Consolidation behavior of Ariake clay under constant rate of strain

Dissertation Submitted to Saga University in Fulfillment of the Requirement for the Degree of Doctor of Engineering

by

Rui Jia

Dissertation Supervisor: Prof. Chai Jinchun

Examination Committee: Prof. Chai Jinchun
Prof. Ishibashi Kouji
Associate Prof. Sakai Akira
Associate Prof. Hino Takenori

External Examiners: Prof. Hong Zhenshun

Rui Jia
Nationality: Chinese
Previous degrees: Bachelor of Engineering (Civil)
Hohai University, Nanjing, China

Master of Engineering (Civil)
Hohai University, Nanjing, China

Department of Engineering Systems and Technology
Graduate School of Science and Engineering
Saga University
Saga, Japan
2010
ACKNOWLEDGEMENTS

The author wishes to express his profound and sincere gratitude to his supervisor, Prof. Chai Jinchun, for his understanding, encouraging, excellent guidance and constant support throughout. His wide knowledge and his logical way of thinking have been great value for the author. Without Prof. Chai’s unstinted help in both academic and personal concerns, this dissertation work could not be completed.

The author is also very grateful to Assoc. Prof. Hino Takenori and Negami Takehito for providing the laboratory materials. Sincere thanks and appreciation are due to Prof. Ishibashi Kouji, Assoc. Prof. Sakai Akira, Assoc. Prof. Hino Takenori for their help, suggestions, and serving as members of his doctoral program committees. Sincere thanks and appreciation are due to Prof. Hong Zhenshun of the University of Southeast for his views and constructive comments and serving as external examiner. Sincere thanks and appreciation are due to former supervisor Prof. Zhu Wei of the University of Hohai for his help and encouragement. Thanks are also extended to the soil laboratory staff Saito Akinori, for help to conduct experiments.

The author would like to acknowledge with appreciation the scholarship grant provided by Saga University which made it possible to pursue his doctoral studies at Saga University, Saga, Japan.

Sincere thanks are due to my colleagues Mr. Ong Chin Yee, Mr. Pongsivasathin Supasit, Mr. Putera Agung Maha Agun, Mrs. Kartika Sari, Mr. Tsutsumi Takeshi, Mr. Kawaguchi Yutaro, Mr. Suematsu Genki and Mr. Shimoda Satoshi for their help and encouragement. A word of thanks goes to the secretaries in our research team, Ms. Komoto Misa and Mrs. Kanada Yasuko for their kindness and help. Sincere thanks are due to all my friends who help me get through three years of graduate school.

Finally, the author wishes to express his love and gratitude to his parents and sister for their understanding and endless love through the duration of his studies.
ABSTRACT

For civil engineering projects related to soft clayey deposits, engineers inevitably have to consider the consolidation behaviour of the deposit. Due to the natural viscosity of clayey soils, their consolidation behaviour is strongly influenced by strain rate. Also sedimentary clay, such as soft Ariake clay, exhibits anisotropic consolidation behavior, having different consolidation properties in the vertical and the horizontal directions. An understanding of the strain rate effect on consolidation behaviour and the anisotropic consolidation behavior is useful for designing geotechnical projects such as embankments, foundations in regions with clayey deposits. An odometer can conduct constant rate of strain (CRS) consolidation test with vertical or radial drainage was developed and used to experimentally investigate the consolidation behavior of Ariake clay. A total of 248 CRS consolidation tests were conducted for undisturbed Ariake clay samples from seven boreholes in Saga Plain, Kyushu, Japan. The main items investigated are: consolidation yield stress ($p_c$), compression index ($C_c$), the coefficient of consolidation ($c_v$) and the hydraulic conductivity ($k$).

The test results show that the consolidation behavior of Ariake clay is strain rate dependent. There is a clear relationship between the strain rate effect (SRE) and clay content. Increase of clay content, increases SRE. Regarding the $p_c$ values of Ariake clay, it increased by about 14% with a tenfold increase in the strain rate. The ratio of consolidation yield stress in the horizontal direction ($p_{ch}$) to that in the vertical direction ($p_{cv}$) is in the range of 0.5 to 1.0, and the average value is about 0.7. For $C_c$ value, at a given strain level, the strain rate does not influence $C_c$, which implies that the isotach model is applicable to Ariake clay.

Under a given effective vertical stress, $c_v$ increased with the increase of strain rate resulting mainly from the increase of $k$. Linear regression results in about a 30% increase of $c_v$ for a tenfold increase in the strain rate. The coefficient of consolidation in the horizontal direction ($c_h$) is larger than that in the vertical direction ($c_v$), and it is mainly from the anisotropy of $k$. The ratio of $k$ in the horizontal direction ($k_h$) from radial drainage CRS test to that in the vertical direction ($k_v$) is about 1.65. The $k_h$ value from CRS test using horizontally cut sample with vertical drainage is smaller than that from CRS test using vertically cut sample with radial drainage. The possible reason is the different deformation pattern of the two types of tests.

There are two theories for interpreting CRS test results, i.e. small strain theory and large strain theory. The small strain theory is simple and is adopted into the standards for CRS test. While the large strain theory will be much more appropriate under the condition of large strain and high strain rate. The strain distribution pattern within the sample of CRS test obtained by the small strain theory and the large strain theory were compared with the results of numerical simulations. The large strain theory compares well with the result of the numerical analysis for a wide range of strain rates. It has been clarified that the large strain theory results in smaller $c_v$ value and less strain rate effect (14% increase of $c_v$ for a tenfold increase in the strain rate). It is suggested that small strain theory can only be used for small strain rate ($r$) with $c_v/rH^2 \geq 10$ ($H$ is the thickness of soil sample).

