$LR$</th>
<th>$SLR$ (kPa/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>O-1a</td>
<td>0</td>
<td>8</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/10</td>
</tr>
<tr>
<td>O-1b</td>
<td>0</td>
<td>8</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/60</td>
</tr>
<tr>
<td>O-1c</td>
<td>0</td>
<td>8</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/120</td>
</tr>
<tr>
<td>O-1d</td>
<td>0</td>
<td>8</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/360</td>
</tr>
<tr>
<td>O-2a</td>
<td>0</td>
<td>8</td>
<td>80</td>
<td>-40</td>
<td>2</td>
<td>10/60</td>
</tr>
<tr>
<td>O-2b</td>
<td>0</td>
<td>8</td>
<td>80</td>
<td>-40</td>
<td>2</td>
<td>10/90</td>
</tr>
<tr>
<td>O-2c</td>
<td>0</td>
<td>8</td>
<td>80</td>
<td>-40</td>
<td>2</td>
<td>10/360</td>
</tr>
<tr>
<td>O-3a</td>
<td>0</td>
<td>4</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/10</td>
</tr>
<tr>
<td>O-3b</td>
<td>0</td>
<td>4</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/15</td>
</tr>
<tr>
<td>O-3c</td>
<td>0</td>
<td>4</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/90</td>
</tr>
<tr>
<td>O-3d</td>
<td>0</td>
<td>4</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/360</td>
</tr>
<tr>
<td>O-4a</td>
<td>20</td>
<td>8</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/30</td>
</tr>
<tr>
<td>O-4b</td>
<td>20</td>
<td>8</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/60</td>
</tr>
<tr>
<td>O-4c</td>
<td>20</td>
<td>8</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/90</td>
</tr>
<tr>
<td>O-5a</td>
<td>40</td>
<td>8</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/30</td>
</tr>
<tr>
<td>O-5b</td>
<td>40</td>
<td>8</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/60</td>
</tr>
<tr>
<td>O-5c</td>
<td>40</td>
<td>8</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/90</td>
</tr>
<tr>
<td>O-6a</td>
<td>80</td>
<td>8</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/30</td>
</tr>
<tr>
<td>O-6b</td>
<td>80</td>
<td>8</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/60</td>
</tr>
<tr>
<td>O-6c</td>
<td>80</td>
<td>8</td>
<td>80</td>
<td>-80</td>
<td>1</td>
<td>10/90</td>
</tr>
</tbody>
</table>

$LR$ means loading ratio is defined as surcharge load divided by the vacuum pressure, and $SLR$ means surcharge loading rate.
4.2.4 Test Procedures

4.2.4.1 Sample preparation

The excavated soil sample was mixed thoroughly, by using an electric hand mixer, then removing the shell skeleton by using 420 μm in diameter sieve (JIS Z8801). Next, the water content of the sample was adjusted to higher than its liquid limit by adding tap water, then the slurry sample was de-aired under approximately 100 kPa of vacuum pressure.

4.2.4.2 Pre-consolidation

The de-aired slurry was put into a consolidation container of 60 mm in diameter and 60 mm in height (20 mm height of consolidation ring plus a 40 mm height collar), and consolidated under 20 kPa pressure for 24 hours. Next 20 mm thick specimen was cut from pre-consolidated sample for further consolidation test.

![Cylindrical device for making hole at the centre of specimen](image)

**Fig. 4.4 Cylindrical device for making hole at the centre of specimen**

4.2.4.3 Consolidation test

(1) **Set-up of the consolidation tests**

For the radial drainage oedometer test, an 8 mm or 4 mm in diameter hole was formed at the centre of the specimen to allow installation of an annular porous stone, which acted as a central drain. The hole was made by a cylindrical device, which is
shown in Fig. 4.4. The inner diameter of the cylindrical device is the same as the outer diameter of the consolidation ring, and the tube at the centre of the device has an outer diameter of 8 mm or 4 mm. The device was set in such a way that the consolidation ring with the soil specimen inside it could be placed in the cylindrical device, and then the centre tube was pushed into the soil specimen. The soil was extracted from the central tube of the device, and forming a central hole through the sample. Then the drain was inserted through the central hole.

(2) Loading procedure and test duration

To investigate the effect of the initial vertical effective stress ($\sigma_{v0}'$) in the specimen on the deformation behaviour of the specimen during subsequent consolidation involving application of a vacuum pressure, each specimen was first consolidated under a predetermined surcharge load in the range: $\sigma_{v0}' = 0 \sim 80$ kPa. An incremental consolidation pressure was then applied by combining a surcharge load ($\Delta \sigma_s$) and a vacuum pressure ($\Delta \sigma_{vac}$). For all tests, the vacuum pressure was fully applied at the start of the test, but the surcharge load was applied in a stepwise manner (Fig.4.5).

![Fig. 4.5 Illustration of applied the combination of surcharge load and vacuum pressure](image)
4.2.5 Test results

4.2.5.1 Introduction

Under combined surcharge load and vacuum pressure, the effects of loading condition (i.e. surcharge loading rate (SLR) and the ratio between surcharge loading to a vacuum pressure (LR)) and initial effective stress in a sample to the deformation characteristics of a soil sample were investigated using oedometer consolidation tests. The results are presented viz. measured settlement, excess pore pressure and lateral earth pressure.

Further, the results were analysed to identify the condition, under which the combined loads will induce less lateral deformation in the field condition.

4.2.5.2 Settlement

Typical time-settlement curves from oedometer test under combined surcharge load and vacuum pressure are shown in Fig. 4.6 to Fig. 4.9.

(1) **Effect of loading condition (SLR and LR)**

Fig. 4.6 shows the settlement result for the LR = 1.0 (\(\sigma'_s = 80\) kPa and \(\sigma'_\text{vac} = 80\) kPa) cases, the value of SLR were 10/360, 10/60 and 10/10 kPa/min. Increase the SLR increased the rate of settlement. There is no significant different at the final settlement.

(2) **Effect of drain size (\(d_w\))**

The measured settlements of \(d_w = 8\) mm and \(4\) mm are shown in Fig. 4.6 and Fig. 4.7, respectively. It can be seen that under \(d_w = 8\) mm result faster rate of settlement than the \(d_w = 4\) mm. The larger the diameter of the drain, the faster the pore water drain out, therefore the larger the rate of settlement.