Comparing the $p_c$ and $c_v$ values from the incremental loading (IL) and CRS tests suggests that the $p_c$ and $c_v$ values from CRS tests for a strain rate of 0.02%/min are comparable with those of IL tests. The analysis shows that for Ariake clay tested, the average strain rate for up to 90% degree of consolidation is about 0.02%/min.
# 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>i</td>
</tr>
<tr>
<td></td>
<td>Acknowledgements</td>
<td>ii</td>
</tr>
<tr>
<td></td>
<td>Abstract</td>
<td>iii</td>
</tr>
<tr>
<td></td>
<td>Table of Contents</td>
<td>vi</td>
</tr>
<tr>
<td></td>
<td>List of Figures</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>List of Tables</td>
<td>xi</td>
</tr>
<tr>
<td></td>
<td>List of Notations</td>
<td></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>Objective and scope of research</td>
<td>1</td>
</tr>
<tr>
<td>1.3</td>
<td>Tasks need to be investigated</td>
<td>2</td>
</tr>
<tr>
<td>1.3.1</td>
<td>Strain rate effect on consolidation behavior</td>
<td>2</td>
</tr>
<tr>
<td>1.3.2</td>
<td>Anisotropic consolidation behavior</td>
<td>3</td>
</tr>
<tr>
<td>1.3.3</td>
<td>Suitable theory for interpreting CRS test results</td>
<td>3</td>
</tr>
<tr>
<td>1.4</td>
<td>Organization of this thesis</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>LITERATURE REVIEW</td>
<td>5</td>
</tr>
<tr>
<td>2.1</td>
<td>Introduction</td>
<td>5</td>
</tr>
<tr>
<td>2.2</td>
<td>Theoretical study on interpreting CRS test results</td>
<td>5</td>
</tr>
<tr>
<td>2.2.1</td>
<td>Small strain theory</td>
<td>5</td>
</tr>
<tr>
<td>2.2.2</td>
<td>Large strain theory</td>
<td>8</td>
</tr>
<tr>
<td>2.2.3</td>
<td>Summary and comments</td>
<td>10</td>
</tr>
<tr>
<td>2.3</td>
<td>Strain rate effect on consolidation behavior</td>
<td>11</td>
</tr>
<tr>
<td>2.3.1</td>
<td>Strain rate effect on stress-strain curves</td>
<td>11</td>
</tr>
<tr>
<td>2.3.2</td>
<td>Strain rate effect on consolidation yield stress ((p_c))</td>
<td>12</td>
</tr>
<tr>
<td>2.3.3</td>
<td>Relation between the strain rate effect and soil properties</td>
<td>13</td>
</tr>
<tr>
<td>2.3.4</td>
<td>Summary and comments</td>
<td>13</td>
</tr>
<tr>
<td>2.4</td>
<td>Anisotropic consolidation behavior</td>
<td>14</td>
</tr>
<tr>
<td>2.4.1</td>
<td>Anisotropic consolidation behavior investigated by IL test</td>
<td>14</td>
</tr>
<tr>
<td>2.4.2</td>
<td>Anisotropic consolidation behavior investigated by CRS test</td>
<td>15</td>
</tr>
<tr>
<td>2.4.3</td>
<td>Summary and comments</td>
<td>16</td>
</tr>
<tr>
<td>3</td>
<td>LABORATORY EXPERIMENTARY INVESTIGATIONS</td>
<td>17</td>
</tr>
<tr>
<td>3.1</td>
<td>Introduction</td>
<td>17</td>
</tr>
<tr>
<td>3.2</td>
<td>Soil samples</td>
<td>17</td>
</tr>
<tr>
<td>3.3</td>
<td>Test equipment and method</td>
<td>21</td>
</tr>
<tr>
<td>3.3.1</td>
<td>CRS test with vertical drainage</td>
<td>21</td>
</tr>
<tr>
<td>3.3.2</td>
<td>CRS test with radial drainage</td>
<td>24</td>
</tr>
<tr>
<td>CHAPTER</td>
<td>TITLE</td>
<td>PAGE</td>
</tr>
<tr>
<td>---------</td>
<td>-------</td>
<td>------</td>
</tr>
<tr>
<td>3.3.3</td>
<td>IL test</td>
<td>28</td>
</tr>
<tr>
<td>3.4</td>
<td>Cases tested</td>
<td>28</td>
</tr>
<tr>
<td>4</td>
<td>VERTICAL DRAINAGE CRS TEST RESULTS</td>
<td>30</td>
</tr>
<tr>
<td>4.1</td>
<td>Introduction</td>
<td>30</td>
</tr>
<tr>
<td>4.2</td>
<td>Strain rate effect on consolidation yield stress ($p_c$)</td>
<td>30</td>
</tr>
<tr>
<td>4.2.1</td>
<td>Strain rate effect on $p_c$</td>
<td>30</td>
</tr>
<tr>
<td>4.2.2</td>
<td>Stepwise CRS test results</td>
<td>34</td>
</tr>
<tr>
<td>4.3</td>
<td>Strain rate effect on compression index ($C_c$)</td>
<td>37</td>
</tr>
<tr>
<td>4.3.1</td>
<td>Strain rate effect on $C_c$ for a given strain</td>
<td>37</td>
</tr>
<tr>
<td>4.3.2</td>
<td>Strain rate effect on $C_c$ for a given stress</td>
<td>40</td>
</tr>
<tr>
<td>4.4</td>
<td>Strain rate effect on coefficient of consolidation ($c_v$)</td>
<td>41</td>
</tr>
<tr>
<td>4.4.1</td>
<td>Strain rate effect on $c_v$</td>
<td>41</td>
</tr>
<tr>
<td>4.4.2</td>
<td>Strain rate effect on coefficient of volume compressibility ($m_v$)</td>
<td>46</td>
</tr>
<tr>
<td>4.4.3</td>
<td>Strain rate effect on hydraulic conductivity ($k$)</td>
<td>50</td>
</tr>
<tr>
<td>4.5</td>
<td>Comparison of the test results of CRS and IL</td>
<td>56</td>
</tr>
<tr>
<td>4.5.1</td>
<td>Comparing $p_c$ values from CRS and IL test</td>
<td>56</td>
</tr>
<tr>
<td>4.5.2</td>
<td>Comparing $c_v$ values from CRS and IL test</td>
<td>58</td>
</tr>
<tr>
<td>4.6</td>
<td>Effect of clay content on strain rate effect</td>
<td>60</td>
</tr>
<tr>
<td>4.7</td>
<td>Summary</td>
<td>61</td>
</tr>
<tr>
<td>5</td>
<td>RADIAL DRAINAGE CRS TEST RESULTS AND COMPARISONS</td>
<td>63</td>
</tr>
<tr>
<td>5.1</td>
<td>Introduction</td>
<td>63</td>
</tr>
<tr>
<td>5.2</td>
<td>Consolidation yield stress ($p_{cv}$ and $p_{ch}$)</td>
<td>63</td>
</tr>
<tr>
<td>5.3</td>
<td>Coefficient of consolidation ($c_v$, $c_{ch}$ and $c_{hv}$)</td>
<td>67</td>
</tr>
<tr>
<td>5.4</td>
<td>Coefficient of volume compressibility ($m_v$ and $m_h$)</td>
<td>70</td>
</tr>
<tr>
<td>5.5</td>
<td>Hydraulic conductivity ($k_v$ and $k_h$)</td>
<td>72</td>
</tr>
<tr>
<td>5.6</td>
<td>Excess pore water pressure ($u_b$ and $u_e$)</td>
<td>75</td>
</tr>
<tr>
<td>5.7</td>
<td>Strain rate effect on horizontal consolidation characteristics</td>
<td>78</td>
</tr>
<tr>
<td>5.7.1</td>
<td>Strain rate effect on $p_{ch}$</td>
<td>78</td>
</tr>
<tr>
<td>5.7.2</td>
<td>Strain rate effect on $c_{h}$ and $k_{h}$</td>
<td>78</td>
</tr>
<tr>
<td>5.8</td>
<td>Summary</td>
<td>79</td>
</tr>
<tr>
<td>6</td>
<td>NUMERICAL INVESTIGATIONS AND DISCUSSIONS</td>
<td>81</td>
</tr>
<tr>
<td>6.1</td>
<td>Introduction</td>
<td>81</td>
</tr>
<tr>
<td>6.2</td>
<td>Strain distribution within the sample of CRS test</td>
<td>81</td>
</tr>
<tr>
<td>6.2.1</td>
<td>Small strain theory</td>
<td>81</td>
</tr>
<tr>
<td>6.2.2</td>
<td>Large strain theory</td>
<td>83</td>
</tr>
<tr>
<td>6.2.3</td>
<td>Comparison of the theoretical strain distributions with numerical results</td>
<td>85</td>
</tr>
<tr>
<td>CHAPTER</td>
<td>TITLE</td>
<td>PAGE</td>
</tr>
<tr>
<td>---------</td>
<td>-------</td>
<td>------</td>
</tr>
<tr>
<td>6.3</td>
<td>Comparing interpreted $c_v$ values from CRS test</td>
<td>89</td>
</tr>
<tr>
<td>6.3.1</td>
<td>Comparing interpreted $c_v$ values</td>
<td>89</td>
</tr>
<tr>
<td>6.3.2</td>
<td>Comparing interpreted strain rate effect on $c_v$</td>
<td>92</td>
</tr>
<tr>
<td>6.4</td>
<td>Summary</td>
<td>94</td>
</tr>
<tr>
<td>7</td>
<td>CONCLUDING REMARKS</td>
<td>95</td>
</tr>
<tr>
<td>7.1</td>
<td>Conclusions</td>
<td>95</td>
</tr>
<tr>
<td>7.2</td>
<td>Future works</td>
<td>96</td>
</tr>
<tr>
<td></td>
<td>REFERENCE</td>
<td>97</td>
</tr>
<tr>
<td>FIGURE NO.</td>
<td>TITLE</td>
<td>PAGE</td>
</tr>
<tr>
<td>-----------</td>
<td>----------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>2.1</td>
<td>Strain distribution within the sample (Smith and Wahls method)</td>
<td>6</td>
</tr>
<tr>
<td>2.2</td>
<td>Strain distribution within the sample (Wissa et al.'s method)</td>
<td>7</td>
</tr>
<tr>
<td>2.3</td>
<td>Strain distribution within the sample (Umehara and Zen method)</td>
<td>9</td>
</tr>
<tr>
<td>2.4</td>
<td>Strain distribution within the sample (Lee method)</td>
<td>10</td>
</tr>
<tr>
<td>3.1</td>
<td>Borehole locations</td>
<td>17</td>
</tr>
<tr>
<td>3.2</td>
<td>Sample preparation process for testing</td>
<td>18</td>
</tr>
<tr>
<td>3.3</td>
<td>Soil profile and some index properties</td>
<td>21</td>
</tr>
<tr>
<td>3.4</td>
<td>Illustration of the CRS test with vertical drainage device</td>
<td>22</td>
</tr>
<tr>
<td>3.5bis</td>
<td>Variations of $\varepsilon_B / \varepsilon_T$</td>
<td>23</td>
</tr>
<tr>
<td>3.6</td>
<td>Illustration of the CRS test with radial drainage device</td>
<td>25</td>
</tr>
<tr>
<td>3.7</td>
<td>Photograph of the cylindrical control device</td>
<td>26</td>
</tr>
<tr>
<td>3.8</td>
<td>Water flow through a cylindrical surface</td>
<td>28</td>
</tr>
<tr>
<td>3.9</td>
<td>Loading and drainage condition for three types of CRS test</td>
<td>29</td>
</tr>
<tr>
<td>4.1</td>
<td>Stress-strain rate relation (BH-1, 5-5.9 m)</td>
<td>30</td>
</tr>
<tr>
<td>4.2</td>
<td>Stress-strain-rate relation (BH-1, 6-6.9 m)</td>
<td>31</td>
</tr>
<tr>
<td>4.3</td>
<td>Stress-strain-rate relation (BH-2, 3-3.85 m)</td>
<td>31</td>
</tr>
<tr>
<td>4.4</td>
<td>Stress-strain-rate relation (BH-3, 6-6.9 m)</td>
<td>31</td>
</tr>
<tr>
<td>4.5</td>
<td>Stress-strain-rate relation (BH-4, 2-2.9 m)</td>
<td>32</td>
</tr>
<tr>
<td>4.6</td>
<td>Stress-strain-rate relation (BH-4, 9-9.9 m)</td>
<td>32</td>
</tr>
<tr>
<td>4.7</td>
<td>Stress-strain-rate relation (BH-5, 9-9.8 m)</td>
<td>32</td>
</tr>
<tr>
<td>4.8</td>
<td>Stress-strain-rate relation (BH-6, 17-17.8 m)</td>
<td>33</td>
</tr>
<tr>
<td>4.9</td>
<td>Stress-strain-rate relation (BH-7, 9-9.85 m)</td>
<td>33</td>
</tr>
<tr>
<td>4.10</td>
<td>Stress-strain-rate relation (BH-7, 14-14.75 m)</td>
<td>33</td>
</tr>
<tr>
<td>4.11</td>
<td>RPC-normalised strain rate relation</td>
<td>34</td>
</tr>
<tr>
<td>4.12</td>
<td>Stepwise CRS stress-strain-rate relation (BH-1, 5-5.9m)</td>
<td>35</td>
</tr>
<tr>
<td>4.13</td>
<td>Stepwise CRS stress-strain-rate relation (BH-2, 6-6.9m)</td>
<td>35</td>
</tr>
<tr>
<td>4.14</td>
<td>Stepwise CRS stress-strain-rate relation (BH-3, 8-8.9m)</td>
<td>35</td>
</tr>
<tr>
<td>4.15</td>
<td>Stepwise CRS stress-strain-rate relation (BH-4, 8-8.9m)</td>
<td>36</td>
</tr>
<tr>
<td>4.16</td>
<td>Comparison of SRE</td>
<td>37</td>
</tr>
<tr>
<td>4.17</td>
<td>$C_t$ and $C_c$-strain-rate relation (BH-1, 5-5.9m)</td>
<td>38</td>
</tr>
<tr>
<td>4.18</td>
<td>$C_t$ and $C_c$-strain-rate relation (BH-1, 6-6.9m)</td>
<td>38</td>
</tr>
<tr>
<td>4.19</td>
<td>$C_t$ and $C_c$-strain-rate relation (BH-3, 6-6.9m)</td>
<td>38</td>
</tr>
<tr>
<td>4.20</td>
<td>$C_t$ and $C_c$-strain-rate relation (BH-4, 8-8.9m)</td>
<td>39</td>
</tr>
<tr>
<td>4.21</td>
<td>$C_t$ and $C_c$-strain-rate relation (BH-6, 17-17.8m)</td>
<td>39</td>
</tr>
<tr>
<td>4.22</td>
<td>$C_t$ and $C_c$-strain-rate relation (BH-7, 14-14.75m)</td>
<td>39</td>
</tr>
<tr>
<td>4.23</td>
<td>$C_t$ and $C_c$-stress-rate relation (BH-1, 5-5.9m)</td>
<td>40</td>
</tr>
<tr>
<td>4.24</td>
<td>$C_t$ and $C_c$-stress-rate relation (BH-1, 6-6.9m)</td>
<td>40</td>
</tr>
<tr>
<td>4.25</td>
<td>$C_t$ and $C_c$-stress-rate relation (BH-4, 8-8.9m)</td>
<td>41</td>
</tr>
<tr>
<td>4.26</td>
<td>$C_t$ and $C_c$-stress-rate relation (BH-6, 17-17.8m)</td>
<td>41</td>
</tr>
</tbody>
</table>
4.27  $c_v$-stress-strain rate relation (BH-1, 5-5.9 m)  \hspace{1cm} 42
4.28  $c_v$-stress-strain rate relation (BH-1, 6-6.9 m)  \hspace{1cm} 42
4.29  $c_v$-stress-strain rate relation (BH-2, 3-3.85 m)  \hspace{1cm} 42
4.30  $c_v$-stress-strain rate relation (BH-3, 6-6.9 m)  \hspace{1cm} 43
4.31  $c_v$-stress-strain rate relation (BH-4, 2-2.9 m)  \hspace{1cm} 43
4.32  $c_v$-stress-strain rate relation (BH-4, 9-9.9 m)  \hspace{1cm} 43
4.33  $c_v$-stress-strain rate relation (BH-5, 9-9.8 m)  \hspace{1cm} 44
4.34  $c_v$-stress-strain rate relation (BH-6, 17-17.8 m)  \hspace{1cm} 44
4.35  $c_v$-stress-strain rate relation (BH-7, 9-9.85 m)  \hspace{1cm} 44
4.36  $c_v$-stress-strain rate relation (BH-7, 14-14.75 m)  \hspace{1cm} 45
4.37bis  Strain rate effect on $c_v$  \hspace{1cm} 45
4.38  $m_v$-stress-strain rate relation (BH-1 5-5.9 m)  \hspace{1cm} 46
4.39  $m_v$-stress-strain rate relation (BH-1 6-6.9 m)  \hspace{1cm} 46
4.40  $m_v$-stress-strain rate relation (BH-2 3-3.85 m)  \hspace{1cm} 47
4.41  $m_v$-stress-strain rate relation (BH-3 6-6.9 m)  \hspace{1cm} 47
4.42  $m_v$-stress-strain rate relation (BH-4 2-2.9 m)  \hspace{1cm} 47
4.43  $m_v$-stress-strain rate relation (BH-4 9-9.9 m)  \hspace{1cm} 48
4.44  $m_v$-stress-strain rate relation (BH-5 9-9.8 m)  \hspace{1cm} 48
4.45  $m_v$-stress-strain rate relation (BH-6 17-17.8 m)  \hspace{1cm} 48
4.46  $m_v$-stress-strain rate relation (BH-7 9-9.85 m)  \hspace{1cm} 49
4.47  $m_v$-stress-strain rate relation (BH-7 14-14.75 m)  \hspace{1cm} 49
4.48  Effect of strain rate on $C_c$ and $e$ under a given $\sigma_v'$  \hspace{1cm} 50
4.49  $k$-stress-strain rate relation (BH-1 5-5.9 m)  \hspace{1cm} 50
4.50  $k$-stress-strain rate relation (BH-1 6-6.9 m)  \hspace{1cm} 51
4.51  $k$-stress-strain rate relation (BH-2 3-3.85 m)  \hspace{1cm} 51
4.52  $k$-stress-strain rate relation (BH-3 6-6.9 m)  \hspace{1cm} 51
4.53  $k$-stress-strain rate relation (BH-4 2-2.9 m)  \hspace{1cm} 52
4.54  $k$-stress-strain rate relation (BH-4 9-9.9 m)  \hspace{1cm} 52
4.55  $k$-stress-strain rate relation (BH-5 9-9.8 m)  \hspace{1cm} 52
4.56  $k$-stress-strain rate relation (BH-6 17-17.8 m)  \hspace{1cm} 53
4.57  $k$-stress-strain rate relation (BH-7 9-9.85 m)  \hspace{1cm} 53
4.58  $k$-stress-strain rate relation (BH-7 14-14.75 m)  \hspace{1cm} 53
4.59  Strain rate effect on $k$  \hspace{1cm} 54
4.60  Flow rate-gradient relationship in 45.9% Li-clay paste  \hspace{1cm} 55
(\hspace{1cm} data from Miller and Low, 1963)  \hspace{1cm} 55
4.61  Excess pore water pressure variation (BH-1, 5-5.9 m)  \hspace{1cm} 55
4.62  Average hydraulic gradient-strain rate relation (BH-1, 5-5.9 m)  \hspace{1cm} 56
4.63  Comparing $p_c$ values from CRS and IL tests (BH-1)  \hspace{1cm} 56
4.64  Comparing $p_c$ values from CRS and IL tests (BH-2)  \hspace{1cm} 57
4.65  Comparing $p_c$ values from CRS and IL tests (BH-3)  \hspace{1cm} 57
4.66  Comparing $C_r$ and $C_c$ from CRS and IL tests (BH-1, 5-5.9 m)  \hspace{1cm} 58
4.67  Comparing $c_v$ values from CRS and IL tests (BH-1, 5-5.9 m)  \hspace{1cm} 58
4.68  Comparing $c_v$ values from CRS and IL tests (BH-1, 12-12.9 m)  \hspace{1cm} 59
4.69  Comparing $c_v$ values from CRS and IL tests (BH-1, 14-14.9 m)  \hspace{1cm} 59
4.70 Average strain rate of IL test up to 90% degree of consolidation (BH-1, 5-5.9 m) 60
4.71 Relation between SRE and clay content (BH-4, sw-LH test) 61
4.72 Relation between SRE and clay content (BH-4, sw-HL test) 61
5.1 Friction of O-ring (0.02%/min-0.2%/min) 64
5.2 Friction of O-ring (0.2%/min-0.02%/min) 64
5.3 Comparison of stress-strain relation (BH-5 15.5-16.3 m) 65
5.4 Comparison of stress-strain relation (BH-6 17-17.8 m) 65
5.5 Comparison of stress-strain relation (BH-7 9-9.85 m) 65
5.6 Comparison of stress-strain relation (BH-7 14-14.75 m) 65
5.7 The ratio of $p_{ch}$ to $p_{cv}$ from CRS-V-V test 67
5.8 The ratio of $p_{cv}$ from CRS-V-R test to $p_{cv}$ from CRS-V-V test 67
5.9 Comparison of $c_v$, $c_{hh}$ and $c_{hv}$ (BH-5 15.5-16.3 m) 68
5.10 Comparison of $c_v$, $c_{hh}$ and $c_{hv}$ (BH-6 17-17.8 m) 68
5.11 Comparison of $c_v$, $c_{hh}$ and $c_{hv}$ (BH-7 9-9.85 m) 68
5.12 Comparison of $c_v$, $c_{hh}$ and $c_{hv}$ (BH-7 14-14.75 m) 69
5.13 The ratio of $c_{hh}$ to $c_v$ 69
5.14 The ratio of $c_{hv}$ to $c_v$ 70
5.15 Comparison of $m_v$ and $m_h$ (BH-5 15.5-16.3 m) 70
5.16 Comparison of $m_v$ and $m_h$ (BH-6 17-17.8 m) 71
5.17 Comparison of $m_v$ and $m_h$ (BH-7 9-9.85 m) 71
5.18 Comparison of $m_v$ and $m_h$ (BH-7 14-14.75 m) 71
5.19 Comparison of $k_v$ and $k_h$ (BH-5 15.5-16.3 m) 72
5.20 Comparison of $k_v$ and $k_h$ (BH-6 17-17.8 m) 72
5.21 Comparison of $k_v$ and $k_h$ (BH-7 9-9.85 m) 73
5.22 Comparison of $k_v$ and $k_h$ (BH-7 14-14.75 m) 73
5.23 The ratio of $k_h$ from CRS-H-V test to $k_v$ from CRS-V-V test 74
5.24 The ratio of $k_h$ from CRS-V-R test to $k_v$ from CRS-V-V test 74
5.25 Variations of $k_h/k_v$ with depth 75
5.26 Comparison of $u_b$ and $u_e$ (BH-5 15.5-16.3 m) 76
5.27 Comparison of $u_b$ and $u_e$ (BH-6 17-17.8 m) 76
5.28 Comparison of $u_b$ and $u_e$ (BH-7 9-9.85 m) 77
5.29 Comparison of $u_b$ and $u_e$ (BH-7 14-14.75 m) 77
5.30 The ratio of $u_e$ from CRS-V-R test to $u_b$ from CRS-V-V test 77
5.31 Strain rate effect on $p_{ch}$ 78
5.32 Strain rate effect on $c_h$ 78
5.33 Strain rate effect on $k_h$ 79
6.1 Strain distributions (small strain theory $c_v/rH^2 = 100$) 82
6.2 Strain distributions (small strain theory $c_v/rH^2 = 10$) 82
6.3 Strain distributions (small strain theory $c_v/rH^2 = 1$) 82
6.4 Strain distributions (large strain theory $c_v/rH_0^2 = 100$) 84
6.5 Strain distributions (large strain theory $c_v/rH_0^2 = 10$) 84
6.6 Strain distributions (large strain theory $c_v/rH_0^2 = 1$) 84
6.7 Strain increment distributions (large strain theory) 85
<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.8</td>
<td>Mesh used in the FEM simulation</td>
</tr>
<tr>
<td>6.9</td>
<td>Comparison of the stress-strain curves</td>
</tr>
<tr>
<td>6.10</td>
<td>Comparison of excess pore water pressure variation</td>
</tr>
<tr>
<td>6.11</td>
<td>Comparison of strain distributions ($c_v/rH_0^2 = 100$)</td>
</tr>
<tr>
<td>6.12</td>
<td>Comparison of strain distributions ($c_v/rH_0^2 = 10$)</td>
</tr>
<tr>
<td>6.13</td>
<td>Comparison of strain distributions ($c_v/rH_0^2 = 1$)</td>
</tr>
<tr>
<td>6.14</td>
<td>Comparison of $c_v$ (BH-1 5-5.9 m, 0.02%/min)</td>
</tr>
<tr>
<td>6.15</td>
<td>Comparison of $c_v$ (BH-1 6-6.9 m, 0.02%/min)</td>
</tr>
<tr>
<td>6.16</td>
<td>Comparison of $c_v$ (BH-1 5-5.9 m, 0.2%/min)</td>
</tr>
<tr>
<td>6.17</td>
<td>Comparison of $c_v$ (BH-1 6-6.9 m, 0.2%/min)</td>
</tr>
<tr>
<td>6.18</td>
<td>Variations of $\varepsilon_B/\varepsilon_T$ (large strain theory) (Fig. 3.5bis)</td>
</tr>
<tr>
<td>6.19</td>
<td>Variations of $\varepsilon_B/\varepsilon_T$ (small strain theory)</td>
</tr>
<tr>
<td>6.20</td>
<td>Comparison of the $c_v$ (0.02%/min)</td>
</tr>
<tr>
<td>6.21</td>
<td>Comparison of the $c_v$ (0.2%/min)</td>
</tr>
<tr>
<td>6.22</td>
<td>Comparison of the strain rate effect on $c_v$</td>
</tr>
<tr>
<td>6.23</td>
<td>Strain rate effect on $c_v$ (large strain theory)</td>
</tr>
<tr>
<td>6.24</td>
<td>Strain rate effect on $c_v$ (small strain theory) (Fig. 4.37bis)</td>
</tr>
</tbody>
</table>
# 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>Cases tested</td>
<td>29</td>
</tr>
<tr>
<td>6.1</td>
<td>Parameters for finite element analysis</td>
<td>86</td>
</tr>
</tbody>
</table>
LIST OF NOTATIONS