(3) **Effect of initial effective stress (\(\sigma'_v0\))**

The measured settlement induced by surcharge load and vacuum pressure at a different initial effective stress (\(\sigma'_v0\)) are compared in Fig. 4.8 and Fig. 4.9. The smaller the initial effective stress the larger the settlement.
Fig. 4.6 Time-settlement curves ($LR = 1.0; d_w = 8 \text{ mm}$)

Fig. 4.7 Time-settlement curves ($LR = 1.0; d_w = 4 \text{ mm}$)
Fig. 4.8 Time-settlement curves ($SLR = 10$ kPa/30 min)

Fig. 4.9 Time-settlement curves ($SLR = 10$ kPa/60 min)
4.2.5.3 Lateral earth pressure \((LEP)\) increment

(1) Effect of surcharge loading rate \((SLR)\)

The variation of lateral earth pressures \((LEP)\) increment acting on the soil specimen with the different \(SLR\) are presented on Fig. 4.10 to Fig. 4.14. The \(LEP\) at any given time tended to decrease during the consolidation period. However when the incremented load was applied the \(LEP\) was increased. The larger the \(SLR\), the larger the \(LEP\) during the surcharge loading application was. The negative total earth pressure indicated possible formation of micro-gap between the consolidation ring and the soil specimen. Effect of loading ratio \((LR)\) to \(LEP\) is shown in Fig. 4.10 and Fig. 4.11. The vacuum pressure applied for \(LR= 1.0\) and \(LR= 2.0\) were \(-80\) kPa and \(-40\) kPa, respectively, therefore at the end of consolidation for \(LR= 1.0\) \((\sigma'_{vac} = -80\) kPa\) result in less (maximum in negative value) \(LEP\) than for \(LR= 2.0\) \((\sigma'_{vac} = -40\) kPa\).

\[\text{Fig. 4.10 Effect of surcharge loading rate to the total lateral earth pressure} \quad (LR = 1.0; \; d_w = 8 \text{ mm})\]
Fig. 4.11 Effect of surcharge loading rate to total lateral earth pressure

\( (LR = 2.0; \ d_w = 8 \text{ mm}) \)

(2) Effect of initial effective stress (\( \sigma'_{v0} \))

The measured lateral earth pressures (LEP) increment at different initial effective stress (\( \sigma'_{v0} \)) shown in Fig. 4.12. Fig. 4.13 and Fig. 4.14 for \( \sigma'_{v0} = 20, 40 \) and 80 kPa, respectively. It can be seen that a different of LEP between two values of SLR is significant for smaller initial effective stress than the larger one. In Fig. 4.15 and Fig. 4.16, it can be seen that the smaller \( \sigma'_{v0} \) the larger the measured LEP increment. For the deviation of the LEP (maximum to the minimum) at any given \( \sigma'_{v0} \) during the surcharge load application shows that the smaller the \( \sigma'_{v0} \) the larger the deviation of LEP (Fig. 4.15 and Fig. 4.16). The larger the initial effective stress means the denser the soil specimen. Under the same load increment, the denser a sample is the smaller the amount of deformation will be.
Fig. 4.12 Variation of total lateral earth pressure ($\sigma'_{v0} = 20$ kPa)

Fig. 4.13 Variation of total lateral earth pressure ($\sigma'_{v0} = 40$ kPa)
Fig. 4.14 Variation of total lateral earth pressure ($\sigma_v^\prime = 80\text{kPa}$)

Fig. 4.15 Variation of total lateral earth pressure ($SLR = 10\text{kPa}/30\text{ min}$)
Fig. 4.16 Variation of total lateral earth pressure ($SLR = 10$ kPa/60 min)

The measured $LEP$ with the similar $SLR$ but different $\sigma'_{v0}$ are shown in Fig. 4.15 and Fig. 4.16. It can be seen that reducing the $\sigma'_{v0}$ increased the $LEP$.

4.2.5.4 Excess pore water pressure

(1) Effect of surcharge loading rate ($SLR$)

The variation of excess pore water pressure induced by combination of vacuum pressure and surcharge load under oedometer radial consolidation test with different value of $SLR$ are shown on Fig. 4.17 to Fig. 4.19. During the consolidation period, the excess pore pressure tends to decrease as the effective stress increased. However when the surcharge load was increased the excess pore pressure increased as well. The maximum pore pressure at the end of the surcharge load application was obtained at the larger $SLR$. The smaller the $SLR$, the smaller the excess pore pressure will be. At the different $LR$ (Fig. 4.17 and Fig. 4.18) the maximum excess pore pressure during surcharge loading application is larger for larger value of $LR$. The vacuum pressure applied is larger at $LR = 1.0$, which induced negative excess pore pressure increment and reduced maximum positive pore pressure.
Fig. 4.17 Excess pore pressure variation ($LR = 1.0; d_w = 8$ mm)

Fig. 4.18 Excess pore pressure variation ($LR = 2.0; d_w = 8$ mm)
Fig. 4.19 Excess pore pressure variation ($LR = 1.0; d_w = 4$ mm)

<table>
<thead>
<tr>
<th>SLR</th>
<th>Case</th>
<th>Pore pressure (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10kPa/10min O-3a</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>10kPa/90min O-3c</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>10kPa/360min O-3d</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 4.20 Excess pore pressure variation ($\sigma_{v0} = 20$ kPa)
Fig. 4.21 Excess pore pressure variation ($\sigma'_v = 40$ kPa)

(2) Effects of initial effective stress ($\sigma'_{v0}$)

While, under a different SLR but the same initial effective stress ($\sigma'_{v0}$) (Fig. 4.20 to Fig. 4.22), it can be seen that the larger the SLR, the higher the excess pore pressure. The excess pore pressure at the same SLR but different initial effective stress is shown in Fig. 4.23 ($SLR = 10$ kPa/30 min) and Fig. 4.24 ($SLR = 10$ kPa/60 min), the smaller the initial effective stress, the larger the excess pore pressure. The applied vacuum pressure was -80 kPa, but the measured values were about -70 to -40 kPa. Since the gauge was calibrated carefully before the tests, the possible reason considered that under the laboratory condition due to vacuum (suction) pressure, mini-drain can become unsaturated and at the interface between the mini-drain and soil forming convex meniscuses, which can reduce the efficiency of the vacuum pressure transferring into the soil sample.
Fig. 4.22 Excess pore pressure variation ($\sigma'_{v0} = 80$ kPa)

Fig. 4.23 Excess pore pressure variation ($SLR = 10$ kPa/30 min)
4.2.5.5 Coefficient of lateral earth pressure acting on the wall ($K_w$)

(1) Definition of $K_w$

Under oedometer condition, outward lateral displacement is restricted and only inward lateral displacement is allowed. While horizontal effective stress is changeable. Chai et al. (2013b) proposed a parameter $K_w$ as an indicator for judging the tendency of lateral displacement of a sample under oedometer condition. $K_w$ is defined as the ratio of horizontal incremental effective stress ($\Delta \sigma'_{hw}$) acting on the wall of the oedometer ring to the vertical incremental effective stress in the specimen ($\Delta \sigma'_v$). It is expressed by Eq. (4.1),

$$
K_w = \frac{\Delta \sigma'_{hw}}{\Delta \sigma'_v}
$$

(4.1)

The value of $\sigma'_{hw}$ is calculated from equation (4.2)

$$
\Delta \sigma'_{hw} = \begin{cases} 
\sigma_{ep} - u_e & (u_e > 0) \\
\sigma_{ep} & (u_e \leq 0)
\end{cases}
$$

(4.2)
where $\sigma_{ep}$ = the measured total horizontal earth pressure, $u_e$ = the measured excess pore water pressure at the level of earth pressure gauge. The vertical effective stress is calculated as in Eq. (4.3) and (4.4),

$$\Delta\sigma_v = \Delta\sigma_z - \bar{u}$$

(4.3)

$$\bar{u} = C_r (u_e + |\Delta\sigma_{vac}|) - |\Delta\sigma_{vac}|$$

(4.4)

where $C_r$ is a constant and can be derived from Barron (1948)'s solution. With the geometry of the sample, 60 mm in diameter and the central drain of 8 mm in diameter, $C_r = 0.857$, while for drain diameter of 4 mm, $C_r = 0.892$.

In this study, the parameter $K_w$ is used to investigate the tendency of lateral displacement of a sample under oedometer condition. When $K_w$ is larger than zero ($K_w > 0$) the sample will have a tendency of outward lateral displacement, vice versa. The negative value of $K_w$ indicates there is a micro-gap between soil sample and the consolidation ring.

(2) Time for calculating $K_w$

In the field, surcharge load induced outward lateral displacement mainly occurs immediately after the load application, i.e. undrained shear deformation, while vacuum pressure induced lateral displacement occurs during consolidation process. For a given geometrical condition, the loading rate will influence the excess pore water pressure generated in the specimen, and therefore the values of $K_w$ during loading, but may not significantly influence the final (end of consolidation) values of $K_w$. Considering most outward lateral displacement occurs during the surcharge load application process, the end of surcharge load application is chosen as a critical point for calculating $K_w$ from laboratory tests, and used it to judge the tendency of lateral displacement under oedometer condition.
Fig. 4.25 Variation of $K_w$ values ($LR = 1.0; d_w = 8$ mm)

Fig. 4.26 Variation of $K_w$ values ($\sigma'_{v0} = 20$ kPa)
Fig. 4.27 Variation of $K_w$ values ($\sigma'_{v0} = 40$ kPa)

Fig. 4.28 Variation of $K_w$ values ($\sigma'_{v0} = 80$ kPa)
Fig. 4.29 Variation of $K_w$ values ($SLR = 10$ kPa/30 min)

(3) Calculated $K_w$

(a) Effect of surcharge loading rate ($SLR$)

The variation of the stress ratio ($K_w$) with time under different surcharge loading rate ($SLR$) are presented on Figs. 4.25 to 4.27. Fig. 4.25 shows the variation of the values of $K_w$ for the cases of $LR = 1.0$, $d_w = 8$ mm, but different $SLR$. It can be seen that (1) with increase of $SLR$, the values of $K_w$ were increased during the surcharge loading process; and (2) influence of $SLR$ on the final value of $K_w$ is not significant. It can also be observed that when $\Delta \sigma_s$ is larger than about 40 kPa, for each surcharge load increment, the maximum $K_w$ value not varied much. The mechanism for this may be due to the surcharge loading induced shear stress increment and the consolidation induced shear strength increment are somehow balanced each other under the oedometer condition.

(b) Effect of initial effective stress ($\sigma'_v$)

With a different initial effective stress ($\sigma'_v$) (Fig. 4.29), the trend shows that the larger the $\sigma'_v$ the smaller the $K_w$ value. The higher the $\sigma'_v$ the smaller the different $K_w$. 
between two values of $SLR$. The denser the soil specimen the smaller the tendency to the lateral displacement (outward or inward).

4.2.6 Summary and comments

The reconstituted Ariake clay was used for radial drainage oedometer consolidation tests. The tests were conducted under combined vacuum pressure and surcharge load. The effects of surcharge loading rate ($SLR$) and loading ratio ($LR$) (a ratio of surcharge load applied divided to vacuum pressure applied), and initial effective stress in the sample on the lateral displacement tendency of soil samples were investigated. Total of 20 (twenty) cases were carried out. The oedometer apparatus was modified to allow the surcharge load and vacuum pressure can be applied together. Only radial drainage was allowed during the test. Under radial oedometer laboratory test, the measured results i.e. settlement, lateral earth pressure, and pore water pressure are presented. A stress ratio ($K_w$) which is defined as the ratio of horizontal effective stress increment ($\Delta \sigma'_{hw}$) acting on the wall of the consolidation ring divided to the vertical effective stress increment in the specimen ($\Delta \sigma'_{v}$) is introduced to indicate the tendency of lateral displacement of the sample, when the $K_w > 0$ there will be a tendency of outward lateral displacement and vice versa. The main findings are summarized as follows:

1) The test results indicate that factors influences lateral displacement were: (1) loading condition $SLR$ and $LR$; (2) initial effective stress ($\sigma'_{w0}$).

2) Reducing the $SLR$ can minimize the lateral displacement. The larger the $LR$ the larger the surcharge load applied, therefore the larger the tendency of outward lateral displacement. The larger the initial effective stress in the sample, the denser the soil, the smaller the tendency of lateral displacement.

4.3 Radial drainage laboratory triaxial consolidation test

4.3.1 Equipment and material used

The modified triaxial device was used on radial drainage laboratory triaxial consolidation test under surcharge load and vacuum pressure has described in detail in Chapter 3. The remolded Ariake clay used for triaxial test is the same as that used for oedometer test, it has described in detail in section 4.2.1 as well.
4.3.2 Cases tested

The reconstituted Ariake clay was used on radial drainage triaxial consolidation test under combination of surcharge load and vacuum pressure. The twelve (12) of test cases are listed in Table 4.3. The initial vertical effective stress ($\sigma_{v0}'$) was set up as 20, 40 and 60 kPa, representing the field condition of shallow, middle and deep depth. The initial at-rest coefficient of lateral earth pressure adopted was 0.6. The ratio of loading ($LR$) was 1.0 with a surcharge load increment ($\Delta\sigma_s$) of 60 kPa and a vacuum pressure ($\sigma_{vac}$) of -60 kPa. For the surcharge load it was applied step-wisely with an incremental of 10 kPa and a time interval between two steps of 30 min to 8 hours.