\( b \) a constant that depends on the variation in void ratio with depth and time

\( BH \) borehole

\( C_c \) compression index

\( C_r \) recompression index

\( c_h \) coefficient of consolidation in the horizontal direction

\( c_{nh} \) coefficient of consolidation from CRS-H-V test

\( c_{hv} \) coefficient of consolidation from CRS-V-R test

\( c_k \) a constant, which can be estimated as 0.4\( e_0 \)

\( c_v \) coefficient of consolidation in the vertical direction

\( CRS \) constant rate of strain

\( CRS-H-V \) CRS test using horizontally cut sample with vertical drainage

\( CRS-V-R \) CRS test using vertically cut sample with radial drainage

\( CRS-V-V \) CRS test using vertically cut sample with vertical drainage

\( e \) void ratio

\( e_{av} \) average void ratio of the sample

\( e_c \) a constant and depends on soil type

\( e_0 \) initial void ratio

\( e_t \) void ratio of the sample at time \( t \)

\( e_{t+\Delta t} \) void ratio of the sample at time \( t + \Delta t \)

\( f \) specific volume

\( H \) thickness of the sample

\( H_0 \) initial thickness of the sample

\( \Delta H \) change of sample thickness in a time interval of \( \Delta t \)

\( H_t \) thickness of the sample at time \( t \)

\( H_{t+\Delta t} \) thickness of the sample at time \( t + \Delta t \)

\( \bar{H} \) average thickness of the sample between \( t \) and \( t + \Delta t \)

\( i \) hydraulic gradient

\( i_{av} \) average hydraulic gradient

\( i_c \) threshold hydraulic gradient

\( I_p \) plasticity index

\( k \) hydraulic conductivity

\( k_0 \) initial hydraulic conductivity

\( k_h \) hydraulic conductivity in the horizontal direction

\( k_v \) hydraulic conductivity in the vertical direction

\( K_0 \) coefficient of earth pressure at rest

\( K_{0(oc)} \) \( K_0 \) value for overconsolidated state

\( m_h \) coefficient of volume compressibility in the horizontal direction

\( m_v \) coefficient of volume compressibility in the vertical direction

\( n \) porosity

\( n \) ratio of radius of specimen to radius of central drain

\( n \) dimensionless parameter
OCR  over consolidation ratio

\( p_{cv} \)  consolidation yield stress in the vertical direction

\( p_{ch} \)  consolidation yield stress in the horizontal direction

\( p_{cc} \)  consolidation yield stress for a given strain rate of \( \dot{\varepsilon} \)

\( p_{c0.02%/min} \) consolidation yield stress under the strain rate of 0.02%/min

PVD  prefabricated vertical drain

\( p' \)  consolidation stress

\( p'y \)  size of the yield locus

\( q \)  flow rate

\( q' \)  deviator

\( r \)  rate of strain

\( r_e \)  the rate of change of the average void ratio

\( r_e \)  radius of specimen

\( r_w \)  radius of central drain

\( R \)  the rate of deformation

RPC  ratio of the consolidation yield stress

SRE  strain rate effect

\( t \)  time

\( T' \)  dimensionless parameter

\( T_v \)  time factor

\( T_h \)  time factor of radial drainage

\( u \)  excess pore water pressure

\( \bar{u}_0 \)  initial average excess pore water pressure

\( u_b \)  excess pore water pressure at the bottom of the sample

\( u_{b,t} \)  excess pore water pressure at the bottom of the sample at time \( t \)

\( u_{b,t+\Delta t} \)  excess pore water pressure at the bottom of the sample at time \( t + \Delta t \)

\( \bar{u}_b \)  average excess pore water pressure at the bottom of the sample between \( t \) and \( t + \Delta t \)

\( u_e \)  excess pore water pressure at the periphery of the sample

\( z \)  the vertical coordinate of a point

\( z_0 \)  original coordinate

\( Z_0 \)  dimensionless parameter

\( v_p \)  vertical displacement rate

\( w_L \)  liquid limit of sample

\( w_n \)  natural water content of sample

\( w_p \)  plastic limit of sample

\( \beta \)  a dimensionless parameter (normalized strain rate)

\( \gamma_w \)  unit weight of water

\( \varepsilon \)  strain

\( \varepsilon_{ave} \)  average strain of sample

\( \varepsilon_B / \varepsilon_T \)  ratio of the strain at the bottom to that at the top of a sample

\( \dot{\varepsilon} \)  strain rate

\( \bar{\varepsilon} \)  average strain of the sample

\( \sigma_v \)  total vertical stress
\( \Delta \sigma_v \) total vertical stress increment in a time interval of \( \Delta t \)

\( \sigma'_v \) effective vertical stress

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

\( \Delta \sigma'_v \) effective vertical stress increment in a time interval of \( \Delta t \)

\( \sigma'_{v,t} \) effective vertical stress at time \( t \)

\( \sigma'_{v,t+\Delta t} \) effective vertical stress at time \( t + \Delta t \)

\( \sigma'_{v0.2\%/min} \) effective vertical stress corresponding the strain rate of 0.2\%/min

\( \sigma'_{v0.02\%/min} \) effective vertical stress corresponding the strain rate of 0.02\%/min

\( \bar{\sigma}'_v \) average effective vertical stress

\( \zeta \) consolidation ratio

\( \bar{\zeta} \) average value of \( \zeta \) within a time increment \( \Delta t \).

\( \varnothing' \) angle of internal friction

\( \nu \) poisson’s ratio

\( \kappa \) slope of unloading-reloading line in \( e - \ln p' \) plot

\( \kappa_1 \) slope of unloading-reloading line in \( \ln(e + e_c) - \ln p' \) plot

\( \lambda \) slope of virgin consolidation line in \( e - \ln p' \) plot

\( \lambda_1 \) slope of virgin consolidation line in \( \ln(e + e_c) - \ln p' \) plot

\( M \) slope of critical state line in \( q - p' \) plot
1 INTRODUCTION

1.1 Background

For convenience of sea transport, many big cities in the world are located on clayey deposit around coast area, thus large number of structures built on soft soils. For civil engineering projects related to soft clayey deposits, engineers inevitably have to consider the consolidation behavior of the deposit.

Because to the natural viscosity of clayey soils, the consolidation behavior of such soils is strongly influenced by strain rate, and therefore strain rate effect has a strong engineering impact, e.g. construction speed and the mobilized strength. An understanding of the strain rate effect on consolidation behaviour is useful for designing geotechnical projects such as embankments, foundations in areas of clayey deposits.

For sedimentary clay, consolidation properties in the horizontal direction are usually different from the consolidation properties in the vertical direction due to the anisotropic nature of soil. During the last decade, the prefabricated vertical drain (PVD) has become one of the most popular ground improvement techniques used to treat the soft clay. The design of PVD ground improvement requires consolidation characteristics in both the vertical and the horizontal directions. An understanding of the anisotropic consolidation behavior under both vertical and horizontal drainage conditions is useful for designing vertical drain improvement.

In geotechnical practice there are two test methods for determining the consolidation characteristics of a clayey soil: namely incremental loading (IL) and constant rate of strain (CRS) consolidation tests. Due to IL test is more convenient to be conducted and the equipment is relatively simple, it is extensively used for geotechnical project related to clayey deposit. However, the strain rate effect on consolidation behavior can only be investigated by the CRS test. Also, in comparison with the IL test, the CRS test has several merits, for example, the test duration can be substantially reduced, and it provides continuous data points for a plot of void ratio ($e$) against effective vertical stress ($\sigma_v'$), which can increase the accuracy of determined consolidation yield stress ($p_c$). So the topic of this study is consolidation behavior under constant rate of strain.

1.2 Objective and scope of research

(1) Strain rate effect on the consolidation behavior of Ariake clay

Although some constant rate of strain CRS test results using undisturbed Ariake clay samples have been reported (Chai et al., 2006; Tanaka et al., 2000, 2006), there has been no systematic study of the strain rate effect on the consolidation behaviour of Ariake clay. In this study, systematically experimented investigation on the strain rate effect will be conducted using both CRS tests and incremental loading (IL) tests in term of consolidation yield stress ($p_c$), compression index ($C_v$), coefficient of consolidation ($c_v$), coefficient of volume compressibility ($m_v$) and hydraulic conductivity ($k$). Further, the degree of the strain rate effect and the clay content will be studied using the same soil samples for CRS
test and grain size distribution test. Also by comparing the results of IL tests with that of CRS tests, to identify at what kind of strain rate, the consolidation characteristics ($p_c$ and $c_v$) obtained from CRS tests will be comparable with those from IL tests.

(2) Anisotropic consolidation behavior of Ariake clay

There are only a few researches on the anisotropic consolidation behavior of Ariake clay. Park (1994) conducted the IL tests for vertically and horizontally cut undisturbed sample to study the anisotropic consolidation behavior of Ariake clay. For Ariake clay, there has been no systematic study about the anisotropic behavior of consolidation. The anisotropic consolidation behaviors of Ariake clay will be systematically investigated through the comparisons of: (1) the CRS test results of radial drainage with that of vertical drainage; and (2) vertically and horizontally cut samples under vertical drainage.

(3) Suitable theory for interpreting CRS test results

There are two theories for interpreting CRS test results, i.e. small strain theory (Smith and Wahls, 1969; Aboshi et al., 1970; Wissa et al., 1971) and large strain theory (Umehara and Zen, 1980; Lee, 1981; Znidarcic et al., 1986). Small strain theory is simple and is adopted into the standard for CRS test (JSA, 2000b; ASTM, 2006). However, under a consolidation pressure of few hundreds kPa or even 1000 kPa, for a soft clay sample with an initial water content more than 100%, the strain can be as large as 30%. This kind of strain cannot be regarded as a small strain and large strain theory will be much more appropriate. Therefore, it is needed to clarify that under what kind condition the small strain theory can yield an acceptable result, and for what kind of condition, the large strain theory may result in a better or more realistic result. The strain distribution within the sample of CRS test obtained by small strain theory and large strain theory will be compared with the results of numerical simulations. The coefficient of consolidation ($c_v$) and strain rate effect on $c_v$ calculated by large strain theory will be compared with that of small strain theory. The applicable range of small strain theory will be suggested.

1.3 Tasks need to be investigated

1.3.1 Strain rate effect on consolidation behavior

(1) Strain rate effect on stress-strain curves and $p_c$ of Ariake clay

Although some CRS test results using undisturbed Ariake clay samples have been reported (Chai et al., 2006; Tanaka et al., 2000, 2006), there has been no systematic study of strain rate effect on stress-strain curves and $p_c$ of Ariake clay. The strain rate effect on stress-strain curves and $p_c$ of Ariake clay will be systematically investigated in this study. By comparing the results of IL tests with that of CRS tests, the strain rate for a routine CRS test will be investigated.

(2) Strain rate effect on coefficient of consolidation ($c_v$)

Regarding the coefficient of consolidation ($c_v$), there are several theoretical studies on obtaining the value of $c_v$ from the CRS test (Aboshi et al., 1970; Lee, 1981; Smith and Wahls, 1969; Umehara and Zen, 1980; Wissa et al., 1971; Znidarcic et al., 1986), but there is no generally accepted conclusion regarding the effect of strain rate on $c_v$. The strain rate effect on $c_v$ and $k$ will be systematically investigated in this study.

(3) The degree of strain rate effect and clay content
The strain rate dependent behavior is due to the viscosity natural of clayey soils. Fundamentally the strain rate effect should be related to clay content and clay mineral of a soil. There are also researches on the relationship between the degree of strain rate effect and the clay content and plasticity index \( I_p \) of clayey soils (Graham et al., 1983; Tanaka et al., 2000), but there is no definite conclusion on this aspect. It is considered that one of the reasons for this kind of situation is that in most laboratory test program, the samples for CRS consolidation test and grain size distribution test are different. The tested samples were used for grain size distribution tests, and the relationship between the strain rate effect and clay content will be studied.

### 1.3.2 Anisotropic consolidation behavior

(1) Consolidation yield stress in horizontal direction \( (p_{ch}) \)

There has been no systematic study of consolidation yield stress in horizontal direction \( (p_{ch}) \) using CRS test. The \( p_{ch} \) will be investigated and compared with \( p_{cv} \) through CRS test using vertically and horizontally cut samples under vertical drainage.

(2) Coefficient of consolidation in horizontal direction \( (c_h) \)

Although the ratio of \( c_h/c_v \) have been reported through the comparison of CRS test with radial drainage and vertical drainage (Yune and Chung, 2005; Sea and Koslanant, 2003), there has been no study of \( c_h \) of Ariake clay through CRS test. The \( c_h \) of Ariake clay will be systematically investigated and compared with \( c_v \) through CRS test results of radial drainage with that of vertical drainage, and vertically and horizontally cut samples under vertical drainage.

### 1.3.3 Suitable theory for interpreting CRS test results

(1) Comparison of theoretical strain distributions with numerical results

Although there are several theoretical studies about strain distribution within the sample of CRS test and interpreting CRS test results (Aboshi et al., 1970; Lee, 1981; Smith and Wahls, 1969; Umehara and Zen, 1980; Wissa et al., 1971; Znidarcic et al., 1986). There has been no study of comparison of theoretical strain distributions with numerical results. The strain distribution pattern within the sample of CRS test will be investigated numerically and the results will be compared with the strain distribution pattern obtained by various theories.