<table>
<thead>
<tr>
<th>Case</th>
<th>Initial stresses</th>
<th>$\Delta\sigma$ (kPa)</th>
<th>$\Delta\sigma_{vac}$ (kPa)</th>
<th>$LR$</th>
<th>SLR (kPa/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-1a</td>
<td>20</td>
<td>12</td>
<td>60</td>
<td>-60</td>
<td>10/30</td>
</tr>
<tr>
<td>T-1b</td>
<td>20</td>
<td>12</td>
<td>60</td>
<td>-60</td>
<td>10/60</td>
</tr>
<tr>
<td>T-1c</td>
<td>20</td>
<td>12</td>
<td>60</td>
<td>-60</td>
<td>10/90</td>
</tr>
<tr>
<td>T-1d</td>
<td>20</td>
<td>12</td>
<td>60</td>
<td>-60</td>
<td>10/480</td>
</tr>
<tr>
<td>T-2a</td>
<td>40</td>
<td>24</td>
<td>60</td>
<td>-60</td>
<td>10/30</td>
</tr>
<tr>
<td>T-3a</td>
<td>40</td>
<td>24</td>
<td>60</td>
<td>-60</td>
<td>10/60</td>
</tr>
<tr>
<td>T-4a</td>
<td>40</td>
<td>24</td>
<td>60</td>
<td>-60</td>
<td>10/90</td>
</tr>
<tr>
<td>T-5a</td>
<td>40</td>
<td>24</td>
<td>60</td>
<td>-60</td>
<td>10/480</td>
</tr>
<tr>
<td>T-3b</td>
<td>60</td>
<td>36</td>
<td>60</td>
<td>-60</td>
<td>10/30</td>
</tr>
<tr>
<td>T-3c</td>
<td>60</td>
<td>36</td>
<td>60</td>
<td>-60</td>
<td>10/60</td>
</tr>
<tr>
<td>T-3d</td>
<td>60</td>
<td>36</td>
<td>60</td>
<td>-60</td>
<td>10/90</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>10/480</td>
</tr>
</tbody>
</table>
4.3.3 Test Procedures

4.3.3.1 Sample preparation

(1) Specimen preparation

(a) A cylinder soil specimen as for conventional triaxial test was made with pre-consolidated Ariake clay (from slurry) under 20 kPa vertical consolidation pressure.

(b) Then the specimen was fixed on a frame (guide) and an 8 mm in diameter central hole was made using a drilling tool manually.

(2) Saturating the specimen

A vacuum pressure of about 20 kPa was applied to the dessicator with the soil specimen inside, for about 1 hour to eliminate possible trapped air bubbles between the rubber membrane, the drain, and the soil specimen. The detail of saturating the specimen has been described in Chapter 3.

4.3.3.2 Pre-consolidation

The anisotropic consolidation stresses were applied to consolidate the specimen for 1 day. For the cases tested, a value of $K_0$ assumed was 0.6 in this step. To avoid excessive shear deformation in the pre-consolidation process, the pre-consolidation pressures were applied in a stepwise manner with a vertical stress increment of 10 kPa and horizontal stress increment of 6 kPa and a time interval of 2 hours. The same procedure for pre-consolidation has been described in detail in Chapter 3.

4.3.3.3 Consolidation under combined loads

After the pre-consolidation step, the vacuum pressure of -60 kPa was applied through the central drain instantaneously. Then the surcharge load increment (10 kPa for each step) was applied stepwise with desired loading rate using dead load. The total surcharge load applied was 60 kPa.
4.3.4 Test results

4.3.4.1 Settlement

(1) Effect of surcharge loading rate (SLR)

Time-settlement relations with different value of SLR are shown in Fig. 4.30. For the oedometer test, only axial deformation was allowed due to restricted by a consolidation ring, while for triaxial test a sample could deform in axial and lateral directions. Therefore SLR influenced settlement. It can be seen that increase SLR, increased the rate of settlement and the final settlement as well. This significant different is due to the more shear deformation induced by the faster (larger) surcharge loading rate.

![Fig. 4.30 Settlement-time relationship (\(\sigma'_{v0} = 20 \text{ kPa}\))](image)

(2) Effect of initial effective vertical stress (\(\sigma'_{v0}\))

Measured settlement curves with different initial effective stress (\(\sigma'_{v0}\)) are shown in Fig. 4.31. It clearly shows that with the same SLR, increase the initial effective stresses (\(\sigma'_{v0}\)) in the sample reduced the settlement.
4.3.4.2 Excess pore pressure

(1) **Effect of surcharge loading rate (SLR)**

Under the combination of surcharge load and vacuum pressure, excess pore pressures are shown on Fig. 4.32 and Fig. 4.33. At the end of the surcharge load application, the higher the SLR, the higher the \( u \) value. The difference of \( u \) values between three values of SLR before elapsed time of 50 min can be considered as the scatter of the measured data. The applied vacuum pressure to the central drain was -60 kPa, but the measured final value was about -45 kPa.

For the case the initial effective stress (\( \sigma'_{e0} \)) equal to 60 kPa (Fig. 4.33), the distinction of \( u \) between the three values of SLR is not significant. At the end of consolidation the minimum \( u \) was about -35 kPa.
Fig. 4.32 Variation of pore water pressure ($\sigma'_{v0} = 20$ kPa)

Fig. 4.33 Variation of pore water pressure ($\sigma'_{v0} = 60$ kPa)
Effect of initial effective vertical stress ($\sigma'_{v0}$)

The excess pore pressure variation with different initial vertical effective stress ($\sigma'_{v0}$), is shown in Fig. 4.34. It can be seen that the higher the initial effective stress in the sample the smaller the excess pore pressure measurement. The possible reason may be due to the higher the initial effective stress in the sample, the denser the sample, the smaller the propagated of vacuum pressure into the sample.

![Graph showing the variation of pore water pressure over time with different initial vertical effective stresses.](image_url)

**Fig. 4.34 Variation of pore water pressure ($SLR = 10 \text{kPa/30 min}$)**

4.3.4.3 Horizontal strain

(1) **Introduction**

The diameter change of the specimen was used to calculate the horizontal strain after consolidation test. The diameter of the specimen was measured before and after the consolidation test (as described in Chapter 3).
(2) Effect of initial effective stress

\[ \sigma'_{v_0} = 20 \text{ kPa} \]

![Fig. 4.35 Shape of the specimen at end of consolidation (\( \sigma'_v = 20 \text{ kPa} \))](image)

\[ \sigma'_{v_0} = 60 \text{ kPa} \]

![Fig. 4.36 Shape of the specimen at end of consolidation (\( \sigma'_v = 60 \text{ kPa} \))](image)
The sketch shapes of cylinder soil after consolidation test for 20 and 60 kPa initial effective stress ($\sigma'_{\gamma_0}$) are shown in Fig. 4.35 and Fig. 4.36, it can be seen that the larger the surcharge loading rate (SLR) the larger the settlement and the larger the outward lateral displacement. It was mentioned previous that a faster surcharge loading rate will induce larger shear deformation and therefore outward lateral displacement. This phenomenon is clearly shown in Fig. 4.37. In that figure the outward lateral displacement (strain tensile) is marked as negative (-) and inward is positive. Under the combination of 60 kPa surcharge load and -60kPa vacuum pressure, all tests resulted in tensile horizontal strain (outward lateral displacement). It can be seen that the smaller the $\sigma'_{\gamma_0}$ the larger the tensile horizontal strain, and the smaller the SLR the smaller the tensile horizontal strain.