(2) Effect of strain distribution pattern on interpreting CRS test results

There has been no study of comparison of CRS test results interpreted by different theories. The \( c_v \) and strain rate effect on \( c_v \) calculated by large strain theory will be compared with that of small strain theory. The applicable range of small strain theory will be suggested.

### 1.4 Organization of this thesis

This dissertation contains seven chapters. Following the introductory chapter (Chapter 1) which describes the background, objective and scope of the work, and tasks need to be investigated, Chapter 2 reviews the literature about theoretical study on interpreting CRS test results, strain rate effect on consolidation behavior and anisotropic consolidation
behavior. Chapter 3 describes relevant details of the experimental investigations (the soil samples, test equipments, test methods and the result interpretation) carried out in this study. Chapter 4 presents the strain rate effect on consolidation behavior of Ariake clay based on the vertical drainage CRS test results. Chapter 5 investigates the anisotropic consolidation behavior of Ariake clay based on the radial drainage CRS test results and comparisons with the vertical drainage CRS test results. Chapter 6 investigate the effect of strain distribution pattern on interpreting CRS test result through the comparison of the strain distribution within the sample of CRS test obtained by small strain theory, large strain theory and numerical simulations. Finally, the conclusions drawn from this study and recommendations for future works are given in Chapter 7.
2 LITERATURE REVIEW

2.1 Introduction

Hamilton and Crawford (1959) and Crawford (1964) reported using constant rate of strain (CRS) consolidation tests for speeding up the testing process and for testing material under strain rates that more closely approximate the field strain rates, which is considered as one of the pioneer works in this area. Subsequently, there are several theoretical studies on interpreting the CRS test results, and numerous CRS test were performed to investigate the effects of strain rate on the stress-strain curve of clays and consolidation yield stress ($p_c$). Recently, some CRS tests with radial drainage test have been reported for investigating the anisotropic consolidation behavior of clay. This literature review mainly contains three parts: (1) theories for interpreting CRS test results; (2) strain rate effect on consolidation behavior; and (3) anisotropic consolidation behavior. Comments are given for the areas more researches are needed.

2.2 Theories for interpreting CRS test results

To make a successful prediction of consolidation process requires that both the theory used to model the field problem and the material properties used by the theory must be appropriate and suitable. Thus the testing procedure must provide reliable and consistent information on the material behavior. Further, the test procedure must be accompanied by methods of analysis that will deduce values of the material properties. There are several theoretical studies about strain distribution within the sample of CRS test and interpreting CRS test results. The existing theories can be divided into small strain theory and large strain theory according to whether considering the change of the thickness of the sample during a time interval.

2.2.1 Small strain theory

(1) Smith and Wahls method

Smith and Wahls (1969) developed the governing equation for CRS test based on the continuity of flow through a soil element as follows:

$$\frac{\partial}{\partial z} \left( \frac{k}{\gamma_w} \frac{\partial u}{\partial z} \right) = \frac{1}{1+e} \frac{\partial e}{\partial t}$$

(2.1)

where $z$ = the vertical coordinate of a point, $k$ = hydraulic conductivity, $e$ = void ratio, $\gamma_w$ = unit weight of water, $u$ = excess pore water pressure and $t$ = time. The $e$ is assumed as a linear function of time and space variable as follows:

$$e = e_0 - r_e \left[ 1 - \frac{b}{r_e} \left( \frac{z - 0.5H}{H} \right) \right]$$

(2.2)

where $e_0$ = initial void ratio, $r_e$ = the rate of change of the average void ratio, $H$ = thickness of the sample; $b$ = a constant that depends on the variation in void ratio with depth and time. By assuming that the term $(1+e)$ in Eq. (2.1) can be replaced by $(1+e_{av})$, where $e_{av}$ is...
not a function of $z$, and introducing the boundary conditions $u(0,t) = 0$ and $\partial u / \partial z (H,t) = 0$, the solution to Eq. (2.1) is obtained in the form as follows:

$$u = \frac{\gamma^e r_e}{k(1 + e_w)} \left[ \left( Hz - \frac{z^2}{2} \right) - \frac{b}{r_e} \left( \frac{z^2}{4} - \frac{z^3}{6H} \right) \right]$$

(2.3)

An average effective vertical stress ($\sigma'_v$), and an average $k$ can be calculated as:

$$\sigma'_v = \sigma_v - \left[ \frac{1/3 - 1/24(b/r_e)}{1/2 - 1/12(b/r_e)} \right] u_v$$

(2.4)

$$k = \frac{\gamma^e r_e H^2}{(1 + e_w) u_v} \left[ \frac{1}{2} - \frac{1}{12} \left( \frac{b}{r_e} \right) \right]$$

(2.5)

By assigning these values to the average void ratio, the effective vertical stress-void ratio and void ratio-permeability relationships are obtained.

The strain distribution within the sample is shown in Fig. 2.1.

![Fig. 2.1 Strain distribution within the sample (Smith and Wahls method)](image)

This theory has two major problems. First, the correctness or reliability of the assumption that the void ratio is a linear function of the time and space variables is difficult to be evaluated, and therefore the accuracy of the obtained material properties is not known. The second problem is that the parameter $b$ is not known, and there is no procedure for its determination. Since the resulting material characteristics depend on the chosen value of $b$, one cannot ascertain which value to use unless some other reference tests are performed on similar specimens.

(2) Wissa et al.’s method

The analysis (Wissa et al., 1971) starts from the governing equation for consolidation written in terms of strains as follows:

$$\frac{\partial \varepsilon}{\partial t} = c_v \frac{\partial ^2 \varepsilon}{\partial z^2}$$

(2.6)
Wissa et al. (1971) obtained the solution for Eq. (2.6) as follows:

$$
\varepsilon(z,t) = rt + \frac{rH^2}{6c_v} \left( 3 - \frac{z^2}{H^2} - 6 \frac{z}{H} + 2 \right) - \frac{2rH^2}{\pi^2 c_v} \sum_{n=1}^{\infty} \frac{\cos n \pi z}{n^2} \exp(-n^2 \pi^2 T_v)
$$  \hspace{1cm} (2.7)

where $\varepsilon$ = vertical strain; $r$ = strain rate; $c_v$ = coefficient of consolidation ($c_v = k/\gamma_w m_v$); $m_v$ = the coefficient of volume compressibility; $T_v$ = time factor ($T_v = c_v t/H^2$).

It can be seen from Eq. (2.7) that the term containing the exponential function vanishes for large values of time and when time tends to infinity, the strains have a parabolic distribution within the specimen. This is called “steady state” by Wissa et al. (1971). For the steady state condition, the term containing the exponential function in Eq. (2.7) is neglected, and a parabolic strain distribution within the specimen is obtained as:

$$
\varepsilon(z,t) = rt + \frac{rH^2}{c_v} \left( \frac{1}{6} \left( 3 - \frac{z^2}{H^2} - 6 \frac{z}{H} + 2 \right) \right)
$$  \hspace{1cm} (2.8)

The strain distribution within the sample is shown in Fig. 2.2.

![Strain distribution](image)

**Fig. 2.2 Strain distribution within the sample (Wissa et al.’s method)**

For the linear assumption ($m_v = d \varepsilon/d \sigma'_v = \text{constant}$), the coefficient of consolidation is obtained as:

$$
c_v = \frac{H^2}{2u_b} \frac{\Delta \sigma_v}{\Delta t}
$$  \hspace{1cm} (2.9)

where $u_b$ = excess pore water pressure at the bottom of the sample, $\Delta \sigma_v$ = total vertical stress increment in a time interval of $\Delta t$ and $\Delta t$ = time interval.

The strains at the top and bottom of the specimen can be calculated by Eq. (2.8), and since the applied load and excess pore water pressure are measured, the stress-strain relationship of the specimen can be determined. Alternatively, the average effective vertical stress is
\[ \sigma'_v = \sigma_v - \frac{2}{3}u_o \]  
(2.10)

which can be associated with average strain \( \varepsilon_{ave} = rt \).

In the case of the nonlinear assumption \( C_c = -d\varepsilon/d\log\sigma'_v = \text{constant} \), the procedure is similar but the expressions for the coefficient of consolidation and the average effective stress are changed to

\[ c_v = -\frac{H_0^2}{2\log(1 - \frac{u_o}{\sigma_v})} \frac{d\log\sigma_v}{dt} \]  
(2.11)

and

\[ \sigma'_v = (\sigma'_v^3 - 2\sigma'_v^2 u_o + \sigma'_v u_o^2)^{1/3} \]  
(2.12)

The expression for the average strain is the same since the strain distribution does not depend on the applied void ratio-effective vertical stress model.

Although the analysis described above is theoretically consistent, some of the assumptions are not always justified. First, the analysis is based on the assumption that coefficient of consolidation is constant throughout the test. The second assumption is that the coefficient of volume change (linear assumption) or compression index (nonlinear assumption) is constant.

The small strain theory is based on the assumption that strain is infinitesimally small. This is standard assumption made also for incremental loading (IL) consolidation test. In the IL test the load increment can be reduced to preserve the small strain assumption, yet no such means are available in the CRS test. A strain magnitude as high as 30% has been reported for CRS tests by Smith and Wahls (1969), while a magnitude of around 20% is not uncommon in the published results (Gorman et al. 1978). Under these circumstances a large strain theory will be much more appropriate.

### 2.2.2 Large strain theory

(1) Umehara and Zen method

The analysis method proposed by Umehara and Zen (1980) is based on the large strain consolidation theory developed by Mikasa (1963). The governing equation is

\[ \frac{\partial \zeta}{\partial t} = c_v \zeta^2 \frac{\partial^2 \zeta}{\partial \zeta_0^2} \]  
(2.13)

where \( \zeta = (1 + e_0)/(1 + e) = 1/(1 - \varepsilon) \).

The governing equation is solved numerically with the appropriate boundary conditions and for various values of a non-dimensional parameter \( c_v/R H_0 \) (where \( R \) is the rate of deformation, and \( H_0 \) is the initial thickness of the sample). A series of charts with curves relating the consolidation ratio to the strains and the non-dimensional parameter \( c_v/R H_0 \) has been constructed.

From the experiment, a ratio \( F \) of strains at the two ends of the specimen can be determined in the same way as suggested by Wissa et al. (1971):

\[ F = \frac{\log(\sigma_v - u_o) - \log(\sigma'_v)}{\log(\sigma'_v) - \log(\sigma'_v)} \]  
(2.14)

where \( \sigma'_v = \text{initial effective vertical stress} \). Using the constructed charts, a value for the
parameter $c_v / R H_0$ corresponding to the ratio $F$ can be obtained. When this parameter is known, the consolidation ratio at both ends of the specimen can be found from other charts. Thus, both the coefficient of consolidation and the effective vertical stress-void ratio relationship are obtained.

The strain distribution within the sample is shown in Fig. 2.3.

![Fig. 2.3 Strain distribution within the sample (Umehara and Zen method)]

(2) Lee’s method

The method proposed by Lee (1981) starts with the governing equation written with the porosity $n$ as the dependent variable:

$$\frac{\partial n}{\partial t} = \frac{\partial}{\partial z} \left( \frac{\partial n}{\partial z} \right)$$

where $z$ is a convective coordinate system, and therefore its domain is variable with time. This formulation includes some unnecessary difficulties in solving the problem because of the variable domain.

A solution to Eq. (2.15) with the appropriate boundary condition is obtained numerically by assuming the coefficient of consolidation to be constant throughout the test. A dimensionless parameter $\beta$ (normalized strain rate) is defined as:

$$\beta = \frac{r H_0^2}{c_v}$$

and solutions for different values of $\beta$ are obtained. It is observed that for small value of $\beta$ (less than 0.2), the finite strain solution is closely approximated by the infinitesimal strain solution. Based on this finding, an analysis procedure, which closely follows that of Wissa et al. (1971) is suggested. The procedure is also divided into two parts: the steady state and transient state. In the transient state, the similarity between infinitesimal and finite strain theories is invoked, and therefore exactly the same analysis as suggested by Wissa et al. (1971) is recommended.

The steady state analysis represents a slight modification of the above. First, it is recognized that a true steady state does not exist during the test when the finite strain formulation is used. Therefore, some additional assumptions are needed in order to apply an analysis similar to the one proposed by Wissa et al. (1971) for the steady state condition.
The first assumption is that the strain distribution within the specimen can be approximated by a parabolic function. This assumption is a result of the analysis when an infinitesimal strain theory is used. After neglecting higher powers of the parameter $\beta$, a simple expression for the coefficient of consolidation is obtained as:

$$c_v = \frac{H^2 \sigma_v}{2u_b \Delta}$$

This expression is equivalent to Eq. (2.9). The only difference is that here $H$ represents the current height of the specimen.

When the value for the coefficient of consolidation is found, strains within the specimen can be calculated. Thus, the void ratio-effective vertical stress relation can be found. In the discussion of the proposed analysis, Lee (1981) recognized a strong possibility that distinct curves can be found when the calculating is performed for strains and stresses at different ends of the specimen. There is no rationale procedure to decide which of the curves is the best approximation to the true material behavior.

The strain distribution within the sample is shown in Fig. 2.4.

![Fig. 2.4 Strain distribution within the sample (Lee method)](image)

2.2.3 Summary and comments

This section presents a summary of theoretical studies about strain distribution within the sample of CRS test and interpreting CRS test results. The different theory result in different strain distribution within the sample of CRS test and different method on interpreting CRS test results due to the different simplifying assumptions. The existing theories can be divided into small strain theory and large strain theory according to whether considering the change of the thickness of the sample during a time interval. Small strain theory is simple and is adopted into the standard for CRS test (JSA 2000b; ASTM 2006). Large strain theory will be much more appropriate under the condition of large strain. For clayey soil deposit, the strain is not small. So there is a question, under what kind condition the small strain theory can yield an acceptable result? And For what kind of condition, the large strain theory may result in a better or more realistic result? In this study, wissa et al.’s theory (small strain theory) and Umehara and Zen method (large strain theory) will be used to investigate the difference between small strain theory and large
strain theory, and the difference of interpreted $c_v$ value by small strain theory and large
strain theory.

2.3 Strain rate effect on consolidation behavior

2.3.1 Strain rate effect on stress-strain curves

In Terzaghi’s theory of consolidation, the effective vertical stress-strain response of the
soil is unique, linear, and time independent. However, as early as 1936 (Buisman, 1936), it
has been recognized that time could be an important factor, since decrease of void ratio
under constant effective vertical stress was observed.

Suklje (1957) proposed a stress-void ratio-consolidation speed model, called the isotach
model, to predict the secondary consolidation process of a field-size clay layer. He found
that a set of averaged values of void ratio and effective stress is corresponding to one value
of ‘specific speed of consolidation per unit of layer thickness”, which is actually the
gradient of exit velocity of pore water flow.

Crawford (1964) illustrated the substantial influence of rate of testing on the confined
compression characteristic of undisturbed clay. The great difference between laboratory
and filed rates of consolidation existed. So any consolidation theory developed in the
laboratory must be evaluated by field observations.