![Fig. 4.37 Horizontal Strain – SLR relationship under triaxial test with different initial pressure ($\sigma'_{\gamma_0}$)](image)

**4.3.4.4 Stress ratio ($K$)**

Under an ordinary triaxial condition the total confining pressure is a constant, but there is no restriction on horizontal displacement. Thus if the horizontal effective stress ($\sigma'_h$) is less than the stress ($K_0.\sigma'_\gamma$) required to maintain a no horizontal...
deformation \((K_0)\) state, there will be outward lateral deformation. Further, if \(\sigma'_h > K_0 \sigma'_v\), there will be inward lateral displacement.

A stress ratio \((K)\) is defined as the ratio of horizontal effective stress \((\sigma'_h)\) to the value in the vertical direction \((\sigma'_v)\) as \(K\). Therefore, the tendency of lateral displacement can be easily judged using the \(K\) (Eq. (3.1)).

\[
K = \frac{\sigma'_h}{\sigma'_v} \quad (3.1\text{bis})
\]

For a triaxial test with a drain at the centre of the soil specimen, to calculate the average effective stress in the specimen, the consolidation theory for vertical drain needs to be used. In Hansbo’s (1981) solution for the consolidation due to the effect of a vertical drain, assuming an equal vertical strain condition, the excess pore water pressure distribution in the radial \((r)\) direction is shown by equation (4.5):

\[
u(r) \propto \left( \frac{r_e^2}{r_w^2} \ln\left(\frac{r}{r_w} - \frac{r^2 - r_w^2}{2}\right) \right)
\]

where \(r_e\) = the radius of a unit cell of PVD improvement (here the radius of the soil specimen); and \(r_w\) = the equivalent radius of the vertical drain. For the triaxial test conducted in this study, the pore water pressure is measured at a location with a radial distance of \(r_a = 17\) mm on the base pedestal and the value is designated as \(u_a\). To calculate the average pore pressure in the specimen from the measured value of \(u_a\), the radial distribution given by Eq. (4.5) has to be considered. By integrating Eq. (4.5) with cross-sectional area, a parameter \(\beta\) is introduced as in equation (4.6),

\[
\beta = \frac{\ln\left( \frac{r_e}{r_w} \right) - \frac{3}{4} + \frac{r_w^2}{r_e^2} - \frac{r_w^4}{4r_e^4}}{\left[ 1 - \frac{r_w^2}{r_e^2} \right] \ln\left( \frac{r_a}{r_w} \right) - \frac{r_w^2 - r_e^2}{2r_e^2}}
\]

Then for a combined vacuum and surcharge loading, \(\sigma'_v\) can be calculated as in equation (4.7),

\[
\sigma'_v = \sigma'_{v0} + \Delta \sigma_v + |\Delta \sigma_{vac}| \cdot (u_a + |\Delta \sigma_{vac}|) \cdot \beta
\]

(4.7)
where $\sigma_{v0}'$ = the initial vertical effective stress; $\Delta \sigma_s$ = the applied surcharge load; and $\Delta \sigma_{vac}$ = the applied vacuum pressure. The value of $\sigma_h'$ can be expressed as in equation (4.8),

$$\sigma_h' = \sigma_{h0}' + |\Delta \sigma_{vac}| + (u_u + |\Delta \sigma_{vac}|) \cdot \beta$$

(4.8)

where $\sigma_{h0}'$ = the initial horizontal effective stress.

It is considered that the closer the $K$ value to $K_0$, the smaller the lateral displacement. It was explained previously that considering most outward lateral displacement occurs during the surcharge load application process, the end of surcharge load application is chosen as a critical point for calculating $K$ of laboratory triaxial tests, and used it to judge the tendency of lateral displacement.

(1) **Effect of surcharge loading rate (SLR) to the stress ratio ($K$)**

The calculated values of stress ratio ($K$) by Eqs (3.1), (4.7) and (4.8) are presented in Fig. 4.38 and Fig. 4.39. For these three values of SLR, during the consolidation period, the $K$ value tend to increase with elapsed time, however when the
incremental surcharge loading was applied, the $K$ values gradually reduced with the application of the surcharge load. Considering the condition at the end of load application that the larger the $SLR$ the smaller the stress ratio ($K$). Therefore, reducing the $SLR$ increased the value of $K$ at the end of surcharge loading application. During the final consolidation process there is no significant difference on $K$ value with different $SLR$. For remoulded Ariake clay, the coefficient of at-rest earth pressure is about 0.4 (Chai and Kawaguchi 2011). The $K < K_0$ means that the sample would deform lateral in the direction of increased the diameter of the sample (i.e. tensile horizontal strain) at that time. With the progress of consolidation and increase of vacuum pressure, the $K$ value was gradually increased and at the time of termination of the tests, the values are about 0.45.

\[ \sigma'_{v0} = 60 \text{ kPa} \]
\[ \sigma_{vac} = -60 \text{ kPa} \]
\[ \sigma_s = 60 \text{ kPa} \]

Fig. 4.39 Time – $K$ variation ($\sigma'_{v0} = 60$ kPa)

(2) **Effect of initial effective stress ($\sigma'_{v0}$)**

Fig. 4.40 shows $K$ variation under the same surcharge loading rate ($SLR = 10$ kPa/ 30 min) but different initial effective stress ($\sigma'_v = 20, 40, 60$ kPa). The smaller the $\sigma'_v$ the smaller the $K$ value at the end of the surcharge load application.
4.3.4.5 $K - \varepsilon_H$ relationship

The $K$ value versus horizontal strain for both the undisturbed sample (the detail was described in Chapter 3) and the reconstituted samples is shown in Fig. 4.41. For the combined load the value of $K$ corresponding to the end of surcharge load application, and for the vacuum pressure alone values of $K$ are final values. There are close relationship between stress ratio ($K$) and horizontal strain, it is considered that when $K = K_0$ will result in minimum lateral displacement. The increase the initial effective stress ($\sigma'_0$) reduced the strain close to zero and increased the stress ratio ($K$) value close to $K_0 = 0.41$. As for the effect of $SLR$, reduce the $SLR$ increased the $K$ value (close to $K_0 = 0.41$) and reduced the horizontal strain to close to zero. The data points for undisturbed sample are scatter, and it may be due to the spatial variation of soil properties of the sample. The tendency shows that, when the stress ratio ($K$) less than $K_0 = 0.41$, it will result in larger outward lateral displacement and vice versa.
4.3.5 Summary and comments

The radial drainage laboratory triaxial consolidation test has been conducted under combination of surcharge load and vacuum pressure using undisturbed Ariake clay. An ordinary triaxial apparatus was modified to allow the surcharge load and vacuum pressures can be applied together. Only radial drainage was allowed during the consolidation process. The measurements were settlement, pore pressure and the lateral displacement (horizontal strain). The lateral displacement was only measured at end of the consolidation test.

Further a stress ratio ($K$) was introduced as indicator for judging the tendency of lateral displacement. $K$ is defined as the ratio of horizontal effective stress divided by the vertical effective stress acting on the soil sample. When the $K > K_0$ ($K_0$ is the coefficient of at-rest lateral earth pressure), the soil sample tend to become slimmer (inward lateral displacement), and when $K < K_0$, the tendency is outward lateral displacement.

![Fig. 4.41 $K - \varepsilon_h$ relationship](image)

Fig. 4.41 $K - \varepsilon_h$ relationship
The tests were conducted to investigate the effect of loading condition (surcharge loading rate (SLR) and loading ratio (LR) which is the ratio of surcharge loading applied to a vacuum pressure applied) to the lateral displacement. The following results were obtained:

a) Reducing the surcharge loading rate (SLR) reduced the settlement, reduced the lateral displacement and increased the stress ratio (K).

b) Increasing the initial stress (σ′₀/v₂⁰) in the sample, reduced the settlement and the lateral displacement, and increased the value of K.

c) Values of optimum SLR resulting in minimum lateral displacement will increase with the increasing of initial effective stress in the soil.

d) When K ≈ K₀ the horizontal strain of the sample is close to zero.

4.4 Design method for minimizing lateral displacement

4.4.1 Main factors influencing the lateral displacement

4.4.1.1 Effect of loading ratio (LR)

Loading ratio (LR) is defined as a ratio of the surcharge load applied to the vacuum pressure applied. The larger the LR means the larger the magnitude of surcharge load applied. Under the combination of surcharge load and vacuum pressure, in case of surcharge load dominates, there is a tendency of outward lateral displacement. For the oedometer condition the sample was restricted by consolidation ring, therefore the outward lateral displacement wasn’t allowed. A stress ratio Kw is used as indicator for judging the tendency of outward lateral displacement. Chai et al. (2005) proposed that under oedometer condition, if there is no effective confinement from the consolidation ring, and vacuum pressure alone can maintain K₀ condition of the sample, there will be still no gap occurring between the consolidation ring and the soil sample. This condition is the same as Kw = 0. Therefore when the Kw > 0 the sample tend to deforms with outward lateral displacement and vice versa. The smaller the LR value the smaller the tendency of outward lateral displacement.
Fig. 4.42 SLR – $K_w$ relationship

Fig. 4.43 SLR – $K$ relationship
4.4.1.2 Effect of Surcharge loading rate (SLR)

Considering most outward lateral displacement occurs during the surcharge load application process, the end of surcharge load application is chosen as a critical point for calculating $K_w$ or $K$ ($K$ is a ratio of horizontal effective stress increment to the vertical effective stress increment under triaxial condition) from laboratory tests, and used them to judge the tendency of lateral displacement of a sample. The $SLR - K_w$ and $SLR - K$ curves are shown in Fig. 4.42 and Fig. 4.43. Reducing the value of $SLR$ reduced the $K_w$ for the oedometer consolidation test and reduced the tendency of outward lateral displacement. Reverse on triaxial consolidation test, reducing the $SLR$ increased the $K$ and reduced the tendency of outward lateral displacement. So that, for both the oedometer and triaxial conditions, reducing $SLR$ reduced the tendency of outward lateral displacement.

4.4.2 Introducing a dimensionless parameter $\alpha$

For a prefabricated vertical drain (PVD) improved deposit, the effect of $SLR$ on lateral displacement of a deposit is related to the consolidation properties of the deposit and the spacing of PVDs. The drainage condition under field is different from that in the laboratory. The degree of consolidation of a PVD improved deposit is controlled by a dimensionless time factor of $T_r$ as in Eq. (4.9),

$$T_r = \frac{c_h t}{4r_s^2 \mu}$$ (4.9)

where $c_h$ = coefficient of consolidation in the horizontal direction, and $t$ = time. Using Hansbo (1981)’s solution, the expression of $\mu$ is:

$$\mu = \ln(r_s/r_e) + (k_h/k_s) \ln(s) - \frac{3}{4} + \frac{2\pi \cdot l^2 \cdot k_h}{3q_w}$$ (4.10)

where $s = r_s/r_w$ ($r_s$ is the radius of the smear zone); $l$ = drainage length of a PVD; $k_h$ and $k_s$ = the horizontal hydraulic conductivities of natural soil and smear zone respectively; and $q_w$ = the discharge capacity of a PVD. In Eq. (4.9), except $t$, all parameters are basic soil properties of a deposit or the geometric parameter of PVD improvement. Let’s define a dimensionless parameter ($\alpha$) as in equation (4.11),

93
\[ \alpha = \frac{SLR}{T_p P_a} = \frac{4 \cdot SLR \cdot r^2 \mu}{c_k P_a} \]  

(4.11)

where \( p_a \) = atmospheric pressure. It is considered that the dimensionless parameter \( \alpha \) can be applied to both the laboratory and the field condition. It is aimed to establish \( \alpha - K_w \) and/or \( \alpha - K \) relationships from the test results. It is considered that an \( \alpha \) value corresponding to \( K_w = 0 \) (odometer condition) or \( K = K_0 \) (triaxial condition) will result in minimum lateral displacement. Then the corresponding value of \( SLR \) will be the optimum value for the purpose of minimizing the lateral displacement under the field condition. Most outward lateral displacement occurs during the surcharge load application process (Chai and Rondonuwu 2015), therefore the end of surcharge load application is chosen as a critical point for calculating \( K_w \) or \( K \) from laboratory tests.