A one-dimensional model of the isotach-law type was proposed by Imai et al. (2003) for
normally consolidated clays which have neither interparticle bonding nor sensitive
structure. At first, the mechanical quantities that can absolutely define a consolidation state
without employing any arbitrary reference state were examined, and it was concluded that
specific volume ($f$), effective vertical stress ($\sigma_v'$) and strain rate ($\dot{\varepsilon}$) were best
variables. Next, the existence of a single relationship $f - \sigma_v' - \dot{\varepsilon}$ after compression yielding was
verified by the results obtained from various types of laboratory consolidation tests with
the measurement of long-term secondary settlement. Then, a way of conceptualizing how
to incorporate this relationship into the new consolidation model was introduced, and
numerical simulations based on the new model were shown for clay layers of field-size.

Yin and Graham (1988) presented a 1-D model for stepped loading using a new concept
for establishing “equivalent times” during time-dependent straining. This model was
developed into a general constitutive equation for continuous loading. The general model
has been used to develop analytical solutions for creep tests, relaxation tests, constant rate
of strain tests and tests with constant rate of stress. Results from three different clays had
been used to examine the validity of the model.

Kabbaj et al. (1988) defined in situ effective vertical stress-strain curves for five
sublayers situated under four different test embankments. These in-situ stress-strain curves
were compared with laboratory stress-strain curves deduced from conventional IL test
(IL24) and from incremental loading tests with reloading at the end of primary
consolidation (ILp) carried out on high quality samples. In the normally consolidated range,
the ILp tests strongly underestimate the in situ strains. The IL24 tests which incorporate
some degree of secondary compression are closer to the in situ curves.

Consolidation settlement of two Holocene marine clays improved by prefabricated
vertical drain was analyzed in order to investigate the applicability of CRS test by using
the stress-strain relations determined by the test carried out at a strain rate of 0.02%/min (Suzuki and Yasuhara, 2004). It was confirmed, from the comparison between the actual settlement and the result of the analysis, that CRS test can be applied directly to detect the stress-strain relations mobilized in the field when clay deposit improved by vertical drain was concerned.

Various types of consolidation test (constant rate of strain tests, controlled gradient tests, incremental consolidation tests and creep tests) were carried out and a rheological clay model had been established by Leroueil et al. (1985). The clay behavior under one-dimensional compression was controlled by a unique stress-strain-strain rate relationship \((\sigma_v' - \varepsilon - \dot{\varepsilon})\) which can be described by only two curves. One is the relationship between preconsolidation pressure and strain rate \((p_c - \dot{\varepsilon})\). Another one is the relationship between normalized effective stress and strain \((\sigma_v'/p_c - \varepsilon)\). Once the two relationships are known for a given soil, any stress-strain-strain rate relationship for the soil may be easily reconstructed.

The applicability of the stress-strain-strain rate model established in laboratory was examined in field conditions under three different test embankments (Leroueil et al. 1988). It was shown that the model applied well to in situ conditions at large strains. On the contrary, at small strains, probably because of disturbance and different stress path, at a given strain and strain rate, the in situ effective stress was higher than the laboratory one.

2.3.2 Strain rate effect on consolidation yield stress \((p_c)\)

The strain rate effect on \(p_c\) has been investigated for nine undisturbed marine clay deposits obtained from various countries including Japan (Tanaka et al. 2000). A series of laboratory tests were conducted on the samples under various strain rates using CRS test. From the data reported, an increase in \(p_c\) of about 17% with a tenfold increase in the strain rate can be calculated.

Laboratory tests on a wide variety of lightly overconsolidated natural clays showed that important engineering properties such as undrained strength and preconsolidation pressure are time-dependent (Graham et al., 1983). Regardless of the soil type, \(p_c\) change by about 10–20% for a tenfold change in strain rate.

The results of IL tests and CRS tests for the soil samples from 5 boreholes in Saga, Japan, have been presented and compared (Chai et al., 2006). For the strain rate tested \((0.02 – 0.1%/\text{min})\), \(p_c\) increased with strain rate.

A series of conventional and special consolidation tests was carried out on clay samples from 11 sites in the Champlain sea basin (Leroueil et al. 1983). The test results showed that, for a given clay at a given depth, there was a unique preconsolidation pressure-strain rate relationship independent of the tests carried out, and the magnitude of the strain rate effect per log cycle of strain rate on \(p_c\) is between 14% at Berthierville and 10% at Louiseville. The relationships obtained in different Champlain clays can be normalized. Correlations between the preconsolidation pressure values obtained from different special consolidation tests and the conventional test were established, and a method of estimating in-situ preconsolidation pressure was suggested.

Both the compressibility and the undrained shear strength of a naturally cemented heavily overconsolidated clay were found to be profoundly influenced by the time effects
Slow rates of strain in CRS tests resulted in large reductions in apparent preconsolidation pressure and increased compressibility. Tanaka et al. (2006) assumed that the isotaches model can be applied to the irrecoverable strain. The strain rate effect at very small strain rate was obtained from a series of relaxation tests for Osaka Pleistocene clays. In the investigation the “strain rate dependency ratio” (SRDR) is defined as the ratio of the stress in the same strain under the objective strain rate, based on the strain rate of 0.02%/min. It was revealed that the SRDR at infinite small strain rate is about 0.7, which corresponds to the inverse value of OCR for the Osaka Pleistocene clays.

Based on the CRS tests performed on soft Berthierville silty clay from Canada at different stain rates and temperatures, the viscous behavior of natural clays were investigated (Yashima et al. 1998). The behavior of natural clays during one dimensional consolidation was influenced strongly by strain rate as well as temperature. The preconsolidation pressure was a function of both strain rate and temperature.

2.3.3 Relation between the strain rate effect and soil properties

Tanaka et al. (2000) investigated the strain rate effect on \( p_c \) using total 9 types of intact clay specimens collected in various areas. An attempt was made to correlate the strain rate effect on fundamental soil properties such as plasticity index (\( I_p \)), clay content and over consolidation ratio (OCR). However, it has been found that no strong correlation exists between the strain rate effect and fundamental soil properties.

Graham et al. (1983) reported that the effect of strain rate on \( p_c \) is essentially independent of test conditions, and soil type as expressed by \( I_p \), and the influence of strain rate on undrained shear strength appears to be independent of soil plasticity, test type or stress history during laboratory reconsolidation.

2.3.4 Summary and comments

This section presents a summary of studies about strain rate effect on consolidation behavior. The compressibility or stress-strain curve is strongly influenced by the strain rate both in laboratory and in situ, (Leroueil, 1988). Numerous CRS tests have been performed on natural clays at various strain rates. All the tests show that the strain rate influences the compression curve of clays: at a given strain, the higher the strain rate, the higher the effective vertical stress. An effective vertical stress-strain-strain rate model \( (\sigma'_v - \varepsilon - \dot{\varepsilon}) \) seems to be most representative of the rheological behavior of natural clays. This means that whatever the test, at a given strain, there is a unique relation between the effective vertical stress and strain rate. This behavior can be described by curves of equal strain rate in \( \sigma'_v - \varepsilon \) diagram and can be called the isotach model. The consolidation yield stress \( (p_c) \) increase by about 10-20% with a tenfold increase in the strain rate. In this study, the strain rate effect on stress-strain curves and \( p_c \) of Ariake clay will be systematically investigated.

The strain rate dependent behavior is due to the viscosity natural of clayey soils. Fundamentally the strain rate effect should be related to clay content and clay mineral of a soil. There is no correlation exists between the strain rate effect and the soil properties was found in the literature (Graham et al., 1983; Tanaka et al. 2000). It is considered that one of the reasons is that different samples had to be used for different strain rates and the
samples for CRS test and grain size distribution test were also different. In this study, this problem will be avoided. Same sample for different strain rates and the tested samples will be used for grain size distribution test to investigate the relation between the strain rate effect and clay content.

2.4 Anisotropic consolidation behavior

2.4.1 Anisotropic consolidation behavior investigated by IL test

As the oedometer test does not permit horizontal drainage, Rowe cell or other consolidometers which have provisions for horizontal drainage have to be used to determine coefficient of consolidation ($c_h$) and hydraulic conductivity ($k_h$) of soil in horizontal direction. The Rowe cell was developed by Rowe and Barden (1966) to overcome the disadvantages of oedometer cell and to study the effect of soil fabric. Unlike the oedometer test, the vertical load is applied by hydraulic pressure through a diaphragm. Both the equal stress and equal strain boundary conditions can be simulated using either a rigid loading plate or a flexible porous plastic disk. The specimen is sealed within the cell. As such, a back-pressure can be applied, and the volume change and excess pore water pressure can be measured during the test. For measurement of $c_h$ and $k_h$, the drainage conditions can be arranged in one of the following ways: (a) to allow water flow to the periphery and (b) to allow water to flow to the center. In the former, a porous lining, usually porous plastic with the side facing soil smoothed is used. In the latter, a center drain, usually sand drain can be installed. The sand drain is installed by drilling a hole at the center of the specimen and then filling the hole with sand. The procedure for conducting a Rowe cell test is similar to that for an oedometer test, except that the volume change and pore water pressure data are recorded in addition to the vertical displacement.

A comparison of oedometer test results and the Rowe cell test results for Singapore marine clay have been reported by Bo et al. (2003). The two oedometer tests were conducted on a horizontal-cut and a vertical-cut specimen, respectively. The Rowe cell test was conducted with horizontal drainage. The oedometer test on horizontal-cut specimen was meant to measure $c_h$ of soil. $c_v$ (or $c_h$) measured by the two oedometer tests are almost the same, but are smaller than that obtained from Rowe cell test. Therefore, they believe that in terms of measuring $c_h$, the method of using the oedometer test on horizontal-cut specimen is not reliable.

Wong (2005) evaluated the $c_h$ of fibrous peat soil through Rowe cell consolidation test with radial drainage to periphery. Comparison results between Rowe cell consolidation test with radial and double drainage indicated the ratio of $c_h/c_v$ of the soil is in the range from 1.6 to 2.9 for consolidation pressures of 50 kPa, 100 kPa and 200 kPa. This implies that the utilization of horizontal drain maybe suitable for soil improvement to accelerate the settlement of fibrous peat soil.

A new consolidometer has been developed by Chu et al. (1997) to overcome some of the shortcomings of the Rowe cell and combine some of the advantages of the oedometer cell. It adopts the oedometer setup, but allows flows in the horizontal direction and the measurement of volume change and pore water pressure. The device has an enclosed chamber so that the back-pressure can be applied, and volume and pore water pressure...
changes can be measured. The vertical loads on the specimen can be applied in stages through a rigid platen by an oedometer compression frame or by a triaxial compression machine to conduct constant rate of strain tests. The oedometer is also designed as a floating ring to reduce the friction between the specimen and the ring. During the test, all the readings can be taken by a data-logger.

For Ariake clay, Park (1994) reported that the ratio of \( k_h/k_v \) is about 1.5 through IL test with vertically cut sample and horizontally cut sample under vertical drainage.

**2.4.2 Anisotropic consolidation behavior investigated by CRS test**

A consolidometer for peripheral radial drainage under constant rate of strain (CRS) loading was developed by Yune and Chung (2005). Comparative experiments with CRS loading and incremental loading (IL) were carried out in radial drainage and also in vertical drainage. The results obtained from the developed CRS loading test agreed well with those of the conventional incremental loading test for radial drainage. No noticeable difference in compression curves between radial and vertical drainage was perceived. The effect of strain rate for the CRS test was not clearly identified in its selected range of 4.2\%/h to 12.9\%/h. In the overconsolidated state, the drainage direction effect is hardly observed, and \( c_r \) values in a normally consolidated state are higher than \( c_v \) values, reflecting that the anisotropy of permeability (i.e., higher permeability in horizontal direction) appeared to be noticeable in virgin compression. Higher anisotropy in the undisturbed sample (\( c_h/c_v = 1.7 \)) than that in reconstituted sample (\( c_h/c_v = 1.3 \)) is observed.

A method of determining the consolidation characteristics of soft Bangkok clay under the center radial drainage condition by using a newly developed CRS consolidometer was reported by Sea and Juinmarongrit (2003). A series of CRS tests under radial drainage were compared to the results of oedometer tests under vertical drainage. The CRS compression curves at a strain rate of \( 1 \times 10^{-6}/s \) agree with those obtained from conventional oedometer tests. Hence, the CRS test at this strain rate can be used for routine testing on Bangkok clay. The ratios of \( k_h/k_v \) and \( c_h/c_v \) increased from 1.5 to 3 with increase in effective vertical stress from 20 to 500kPa and at effective overburden stress the ratio is about 1.45, indicating that Bangkok clay is anisotropic. The horizontal coefficient of permeability and consolidation obtained from CRS tests under radial drainage are independent of strain rate. The horizontal coefficients of consolidation from field piezoprobe tests are in good agreement with the results from laboratory CRS tests.

While Sea and Koslanant (2003) investigated the anisotropic consolidation behavior of soft Bangkok clay by means of CRS tests with vertical and radial drainage conditions. The results of CRS tests under different drainage conditions indicated that soft Bangkok clay is almost isotropic in its natural state. The ratios of \( k_h/k_v \) and \( c_h/c_v \) are close to unity at in-situ effective stress with an overconsolidation ratio of 2. The ratios increased with increasing stress due to stress-induced anisotropy of the soil.

For determining the \( k_h \) of clay, a consolidometer for center radial drainage under CRS loading and method for determining \( k_h \) was developed by Moriwaki and Satoh (2009). The test results indicated that ratio of \( k_h/k_v \) is about 10 in the initial virgin compression range, the ratio decrease when the stress increase due to the original structure was destroyed.
2.4.3 Summary and comments

This section presents a summary of studies about anisotropic consolidation behavior investigated by IL and CRS tests. Some IL and CRS tests with radial drainage were performed on natural clays to investigate the anisotropic consolidation behavior. All the test results showed that the consolidation properties in the horizontal direction ($c_h$ and $k_h$) are bigger than that in the vertical direction ($c_v$ and $k_v$) due to the anisotropic nature of soil. Comparison with the IL test with radial drainage, the CRS test with radial drainage also has the merits same with vertical drainage, for example, the test duration can be substantially reduced, and it provides continuous data points for a plot of void ratio ($e$) against effective vertical stress ($\sigma_v'$), which can increase the accuracy of determined consolidation yield stress ($p_c$). The ratio of $c_h/c_v$ is suggested as 1.3 and 1.7 by Yune and Chung (2005) for Korea reconstituted and undisturbed clay respectively. The ratio of $c_h/c_v$ is suggested as 1.45 by Sea and Juirnarongrit (2003) for Bangkok clay under the overburden pressure. For Ariake clay Park suggested that the ratio of $c_h/c_v$ is 1.5. But this result is only from the IL test using horizontally cut sample. In this study, the ratio of $c_h/c_v$ for Ariake clay will be investigated through CRS test using horizontally cut sample and CRS test with radial drainage.
3 LABORATORY EXPERIMENTAL INVESTIGATIONS

3.1 Introduction

In general, the experimental program is to investigate consolidation behavior of Ariake clay. The constant rate of strain (CRS) tests using vertically cut sample with vertical drainage and stepwise CRS tests using vertically cut sample with vertical drainage were used to investigate the strain rate effect on consolidation behavior of Ariake clay. The anisotropic consolidation behaviors were investigated through CRS tests using horizontally cut sample and CRS tests using vertically cut sample with radial drainage. In engineering practice, Incremental loading (IL) tests are more widely used than CRS tests. IL tests were employed to investigate at what kind of strain rate, the consolidation characteristics obtained from CRS tests will be comparable with those from IL tests.

This chapter gives the relevant details of the experimental investigations carried out in this study. The soil samples, test equipments, test methods and the result interpretation are described for each kind of test.

3.2 Soil samples

Undisturbed soil samples were obtained from seven boreholes (BH) in the Saga Plain with the locations as indicated in Fig. 3.1. The Japanese standard thin-wall sampler was used to obtain the undisturbed samples. In all, 47 sample tubes (1 m long, with depth varying from about 1.0 m to 21.0 m from the ground surface) were obtained.

![Fig. 3.1 Borehole locations](image)

After the sample tubes had been transported into the laboratory, soil samples were extruded from the thin-wall tubes, sealed with wax, and stored under water for consolidation tests. The sample preparation process for testing is shown in Fig. 3.2.
Among the soft clay deposit in Saga Plain, there are marine clay layers (Ariake clay) and nonmarine clay layers (Hasuike formation) (Miura et al., 1998). No detailed geological study is carried out to distinguish the formation histories of the samples, and all samples are called Ariake clay in this study.

In generally, the natural water contents $w_n$ of the samples were more than 100%, and slightly higher than their corresponding liquid limits $w_L$. The main clay mineral of Ariake
clay is smectite (Ohtsubo et al., 1995). The clay content (< 5 μm) is in the range of 20–70%, and $I_p$ is about 50. Figures 3.3 (a)-(g) show the soil profile and some of the index properties of the soil samples from BH-1 to BH-7. In these figures, C stands for clay, M for silt, S for sand, G for gravel, and $w_p$ for plastic limit.
3.3 Test equipment and method

3.3.1 CRS test with vertical drainage

(1) Equipment

The set-up of the CRS test with vertical drainage is illustrated in Fig. 3.4. The device consists of axial displacement control and back-pressure application systems. During a test, drainage was allowed only at the top surface of the sample. The axial displacement, axial load and excess pore water pressure at the bottom of the sample were recorded by a computer through a data logger.
(2) Test method

The soil sample has a diameter of 60 mm and a nominal height of 20 mm. To increase the degree of saturation, a back-pressure of 200 kPa was applied throughout the tests. CRS tests were carried out according to Japanese Industrial Standard JIS A 1227 (JSA, 2000b). The strain rate adopted was 0.02–0.2%/min. Two types CRS tests using vertically cut sample with vertical drainage were conducted, one is with a fixed strain rate, and the other is changing the strain rate once during the test (stepwise CRS test). For the stepwise CRS tests when the vertical strain reached 10% the strain rate was changed. Two types of stepwise CRS test were carried out: one type started with a higher strain rate of 0.2%/min, and changed the strain rate to 0.02%/min (sw-HL test), and the other started with a strain rate of 0.02%/min and changed it to 0.2%/min (sw-LH test).

(3) Result interpretation

1) Small strain theory

For the CRS test with vertical drainage, assuming that the distribution of excess pore pressure within a sample is parabolic (JSA, 2000), the average effective vertical stress \( \sigma'_v \) in a sample is calculated as:

\[
\sigma'_v = \sigma_v - \frac{2}{3} u_b
\]  

(3.1)

where \( \sigma_v \) = the total vertical stress and \( u_b \) = the excess pore pressure at the bottom of the sample. The value of \( c_v \) can be calculated as follows:

\[
c_v = \frac{\Delta \sigma_v \bar{H}}{2 \bar{u}_b \Delta t}
\]  

(3.2)

where \( \Delta \sigma_v \) = the total vertical stress increment in a time interval of \( \Delta t \); \( \bar{H} \) and \( \bar{u}_b \) = the
average sample thickness and excess pore water pressure at the bottom of the sample, and they can be calculated as follows:

\[
\overline{H} = \frac{H_t + H_{t+\Delta t}}{2}
\]

(3.3)

\[
\overline{u}_t = \frac{u_{b,t} + u_{b,t+\Delta t}}{2}
\]

(3.4)

where \(H_t\) and \(H_{t+\Delta t}\) = the thickness of the sample at time \(t\) and \(\Delta t\), \(u_{b,t}\) and \(u_{b,t+\Delta t}\) = the excess pore water pressure at the bottom of the sample at time \(t\) and \(\Delta t\), respectively.

The coefficient of volume compressibility \((m_v)\) value is calculated by the following equation:

\[
m_v = \frac{\Delta H}{\overline{H} \Delta \sigma'_v}
\]

(3.5)

where \(\Delta H\) = the change of the sample thickness and \(\Delta \sigma'_v\) = the increment of effective vertical stress in a time interval of \(\Delta t\). The hydraulic conductivity \((k)\) is directly calculated from the test results as follows:

\[
k = \frac{\gamma_v \Delta H \overline{H} \Delta \sigma'_v}{2 \pi \Delta t \Delta \sigma'_v}
\]

(3.6)

2) Large strain theory

There is no explicit equation for calculating \(c_v\) with large strain theory, and a diagram method needs to be used. The ratio of the strain at the bottom to that at the top of a sample under different strain rate can be obtained using finite difference method as shown in Fig. 3.5.

![Fig. 3.5bis Variations of \(\varepsilon_B / \varepsilon_T\)]

The change of void ratio \(\Delta e\) is proportional to the change of effective vertical stress in logarithmic scale as follows:

\[
\Delta e = -C_v \log \left( \frac{\sigma'_{v} + \Delta \sigma'_{v}}{\sigma'_{v}} \right)
\]

(3.7)

where \(C_v\) = a constant for the effective vertical stress from \(\sigma'_{v}\) to \(\sigma'_{v} + \Delta \sigma'_{v}\).
Using the stress versus strain relationship given by Eq. (3.7), the ratio of bottom strain to top strain can be represented in terms of vertical effective stress as:

\[
\frac{\varepsilon_B}{\varepsilon_T} = \frac{\log(\sigma_v - u_v) - \log(\sigma_v')}{\log(\sigma_v) - \log(\sigma_v')} \tag{3.8}
\]

The CRS test data can be used in Eq. (3.8) to find \( \varepsilon_B/\varepsilon_T \) corresponding to \( \Delta H_0/H_0 \) at any time, \( t \). This value can then be used in Fig. 3.5 to find \( c_v/rH_0^2 \) and hence \( c_v \) can be determined.

### 3.3.2 CRS test with radial drainage

#### (1) Equipment

The CRS test equipment with radial drainage is illustrated in Fig. 3.6. The equipment was modified from the device for vertical drainage test, and the main modifications are as follows:

1. **Central drainage porous stone**: A 8 mm outside diameter and 5 mm inside diameter cylindrical porous stone is inserted in the middle of the specimen, which is used as the only drainage boundary of the soil specimen during consolidation.

2. **Loading cap**: A stainless steel loading cap is used. A hole with 9 mm in diameter and 10 mm in depth at the center of loading cap allows the cylindrical porous stone to slide freely during consolidation and five holes connected to the central hole on the loading cap provide drainage paths.

3. **A guide ring**: A guide ring is set on the top of the specimen. To prevent the leakage in vertical direction, a greased “o”-ring is set between the guide ring and the wall of the chamber, and a greased “o”-ring is placed between the loading cap and the guide ring. The friction between the “o”-ring and the loading cap was calibrated.

4. **Consolidation ring**: A consolidation ring with a hole in the middle height of the wall was newly manufactured. The pore water pressure gauge is installed through the hole.

5. **Bottom pedestal**: A new bottom pedestal was manufactured to provide an undrained boundary at the bottom.

6. **An acrylic fiber ring**: It is placed above the guide ring in order to improve the contact between the guide ring and the consolidation ring. A rubber ring is placed between the acrylic fiber ring and the guide ring to ensure the uniform pressure on the guide ring.
Fig. 3.6 Illustration of the CRS test with radial drainage device
(2) Test method

The hole in the center of the specimen for inserting the annular porous stone is made by a cylindrical device as shown in Fig. 3.7. The inner diameter of the cylindrical device is the same as the outside diameter of the consolidation ring, and the pipe at the center has a diameter of 8 mm and height of 12 mm. The device is pushed down slowly into the specimen placed in the consolidation ring and then takes out the soil in the pipe. The hole in the upper part of the specimen is made. Then the consolidation ring with specimen inside is turned over to make the hole in the remaining unpenetrated specimen with the same procedure.

![Fig. 3.7 Photograph of the cylindrical control device](image)

The main difference between the procedures of CRS test with radial drainage and vertical drainage is the way of measuring pore water pressure ($u$). For radial drainage case, $u$ at the outside perimeter of the specimen is measured and the valve at the bottom of the specimen is closed.

(3) Result interpretation

The consolidation with radial flow is governed by the following differential equation:

$$
e_a \left( \frac{\partial^2 u_r}{\partial r^2} + \frac{1}{r} \frac{\partial u_r}{\partial r} \right) = \frac{\partial u_r}{\partial t}$$

(3.9)

where $r$ = the radial distance from the center of the specimen and $u_r$ = excess pore water pressure at $r$. With a central drain and under equal vertical strain assumption, the solution with ideal condition (no smear and no well resistance), which is applicable to test condition in this study, is as follows (Barron 1948):

$$u_r = \frac{\overline{u}_o e^{-\lambda}}{r^2 F(n)} \left[ r^2 \ln(r / r_w) - \left( r^2 - r_w^2 \right) / 2 \right]$$

(3.10)

Let $B(t) = \frac{\overline{u}_o e^{-\lambda}}{r^2 F(n)}$,

$$u_r = B(t) \left[ r^2 \ln(r / r_w) - \left( r^2 - r_w^2 \right) / 2 \right]$$

(3.10a)

where $\overline{u}_o$ = initial excess pore water pressure (uniform); $r_w$ = the radius of the specimen;
\[ \lambda = 8T_s / F(n); \quad T_h = \text{the time factor of radial drainage and it is calculated as} \quad T_h = c_s t / 4r_e^2; \]

\[ F_s = \frac{n^2}{n^2 - 1} \ln(n) - \frac{3n^2 - 1}{4n^2}; \quad r_w = \text{the radius of central drain and} \quad n = r_e / r_w. \]

Juinarongrit (1996) established the equations based on Barron’s theory to determine the \( c_h \) and \( k_h \) from the results of CRS test with radial drainage. By applying boundary condition of at \( r = r_e, \ u_r = u_e, \) then

\[ B(t) = \frac{u_e}{r_e^2 \ln(n) - \left(\frac{r_e^2 - r_w^2}{2}\right)} \]  

(3.11)

Differentiating Eq. (3.10a) with respect to \( r \) gives:

\[ \frac{\partial u_r}{\partial r} = B(t) \left(\frac{r_e^2 - r^2}{r}\right) \]  

(3.12)

Based on Darcy’s law, water flow rate \( (q) \) through a cylindrical surface with a radius of \( r \) and height of \( H \) (Fig. 3.8) can be written as:

\[ q = \frac{k_s d u_r}{\gamma_w d r} 2 \pi r H \]  

(3.13)

Substituting Eq. (3.12) into Eq. (3.13) gives:

\[ q = \frac{2 \pi k_s H}{\gamma_w} B(t) \left(\frac{r_e^2 - r^2}{r}\right) \]  

(3.14)

At \( r = r_w \), the rate of discharge of water can be written as:

\[ q = \frac{2 \pi k_s H \left(\frac{r_e^2 - r_w^2}{r}\right)}{\gamma_w} \]  

(3.15)

Assuming the sample is fully saturated, the rate of discharge of water can also be expressed in terms of vertical displacement rate \( \nu_r \) as:

\[ q = \nu_r \gamma_w \left(\frac{r_e^2 - r_w^2}{r}\right) \]  

(3.16)

Combining Eq. (3.15) and Eq. (3.16), and if \( r_e = 30 \text{ mm} \) and \( r_w = 4 \text{ mm} \), then

\[ k_h = \frac{\nu_r \gamma_w \left[ r_e^2 \ln(n) - \left(\frac{r_e^2 - r_w^2}{2}\right) / 2\right]}{2u_e H} = \frac{0.762r_e^2 \nu_r \gamma_w}{u_e H} \]  

(3.17)

The horizontal coefficient of consolidation is, therefore, given by:

\[ c_h = \frac{k_s}{m \gamma_w} = \frac{0.762 \nu_r \gamma_w}{u_e H m} \]  

(3.18)

The average excess pore pressure in a specimen is:

\[ \bar{u} = \int_w \frac{2 \pi \nu u d r}{A} = \left[ \frac{r_e^4 \ln(n) - 3r_e^4 / 4 + r_e^2 r_w^2 - r_w^2 / 4}{r_e^2 - r_w^2} \right] / \left[ r_e^2 \ln(n) - \left(\frac{r_e^2 - r_w^2}{2}\right) / 2\right] u_e = 0.857u_e \]  

(3.19)

Where \( A = \text{cross-sectional area of the sample} \). So the average effective vertical stress can be expressed as:

\[ \sigma'_v = \sigma_v - \bar{u} = \sigma_v - 0.857u_e \]  

(3.20)
3.3.3 IL test

IL tests were conducted following JIS A 1217 (JSA, 2000a). The soil sample is 60 mm in diameter and typically 20 mm high. The consolidation vertical stress was doubled every 24 h.