4.4.3 Design curves

4.4.3.1 \( \alpha - K_w \) relationship

\( K_w \) versus \( \alpha \) relationship evaluated from the test results at the end of surcharge load application are summarized in Fig. 4.44 for \( \sigma'_{v_0} = 0 \) and Fig. 4.45 for \( \sigma'_{v_0} > 0 \). It can be clearly seen that for a given \( LR \), \( K_w \) increased with the increase of \( \alpha \). Comparing \( LR = 1 \) and 2 cases shows that the larger the \( LR \), the higher the \( K_w \). Although the data are scattered, for \( \sigma'_{v_0} = 0 \) (soil layer near ground surface in the field) cases, using \( K_w = 0 \) criterion, for \( LR = 1 \), \( \alpha \) value of about 2.3 can be estimated and for \( LR = 2 \) cases, the value is about 1.0, i.e. for a larger \( LR \) case, a smaller \( \alpha \) value (slower loading rate) is required to minimize the lateral displacement. The test results also indicate that increasing the initial effective stresses (\( \sigma'_{v_0} \)) in the sample, reduced the \( K_w \) value (Fig. 4.45). Larger initial effective stresses in the sample means higher initial undrained shear strength, and for a given surcharge load, it will result in less undrained shear deformation.

Conceptually, the result in Fig. 4.44 and Fig. 4.45 can be used for design a field \( SLR \) for a prior decided \( LR \) value, i.e. for a given \( LR \) value, an \( \alpha \) value corresponding \( K_w = 0 \) can be estimated from the figure. Then \( SLR \) can be evaluated using coefficient of consolidation of the deposit and parameters related to PVD improvement. It is expressed by Eq (4.12),
Fig. 4.44 $K_w$-$\alpha$ relationship on oedometer consolidation test with $\sigma'_{v_0} = 0$

Fig. 4.45 $K_w$-$\alpha$ relationship on oedometer consolidation test with $\sigma'_{v_0} > 0$
4.4.3.2 \( \alpha - K \) relationship

\( \alpha - K \) relationships from the results of the modified triaxial test are shown in Fig. 4.46 corresponding to the end of surcharge load application. It clearly shows that \( K \) increased with the reducing of \( \alpha \). For a given \( \alpha \) value, \( K \) increased with the increase of initial effective stresses in the sample. Using \( K = K_0 \) (\( \approx 0.41 \) for the samples tested) criterion, for \( \sigma'_{v_0} = 20 \) kPa, an \( \alpha \) value which may result in minimum lateral displacement, of about 0.8 can be estimated, which is smaller than that from the oedometer test results.

![K-\( \alpha \) relationship on triaxial consolidation test](image)

The oedometer test results giving a larger \( \alpha \) values than that from the triaxial test results and the exact reason is not clear yet. For the device used, the lateral earth pressure gauge has a diameter of 10 mm with a flat surface. While, the consolidation ring has an arc inner surface, i.e. the surface of the earth pressure gauge is not matched.
the inner surface of the ring. To ensure a good contact between the sample and the ring, the remolded soil was reconstituted directly inside the ring as explained previously. However, during the consolidation process, the soil around the earth pressure gauge will experience not only vertical compression, but also some complicated shear deformation and that may influence the measured earth pressure (possibly lower than the value supposed to be), and therefore possibly the lower calculated $K_w$ value, and for $K_w = 0$, a larger $\alpha$ value. Considering these factors, it is recommended to use the triaxial test results (Fig. 4.46) for designing SLR of a field preloading project.

### 4.4.4 $\alpha$ versus horizontal strain ($\varepsilon_h$)

For the modified triaxial test, the measured horizontal strains versus $\alpha$ values are shown in Fig. 4.46. Although all samples had tensile horizontal strain, the absolute magnitude of the horizontal strain reduced with reducing $\alpha$ value and increasing initial effective stresses in the samples.

![Fig. 4.47 $\varepsilon_h$-$\alpha$ relationship on triaxial consolidation test](image)

From the results in Fig. 4.46 and the method that proposed, under $\sigma'_{v0} = 20$ kPa, and at $\alpha$ of about 0.8, zero horizontal strain is predicted, but Fig. 4.47 shows that the final horizontal strain about -1%. There are two possible reasons for this apparent
inconsistency. One is that after dismounting the sample, unloading would tend to increase of its size. Another one is that during the consolidation test, a sustained deviator stress was applied to the samples, and it may cause certain creep deformation. Furthermore the target considered is the end of surcharge load application and in terms of horizontal strain it may be different from the final condition.

4.4.5 Summary

A method for designing the surcharge loading rate ($SLR$) to minimize lateral displacement under the combination of vacuum pressure and surcharge load has been proposed. The main factors influencing lateral displacement i.e $SLR$, $LR$ (ratio of surcharge load to the vacuum pressure), initial effective stress, and the spacing of prefabricated vertical drain (PVD) and consolidation properties of a deposit were considered for proposing the method.

The parameter $K_w$ is the ratio of horizontal effective stress increment acting on the inner surface of the consolidation ring of an odometer test to the vertical effective stress increment in the soil sample. While, $K$ is a ratio of horizontal effective stress to the vertical effective stress under triaxial condition. For odometer condition if $K_w > 0$ there is a tendency of outward lateral displacement of a sample and vice versa. Under the triaxial condition, when the $K < K_0 (\approx 0.41)$ there is a tendency of outward lateral displacement of a sample and vice versa.

A dimensionless parameter ($\alpha$) alpha is introduced for determining optimum $SLR$ that will result in minimum lateral displacement for a given $LR$ value.

The $\alpha$-$K_w$ (for oedometer) or $\alpha$-$K$ (for triaxial) relationships were established from the laboratory test results. From $\alpha$-$K_w$ relationship, $\alpha$ value and therefore $SLR$ corresponding to $K_w = 0$, and from $\alpha$-$K$ relationship $\alpha$ ($SLR$) value corresponding to $K = K_0$ will result in minimum lateral displacement under the field condition. If the $SLR$ values from $\alpha$-$K_w$ and $\alpha$-$K$ relationships are different, it is recommended to use the result from $\alpha$-$K$ relationships.
4.5 Summary

A series of radial drainage consolidation laboratory tests (oedometer and triaxial) were conducted under combination of surcharge load and vacuum pressure. A modified oedometer and modified triaxial devices were used for the consolidation tests. The main modification of both oedometer and triaxial devices were: 1) the vacuum pressure and surcharge load can be applied together, 2) the mini-drain was installed into the specimens act as vertical drain. The effect of loading condition which ratio of surcharge loading applied to the vacuum pressure applied (LR) and surcharge loading rate (SLR) to the lateral displacement were investigated.