3.4 Cases tested

Designate the CRS test using vertically cut sample (with respect to in-situ condition) with vertical drainage (with respect to test condition) as CRS-V-V, vertically cut sample with radial drainage as CRS-V-R, and horizontally cut sample with vertical drainage as CRS-H-V. The loading and drainage condition for three types of CRS test is shown in Fig. 3.9. The strain rates used for the CRS tests were 0.02, 0.05, 0.1 and 0.2%/min. A total of 248 CRS tests (of which 134 were CRS-V-V tests, 52 were stepwise CRS-V-V tests, 34 were CRS-H-V tests and 28 were CRS-V-R tests) and 15 IL tests were conducted for the undisturbed samples, as listed in Table 3.1.
Fig. 3.9 Loading and drainage condition for three types of CRS test

Table 3.1 Cases tested

<table>
<thead>
<tr>
<th>Location</th>
<th>Depth: m</th>
<th>Test method</th>
<th>Strain rate: %/min</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>1, 3, 4&lt;sup&gt;a&lt;/sup&gt;, 5, 6&lt;sup&gt;b&lt;/sup&gt;, 7&lt;sup&gt;b&lt;/sup&gt;, 9, 10, 11, 12, 13&lt;sup&gt;b&lt;/sup&gt;, 14, 16</td>
<td>CRS-V-V, Stepwise CRS-V-V</td>
<td>0.02, 0.05, 0.1, 0.2, 0.02-0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>IL</td>
<td>-</td>
</tr>
<tr>
<td>BH-2</td>
<td>3, 6, 14&lt;sup&gt;a&lt;/sup&gt;</td>
<td>CRS-V-V, Stepwise CRS-V-V</td>
<td>0.02, 0.05, 0.1, 0.2, 0.02-0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>IL</td>
<td>-</td>
</tr>
<tr>
<td>BH-3</td>
<td>4, 6, 8, 10</td>
<td>CRS-V-V, Stepwise CRS-V-V</td>
<td>0.02, 0.05, 0.1, 0.2, 0.02-0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>IL</td>
<td>-</td>
</tr>
<tr>
<td>BH-4</td>
<td>1, 2, 3, 4, 5, 6, 7, 8, 9</td>
<td>CRS-V-V, Stepwise CRS-V-V</td>
<td>0.02-0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>IL</td>
<td>-</td>
</tr>
<tr>
<td>BH-5</td>
<td>2, 6, 9, 12&lt;sup&gt;c&lt;/sup&gt;, 15, 20</td>
<td>CRS-V-V, CRS-H-V, CRS-V-R</td>
<td>0.02, 0.02, 0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CRS-V-V</td>
<td>0.02-0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CRS-H-V</td>
<td>0.02, 0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CRS-V-R</td>
<td>0.02, 0.2</td>
</tr>
<tr>
<td>BH-6</td>
<td>4, 7, 17, 20</td>
<td>CRS-V-V, CRS-H-V, CRS-V-R</td>
<td>0.02, 0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CRS-V-V</td>
<td>0.02-0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CRS-H-V</td>
<td>0.02, 0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CRS-V-R</td>
<td>0.02, 0.2</td>
</tr>
<tr>
<td>BH-7</td>
<td>5, 7, 9, 12, 14, 18, 20&lt;sup&gt;d&lt;/sup&gt;, 21</td>
<td>CRS-V-V, CRS-H-V, CRS-V-R</td>
<td>0.02, 0.02, 0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CRS-V-V</td>
<td>0.02, 0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CRS-H-V</td>
<td>0.02, 0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CRS-V-R</td>
<td>0.02</td>
</tr>
</tbody>
</table>

Note: <sup>a</sup> No IL test. <sup>b</sup> No step-wise CRS and IL test.
<sup>c</sup> No CRS-H-V test under the strain rate of 0.02%/min.
<sup>d</sup> No CRS-H-V test under the strain rate of 0.2%/min.
4 VERTICAL DRAINAGE CRS TEST RESULTS

4.1 Introduction

The results of 134 CRS-V-V tests, 52 stepwise CRS-V-V tests and 15 IL tests are presented and compared in this chapter. The effects of strain rate on the consolidation yield stress \( p_c \), compression index \( C_c \) and coefficient of consolidation \( c_v \), and the relationship between the degree of the strain rate effect and the clay content of the samples, are systematically investigated. The \( p_c \) and \( c_v \) values obtained from IL and CRS tests are compared.

4.2 Strain rate effect on consolidation yield stress \( p_c \)

4.2.1 Strain rate effect on \( p_c \)

Figures 4.1 to 4.10 are typical stress–strain–strain rate relationships for Ariake clay. In general, the stress–strain curves shift in a parallel manner to the right with increase of strain rate, similar to the results reported in the literature (Graham et al., 1983; Leroueil et al., 1985; Vaid et al., 1979). However, Figs. 4.1 and 4.5 show that the stress–strain curve for the high strain rate (0.2%/min) merges with that of the low strain rate (0.02%/min) at high strain levels, whereas other figures do not show this kind of response. Therefore it is not conclusive as to whether undisturbed Ariake clay displays a temporary strain-rate effect as some reconstituted clays do (Tatsuoka, 2002). At present, we consider that the isotach model (Suklje, 1957) is applicable to natural Ariake clay.

![Fig. 4.1 Stress-strain-strain rate relation (BH-1, 5-5.9 m)](attachment:image)
Fig. 4.2 Stress-strain-strain rate relation (BH-1, 6-6.9 m)

Fig. 4.3 Stress-strain-strain rate relation (BH-2, 3-3.85 m)

Fig. 4.4 Stress-strain-strain rate relation (BH-3, 6-6.9 m)
Fig. 4.5 Stress-strain-strain rate relation (BH-4, 2-2.9 m)

Fig. 4.6 Stress-strain-strain rate relation (BH-4, 9-9.9 m)

Fig. 4.7 Stress-strain-strain rate relation (BH-5, 9-9.8 m)
Fig. 4.8 Stress-strain-strain rate relation (BH-6, 17-17.8 m)

Fig. 4.9 Stress-strain-strain rate relation (BH-7, 9-9.85 m)

Fig. 4.10 Stress-strain-strain rate relation (BH-7, 14-14.75 m)
To quantify the strain-rate effect on $p_c$, a ratio of $p_c$ (RPC) is defined as:

$$ RPC = \frac{p_{c\dot{\varepsilon}}}{p_{c0.02%/min}} $$

(4.1)

where $p_{c\dot{\varepsilon}}$ is the consolidation yield stress for a given strain rate $\dot{\varepsilon}$ (0.05%/min, 0.1%/min or 0.2%/min), and $p_{c0.02%/min}$ is the consolidation yield stress under a strain rate of 0.02%/min.

The value of $p_c$ is obtained by Casagrande’s method. Figure 4.11 shows RPC plotted against the strain rate obtained from 134 CRS-V-V tests in this study, and some of the test results using Ariake clay samples in the literature (Iribe, 2006). The strain rate is normalised by a strain rate of 0.02%/min. Although the data are scattered, there is a clear tendency for $p_c$ to increase with the strain rate. The linear regression line gives an increase of $p_c$ by about 14% with a tenfold increase in the strain rate.

![Fig. 4.11 RPC-normalised strain rate relation](image)

**4.2.2 Stepwise CRS test results**

One of the reasons for the scatter of the data is that different samples had to be used for different strain rates. The soil samples within a 1 m long tube were treated as the same sample, but because of the spatial variation of soils in a deposit, the properties of the soil within a tube may not be exactly the same. To avoid this problem, stepwise CRS tests were conducted, which used the same sample for two different strain rates. Figures 4.12 to 4.15 show typical results of stepwise CRS tests. From the figures it can be seen that, when the strain rate was increased or decreased abruptly, the effective vertical stress showed an obvious increase or decrease.
Fig. 4.12 Stepwise CRS stress-strain-strain rate relation (BH-1, 5-5.9m)

Fig. 4.13 Stepwise CRS stress-strain-strain rate relation (BH-2, 6-6.9m)

Fig. 4.14 Stepwise CRS stress-strain-strain rate relation (BH-3, 8-8.9m)
Using the isotach model, two stress–strain curves can be deduced from a stepwise CRS test. According to the isotach model, in the virgin consolidation range, the ratio of the effective vertical stresses on two stress–strain curves with different strain rates is the same for any given strain. So the strain rate effect can be checked at any given strain level. To investigate the strain-rate effect (SRE), the effective vertical stress corresponding to 15% strain is chosen arbitrarily, and the SRE is defined as:

\[
SRE = \left( \frac{\sigma_{v0.2\%/min}'}{\sigma_{v0.02\%/min}'} \right)_{\varepsilon=15\%}
\]

where \(\sigma_{v0.2\%/min}'\) and \(\sigma_{v0.02\%/min}'\) are the effective vertical stresses corresponding to strain rates of 0.2%/min and 0.02%/min respectively. For the stepwise CRS test the strain rate was changed at about 10% strain and, effectively, the SRE is evaluated at about 10% strain. By assuming that the isotach model is valid, the SRE at 10% strain is the same as that at 15% strain. Figure 4.16 shows the SRE from 52 stepwise CRS tests. The SRE from the sw-HL tests (0.2%/min–0.02%/min) is larger than that from the sw-LH tests (0.02%/min–0.2%/min) except some samples (higher sand content) from BH-4. When the strain rate was changed, there was a transitional period. The ratio of effective vertical stress corresponding to two different strain rates was determined using the stress–strain curves before and after the change of strain rate. The value used for the new strain rate might still be within the transitional period. When the strain rate was changed from 0.2%/min to 0.02%/min, the effective vertical stress increment was reduced, but dissipation of the excess pore water pressure generated under the strain rate of 0.2%/min needs time. As a result, the excess pore water pressure was larger than the value corresponding to the strain rate of 0.02%/min, and therefore a lower calculated effective vertical stress and larger SRE value were. By contrast, when the strain was changed from 0.02%/min to 0.2%/min, for a saturated soil sample, the increase of excess pore water pressure was immediate, and before a steady state was reached the excess pore water pressure might be overestimated, and therefore a lower effective vertical stress and subsequently a smaller SRE value might be calculated.

From Fig. 4.16, an average SRE value of about 1.14 can be obtained (that is, about a
14% increase in effective vertical stress with a tenfold increase in the strain rate) which is the same to the increased rate for \( p_c \) of about 14% given in Fig. 4.11. From the data reported by Tanaka et al. (2000) using Ariake clay samples, an increase in \( p_c \) of about 17% with a tenfold increase in the strain rate can be calculated. Graham et al. (1983) stated that, regardless of the soil type, \( p_c \) increases by 10–20% for a tenfold increase in strain rate. Therefore the percentage increase in \( p_c \) or effective vertical stress with the strain rate in this study is comparable with these numbers.

![Fig. 4.16 Comparison of SRE](image)

**4.3 Strain rate effect on compression index (\( C_c \))**

**4.3.1 Strain rate effect on \( C_c \) for a given strain**

The typical compression index \( C_c \) and recompression index \( C_r \) from CRS tests corresponding to different strain are depicted in Figs. 4.17 to 4.22. The value of \( C_c \) or \( C_r \) is calculated from:

\[
C_c (C_r) = \frac{e_t - e_{t+\Delta t}}{\log(\sigma'_{v,t+\Delta t} - \sigma'_{v,t})}
\]

(4.3)

where \( e_t \) and \( e_{t+\Delta t} \) = the void ratios of the sample at time \( t \) and \( t + \Delta t \), and \( \sigma'_{v,t} \) and \( \sigma'_{v,t+\Delta t} \) = the effective vertical stress at time \( t \) and \( t + \Delta t \). Although the data are scattered, for a given strain level, generally it can be said that both \( C_r (\sigma'_v < p_c) \) and \( C_c \) show a clear trend of strain rate independence. This result support the isotach model, which has been used to interpret the stepwise CRS test results in this study.
Fig. 4.17 $C_r$ and $C_c$-strain-strain rate relation (BH-1, 5-5.9m)

Fig. 4.18 $C_r$ and $C_c$-strain-strain rate relation (BH-1, 6-6.9m)

Fig. 4.19 $C_r$ and $C_c$-strain-strain rate relation (BH-3, 6-6.9m)
Fig. 4.20 $C_r$ and $C_c$-strain-strain rate relation (BH-4, 8-8.9m)

Fig. 4.21 $C_r$ and $C_c$-strain-strain rate relation (BH-6, 17-17.8m)

Fig. 4.22 $C_r$ and $C_c$-strain-strain rate relation (BH-7, 14-14.75m)
4.3.2 Strain rate effect on $C_c$ for a given stress

If the average effective stress $\bar{\sigma}'_v$ within a time interval $\Delta t$ is defined as:

$$\bar{\sigma}'_v = \sqrt{\sigma'_{v,1} \cdot \sigma'_{v,2+\Delta t}}$$

(4.4)

then, if the values of $C_c$ and $C_r$ are plotted against $\bar{\sigma}'_v$, it can be seen that there is a tendency for $C_c$ to increase with strain rate (Figs. 4.23 to 4.26). This is because of non-linearity of $e - \log \sigma'_v$ plot. Under a given $\sigma'_v$, the higher the strain rate, the smaller the corresponding $e$ value, and therefore a higher $C_c$ value. It should be stated that, for a given stress, the tendency for $C_c$ to increase with strain rate will be decrease or disappear at very high stress due to stress-strain curve will become linear at very high stress.

Fig. 4.23 $C_r$ and $C_c$-stress-strain rate relation (BH-1, 5-5.9m)

Fig. 4.24 $C_r$ and $C_c$-stress-strain rate relation (BH-1, 6-6.9m)
4.4 Strain rate effect on coefficient of consolidation ($c_v$)

4.4.1 Strain rate effect on $c_v$

Figures 4.27 to 4.36 show the variation of $c_v$ with $\bar{\sigma}_v'$ under different strain rates for soil samples from different BHs. In the overconsolidated range the data are scattered, but in the virgin consolidation range there is a clear trend for $c_v$ to increase with increase of the strain rate.
Fig. 4.27 $c_v$-stress-strain rate relation (BH-1, 5-5.9 m)

Fig. 4.28 $c_v$-stress-strain rate relation (BH-1, 6-6.9 m)

Fig. 4.29 $c_v$-stress-strain rate relation (BH-2, 3-3.85 m)
Fig. 4.30 $c_v$-stress-strain rate relation (BH-3, 6-6.9 m)

Fig. 4.31 $c_v$-stress-strain rate relation (BH-4, 2-2.9 m)

Fig. 4.32 $c_v$-stress-strain rate relation (BH-4, 9-9.9 m)
Fig. 4.33 $c_v$-stress-strain rate relation (BH-5, 9-9.8 m)

Fig. 4.34 $c_v$-stress-strain rate relation (BH-6, 17-17.8 m)

Fig. 4.35 $c_v$-stress-strain rate relation (BH-7, 9-9.85 m)
The values of $c_v$ in the virgin compression range were chosen for a detailed investigation of the strain-rate effect on $c_v$. BH-1, 2, 3 and 4 chose $\bar{\sigma'}_v = 100$ kPa and 200 kPa, and BH-5, 6, 7 chose $\bar{\sigma'}_v = 300$ kPa. The results are given in Fig. 4.37 (excluding 12 extremely scattered points). In this figure, the values of $c_v$ at each strain rate are normalised by the value at a strain rate of 0.02%/min. From Fig. 4.37, it can be seen that there is a tendency for $c_v$ to increase with an increase of strain rate. The linear regression gives about a 30% increase for a tenfold increase in the strain rate.

Fig. 4.36 $c_v$-stress-strain rate relation (BH-7, 14-14.75 m)

Fig. 4.37bis Strain rate effect on $c_v$
4.4.2 Strain rate effect on coefficient of volume compressibility ($m_v$)

Figures 4.38 to 4.47 show the variation of $m_v$ with $\bar{\sigma}'_v$ under different strain rates for soil samples from different BHs. The value of $m_v$ is calculated from Eq. (3.5). Although the data are scattered for $\sigma'_v < p_c$, after that the values of $m_v$ are almost the same for different strain rates on a logarithmic scale.