Under radial drainage laboratory oedometer consolidation test, the settlement, lateral earth pressure and excess pore pressure were measured. A coefficient of lateral earth pressure acting on the wall of odoemeter ring, $K_w$, is defined as the ratio of horizontal effective stress increment on the wall to the vertical effective stress increment in the sample, is used as indicator for judging the tendency of lateral displacement. When $K_w > 0$ the sample tends to develop outward lateral displacement, and when the $K_w < 0$ the tendency is inward lateral displacement. It is found that reducing the value of LR, and SLR, reduced the tendency of outward lateral displacement.

For the triaxial tests settlement, excess pore pressure and horizontal strain at end of the consolidation test were measured. A stress ratio, $K$, parameter defined as ratio of horizontal effective stress to the vertical effective stress, was used as indicator for judging the tendency of lateral displacement. If the $K < K_0$ ($K_0$: at-rest earth pressure coefficient) the sample tends to develop outward lateral displacement, and when the $K > K_0$, the tendency is inward lateral displacement. For Ariake clay a $K_0 \approx 0.41$, was adopted. From the test results it is found that by reducing the SLR will increase the value of $K$ (end of surcharge load application) close to $K_0 \approx 0.41$ so that reduced the tendency of outward lateral displacement. Increasing the initial effective stress, $\sigma'_v0$, increased the value of $K$ and reduced the tendency of outward lateral displacement.

Based on the laboratory test results, a method for designing the surcharge loading rate (SLR) to minimize lateral displacement under the combination of vacuum pressure and surcharge load has been proposed as follows:
Firstly, a dimensionless parameter, $\alpha$, which is basically the ratio of SLR and consolidation time factor is introduced.

Then, $\alpha$-$K_w$ (for oedometer) or $\alpha$-$K$ (for triaxial) relationships were established from the laboratory test results. From $\alpha$-$K_w$ relationship, $\alpha$ value and therefore SLR corresponding to $K_w = 0$, and from $\alpha$-$K$ relationship $\alpha$ (SLR) value corresponding to $K = K_0$ will result in minimum lateral displacement under the field condition. If the SLR values from $\alpha$-$K_w$ and $\alpha$-$K$ relationships are different, it is recommended to use the result from $\alpha$-$K$ relationships.
CHAPTER 5
CONCLUSIONS

5.1 Introduction

The factors influencing the deformation characteristics of soft deposit with prefabricated vertical drain (PVD) improvement under a vacuum pressure alone or a vacuum pressure combined with a surcharge load were investigated. The methods for predicting the lateral displacement as well as minimizing the lateral displacement of a deposit were proposed. It has been found that for a given soil deposit the loading condition (surcharge loading rate \((SLR)\) and ratio of the surcharge load to vacuum pressure \((LR)\)), and initial effective stress in the soil are the main influencing factors for lateral displacement. A method for determining an optimum \(SLR\) for resulting in minimum lateral displacement was proposed based on the results of laboratory radial drainage consolidation test. Furthermore, a method has been established for predicting the deformation induced by vacuum consolidation in the field by laboratory triaxial consolidation test results.

5.2 Main factors affecting lateral displacement

Three investigated main factors influencing the lateral displacement are: (1) surcharge loading rate \((SLR)\); (2) ratio of loading \((LR)\) and; (3) initial effective stresses in the soil.

(1) Surcharge loading rate \((SLR)\)

\(SLR\) is a parameter to indicate how fast the surcharge load is applied. Surcharge load induces shear deformation in the ground. Therefore, the outward lateral displacement can be controlled by controlling \(SLR\). Reduction in \(SLR\) will reduce the outward lateral displacement.

(2) Loading ratio \((LR)\)

The larger the \(LR\), the larger the relative magnitude of surcharge load applied and the stronger is the tendency of outward lateral displacement.
(3) Effect of initial stress state ($\sigma'_{i,0}$)

(a) Initial vertical effective stress ($\sigma'_{v,0}$) under oedometer condition

The initial vertical effective stress under no lateral displacement condition ($K_0$ condition) is related to strength and stiffness of a deposit. The higher the value of $\sigma'_{v,0}$, the higher the undrained shear strength, and therefore for a given magnitude of shear stress the less shear stress induced shear deformation.

(b) Effective confinement under triaxial condition

When the initial confining stress is equal to the at-rest earth pressure, it is defined as $K_0$ confinement, and when the confining stress is the active earth pressure, it is called $K_a$ confinement. From the results of laboratory triaxial vacuum consolidation test, it has been found that $K_0$ confinement induced less settlement than $K_a$ confinement. Also under $K_a$ confinement, the outward lateral displacement is larger than that of $K_0$ confinement.

5.3 Proposed method for determining optimum SLR

Under oedometer condition a parameter $K_w$, defined as the ratio of increment effective horizontal stress acting on the inner wall of the consolidation ring to the increment vertical effective stress in the soil sample, was used to evaluate the lateral deformation tendency. While, under triaxial condition, the ratio of horizontal effective stress to that in the vertical direction, $K$, is used to determine the tendency of the lateral displacement of the samples, i.e. inward or outward. When $K_w > 0$ or $K < K_0$ (at-rest earth pressure coefficient), there will be a tendency of outward lateral displacement and vice versa. When $K_w = 0$ or $K = K_0$, the lateral displacement can be minimized. These conditions can be found from the laboratory test results. Both $K_w$ and $K$ are function of $SLR$ and $LR$.

The effect of $SLR$ on the lateral displacement is related to the consolidation rate of a deposit in question. The higher the consolidation rate the lesser the effect of $SLR$. However, the consolidation rate in a field conditions is normally different from that in a laboratory condition. To apply the laboratory test results to the field conditions, a dimensionless parameter $\alpha$, which are the functions of both $SLR$ and rate of consolidation (coefficient of consolidation and parameters related to prefabricated
vertical drain improvement) was proposed. Considering $K_w$ (or $K$) value corresponding to the end of surcharge loads application, $\alpha - K_w$, and $\alpha - K$ relationships were proposed using the laboratory test results. It is suggested that the relationships can be used for designing a field $SLR$ for a known $LR$ value.

5.4 Field stress state under vacuum consolidation

By comparing the results of laboratory triaxial vacuum consolidation test with the field measurements of a vacuum consolidation project in Saga Japan, it has been found that the effective confinement due to gravity force from the surrounding soils to the vacuum treated area is close to the value of active earth pressure for a zone about 5 m depth from the ground surface, and below it, the effective confinement due to gravity force is between active and at-rest states.

5.5 Predicting vacuum consolidation induced ground deformation

A theoretical matching equation has been derived to convert consolidation time in laboratory to the field case. It has been shown that with the matching equation, the results of laboratory vacuum consolidation test under triaxial condition with appropriate initial confining stress can be used to predict field settlement (compression) curves, and lateral displacement profile.
REFERENCES