![Graph showing $m_v$-stress-strain rate relation (BH-1 5-5.9 m)](image1)

![Graph showing $m_v$-stress-strain rate relation (BH-1 6-6.9 m)](image2)
Fig. 4.40 $m_v$-stress-strain rate relation (BH-2 3-3.85 m)

Fig. 4.41 $m_v$-stress-strain rate relation (BH-3 6-6.9 m)

Fig. 4.42 $m_v$-stress-strain rate relation (BH-4 2-2.9 m)
Fig. 4.43 $m_v$-stress-strain rate relation (BH-4 9-9.9 m)

Fig. 4.44 $m_v$-stress-strain rate relation (BH-5 9-9.8 m)

Fig. 4.45 $m_v$-stress-strain rate relation (BH-6 17-17.8 m)
$m_v$ can be calculated from $C_c$, $e$ and $\sigma'_v$ as:

$$m_v = \frac{0.434C_c}{(1 + e)\sigma'_v} \quad (4.5)$$

As shown in Fig. 4.48, even though the $e - \log\sigma'_v$ curves for different strain rates are parallel, for a given $\sigma'_v$ value the slope of the curve of $C_c$ corresponding to different strain rates is different, owing to the non-linearity of the curve: for example, $C_{c2} > C_{c1}$ (see also Figs. 4.23 to 4.26 for the strain-rate effect on $C_c$ for a given $\sigma'_c$). The corresponding value of $e$ value is also different: for example, $e_2 > e_1$. In Eq. 4.5, if $\sigma'_v$ is the same, both $e$ and $C_c$ increase with an increase in strain rate. As a result, $m_v$ does not change much with the strain rate.
Fig. 4.48 Effect of strain rate on $C_c$ and $e$ under a given $\sigma'_v$

### 4.4.3 Strain rate effect on hydraulic conductivity ($k$)

Figs. 4.49 to 4.58 show the variation of $k$ with $\bar{\sigma}'_v$ under different strain rates for soil samples from different BHs. The value of $k$ is directly calculated from the test results as Eq. (3.6). The data are scattered in the overconsolidated range, and in the virgin consolidation range $k$ obviously increases with an increase of strain rate.

![Graph showing the variation of hydraulic conductivity ($k$) with average effective vertical stress ($\bar{\sigma}'_v$)](image)

- Fig. 4.49 $k$-stress-strain rate relation (BH-1 5-5.9 m)
Fig. 4.50 $k$-stress-strain rate relation (BH-1 6-6.9 m)

Fig. 4.51 $k$-stress-strain rate relation (BH-2 3-3.85 m)

Fig. 4.52 $k$-stress-strain rate relation (BH-3 6-6.9 m)
Fig. 4.53 $k$-stress-strain rate relation (BH-4 2-2.9 m)

Fig. 4.54 $k$-stress-strain rate relation (BH-4 9-9.9 m)

Fig. 4.55 $k$-stress-strain rate relation (BH-5 9-9.8 m)
Hydraulic conductivity $k$ (m/s)

Average effective vertical stress $\bar{\sigma}_v$ (kPa)

Fig. 4.56 $k$-stress-strain rate relation (BH-6 17-17.8 m)

Fig. 4.57 $k$-stress-strain rate relation (BH-7 9-9.85 m)

Fig. 4.58 $k$-stress-strain rate relation (BH-7 14-14.75 m)
As for \( c_v \), the values of \( k \) in the virgin compression range were chosen to study the strain rate effect on \( k \), and the results are shown in Fig. 4.59 (12 extremely scattered points are excluded, as for \( c_v \) in Fig. 4.37). In this figure, the values of \( k \) at each strain rate are normalised by the value at a strain rate of 0.02%/min. Although the data are scattered, there is a general tendency for \( k \) increase with an increase in strain rate. Linear regression gives about a 30% increase of \( k \) with a tenfold increase in the strain rate, which means that the increase of \( c_v \) with increasing strain rate is due mainly to the increase in \( k \).

![Graph showing linear fit to the data with equation \( y = 1 + 0.30 \log x \)](image)

Fig. 4.59 Strain rate effect on \( k \)

In geotechnical engineering analysis it is generally accepted that Darcy’s law is valid for water flow in soil. However, there are reports in the literature that the relationship between flow rate \( q \) and hydraulic gradient \( i \) might not be linear when \( i \) is small. For example, Miller and Low (1963) reported that there is a threshold hydraulic gradient \( i_c \) for water flow in Na-clay, and when \( i < i_c \) there would be no flow (Fig. 4.60). Although the reliability of very small flow rate measurement is debatable, the data may support the statement that in clayey soil the \( q-i \) relation is non-linear when \( i \) is small, and this kind of non-linear phenomenon can help to explain the increase in \( k \) with the increasing of strain rate.
Figure 4.61 shows the variation of excess pore water pressure for different strain rates for soil samples from BH-1 at 5–5.9 m depth. At a given strain, the higher the strain rate, the higher the excess pore water pressure at the bottom of the sample, $u_b$. A higher measured $u_b$ means a higher average $i$ value within the sample. The average value of hydraulic gradient, $i_{av}$, is defined as:

$$i_{av} = \frac{u_b}{\gamma \cdot H}$$

(4.6)

where $H$ is the thickness of the sample.

Figure 4.62 shows the variation of $i_{av}$ with strain rate at an effective vertical stress $\bar{\sigma}'_v = 100$ kPa. It can be seen that $i_{av}$ increases with increasing strain rate. For a strain rate increases from 0.02%/min to 0.2%/min, the value of $i_{av}$ increases from about 26 to about 171. Referring to the information in Fig. 4.60, qualitatively it can be said that $k$ apparently increases with an increase in $i$, as shown by the dashed lines in the figure ($k_2 > k_1$).
4.5 Comparison of the test results of CRS and IL

In engineering practice, IL tests are more widely used than CRS tests, and there is a question of at what kind of strain rate, the $p_c$ and $c_v$ values from CRS tests will be comparable with those from IL tests.

4.5.1 Comparison $p_c$ values from CRS and IL test

15 IL tests were conducted for samples from BH-1, BH-2 and BH-3. The values of $p_c$ values from IL and CRS tests are compared in Figs. 4.63, 4.64 and 4.65 for BH-1, BH-2 and BH-3 respectively. Iribe (2006) also conducted both IL and CRS tests for undisturbed Ariake clay samples and the test results are included in Fig. 4.64. It can be seen that the $p_c$ values obtained from CRS tests with a strain rate of 0.02%/min are comparable with that obtained from the IL test.
In the IL test, the stress is applied in a discrete stepwise fashion. In most cases the value of $p_c$ is within a stress increment (not coincident with the stress state at the end of a stress increment), and the point dividing the virgin consolidation range and overconsolidation range will be rounded. However, in the CRS test, the stress is increased gradually and continuously. The yielding point on the stress–strain curve is sharper, and the curve itself is more non-linear than that of the IL test. As a result, the largest difference in the value of $C_c$ from the IL and CRS tests occurs around the $p_c$ value, as shown in Fig. 4.66 for soil samples from BH-1 at 5–5.9m depth. Since the stress-strain curve of the CRS test is continuous, better determination of $p_c$ is possible.
The results given in Fig. 4.66 and Figs. 4.17 to 4.26 indicate that for microstructured Ariake clay, in the virgin compression range, the $e - \log \sigma_v$ relation is non-linear. The CRS test has an advantage over the IL test for evaluating this non-linear characteristic. Chai et al. (2004) reported that for most nonlinear $e - \log \sigma_v$ relationships, in a plot of $\log (e + e_c)$ against $\log \sigma_v$, where $e_c$ is a constant, they are very close to linear, and a model is proposed to consider the non-linear $e - \log \sigma_v$ behaviour in engineering design and analysis.

4.5.2 Comparison $c_v$ values from CRS and IL test

The values of $c_v$ values from IL and CRS tests are compared in Figs. 4.67, 4.68 and 4.69 for three samples from BH-1. The values of $c_v$ from the IL test are determined by the $\sqrt{t}$ method (where $t$ is time). It can be seen that the $c_v$ values obtained from CRS tests with a strain rate of 0.02%/min are comparable with that obtained from the IL test.
Although the test conditions for IL and CRS tests are different, as a reference, the average strain rates from the IL test up to 90% degree of consolidation for a sample from BH-1 at 5–5.9 m depth were calculated, and are shown in Fig. 4.70. It can be seen that in the virgin compression range the average strain rate is about 0.02–0.03%/min, and for the whole range of consolidation vertical stress, the simple average strain rate is about 0.018%/min. This result indicates that for a CRS test with a strain rate close to the average strain rate up to 90% degree of consolidation of IL test, the result may be comparable with that of the IL test. In Japan it is intended to adopt a strain rate of 0.02%/min for a routine CRS test (Suzuki and Yasuhara, 2004).
4.6 Effect of clay content on strain rate effect

There are some researches on the relationship between the degree of strain rate effect and the clay content and plasticity index ($I_p$) of clayey soils, and no correlation between strain rate effect and $I_p$ or clay content was found (Graham et al., 1983; Tanaka et al., 2000; Jia et al. 2010). It is considered that one of the reasons is that different samples had to be used for different strain rates and the samples for CRS test and grain size distribution test were also different. The soil samples within a 1 m long tube were treated as the same sample, but the properties of the soil within a tube may not be exactly the same. To avoid the spatial variation of soils in a tube, the strain rate effect is investigated by stepwise CRS test, which uses the same sample for two different strain rates, and clay content tests were conducted using the samples of stepwise CRS tests for the 9 samples from BH-4.

Figures 4.71 and 4.72 show the relationships of SRE and clay content for sw-LH test and sw-HL test respectively. It can be seen that SRE increase with the increase of clay content. When the clay content increases from 20% to 50%, SRE increases from 1.05 to 1.20. Both the figures show that there is a clear correlation between strain rate effect and clay content. If more test data can be obtained, the relationship of strain rate effect on $p_c$ and clay content may be found. So if the clay content of a soil is known, the strain rate effect on $p_c$ of the soil can be obtained, which enables engineers to consider the strain rate effect into design easily.
4.7 Summary

Based on the test and analysis results presented in this chapter, the following conclusions can be drawn.

1. The consolidation stress–strain curves shift in a parallel manner to the right with the increase of strain rate, which indicates that isotach model is applicable to Ariake clay. Consolidation yield stress ($p_c$) of Ariake clays increased by about 14% with a tenfold increase in the strain rate.

2. For a given strain level, the strain rate does not influence the compression index ($C_c$), as the isotach model implies. While for a given average effective stress, there is a tendency for $C_c$ to increase with strain rate due to the non-linearity of $e - \log \sigma'_v$ relation.

3. Under a given effective vertical stress, the coefficient of consolidation ($c_v$) increased with the increase of strain rate resulting mainly from the increase of the hydraulic conductivity ($k$), and the coefficient of volume compressibility ($m_v$) was almost the same.
for different strain rates. Linear regression results in about a 30% increase of $c_v$ for a tenfold increase in the strain rate.

(4) Comparing the $p_c$ and $c_v$ values from the IL and CRS tests suggests that the $p_c$ and $c_v$ values from CRS tests for a strain rate of 0.02%/min are comparable with those of IL tests. In the $e - \log \sigma'_v$ plot, the compression curve from the CRS test is more non-linear than that from the IL test, and the largest difference in the $C_c$ value from the CRS and IL tests occurred around the value of $p_c$.

(5) There is a clear relationship between the strain rate effect (SRE) and clay content. Increase of clay content, increases SRE.
5 RADIAL DRAINAGE CRS TEST RESULTS AND COMPARISONS

5.1 Introduction

This chapter presents the results of 28 radial drainage CRS tests, and the comparisons of: (1) The CRS test results of radial drainage with that of vertical drainage; and (2) Vertically and horizontally cut samples under vertical drainage, to systematically investigate the anisotropic consolidation behaviors of Ariake clay. The test results are compared in terms of consolidation yield stress ($p_{cv}$ and $p_{ch}$), coefficient of consolidation ($c_v$ and $c_h$), coefficient of volume compressibility ($m_v$ and $m_h$) and hydraulic conductivity ($k_v$ and $k_h$). Here, the subscripts “v” and “h” indicate the values in the vertical and the horizontal directions, respectively. From CRS-V-V test, $p_{cv}$, $c_v$, $m_v$, and $k_v$ can be obtained, from CRS-H-V test, $p_{ch}$, $c_{hv}$, $m_h$ and $k_h$ can be obtained, and from CRS-V-R test, $p_{cv}$, $c_{hv}$, $m_v$ and $k_h$ can be obtained. $c_{hh}$ means the coefficient of consolidation with horizontal drainage and horizontal loading, and $c_{hv}$ means the coefficient of consolidation with horizontal drainage and vertical loading. The test results as well as the discussions on the suitability of the test method for obtaining the horizontal consolidation properties of clayey deposits and anisotropic consolidation behavior of Ariake clay are presented.

5.2 Consolidation yield stress ($p_{cv}$ and $p_{ch}$)

It should be referred that two frictions exist in CRS test with radial drainage comparing with CRS test with vertical drainage. One is the friction of “o”-ring between loading cap and guide ring, and the other is the friction between the soil sample and the porous stone in the middle of the soil sample. The friction of “o”-ring between loading cap and guide ring can be measured when the CRS tests were conducted without soil sample.

Figures 5.1 and 5.2 show the measured friction induced by the “o”-ring between loading cap and guide ring. From Figs. 5.1 and 5.2, it can be seen that the average friction is about 8 kPa when the strain exceed 2.5%. Measured friction induced by the “o”-ring is deducted from the axial loading measurement to provide a precise estimation of the axial load on the soil sample.
Figure 5.3 to 5.6 show typical stress-strain relationships of three types of CRS test for soil samples from BH-5, BH-6 and BH-7 at different depth. It can be seen that the $p_{ch}$ obtained from CRS-H-V test is smaller than $p_{cv}$ obtained from CRS-V-V and CRS-V-R tests. This is expected since for a normally or lightly overconsolidated clay deposit, the maximum horizontal effective stress a soil element experienced is smaller than that of in the vertical direction.
Fig. 5.3 Comparison of stress-strain relation (BH-5 15.5-16.3 m)

Fig. 5.4 Comparison of stress-strain relation (BH-6 17-17.8 m)

Fig. 5.5 Comparison of stress-strain relation (BH-7 9-9.85 m)
Figure 5.7 shows the ratio of $p_{ch}$ to $p_{cv}$ for all the soil samples tested. Majority of the data are in the range of 0.5 to 1.0, and the average value (excluding 2 larger than 1.0 points) is about 0.7. Based on the experimental results, the following expression had been proposed to calculate the coefficient of earth pressure at rest ($K_0$) (Mayne and Kulhawy 1982):

$$K_{0(OC)} = (1 - \sin \phi') \text{OCR} \sin \phi'$$

(5.1)

where $K_{0(OC)}$ = the $K_0$ value for overconsolidated state; OCR = the over consolidation ratio and $\phi'$ = the angle of internal friction.

For Ariake clay, the $p_c$ values from CRS tests with a strain rate of 0.02%/min are comparable with those of IL test. So the $p_c$ values obtained from CRS-V-V tests with the strain rate of 0.02%/min are used to calculate the OCR. The calculated values of OCR are in the range of 1.0 to 2.0, and the average value is about 1.5. This number is close to the back calculated results using measured data of embankment settlement (Chai and Miura 1999).

Assuming $\phi' = 30^\circ$, and OCR = 1.5, From Eq. (5.1), $K_{0(OC)}$ is about 0.61, which is smaller but close to 0.7 obtained from the test results.
Figure 5.8 shows the ratio of $p_{cv}$ obtained from CRS-V-R test to that from CRS-V-V test for all the soil samples tested. The linear regression show that $p_{cv}$ obtained from CRS-V-R test is about 1.04 times of that from CRS-V-V test. Although numerically there is some difference on $p_{cv}$ from the two types of test, it can be said that they are almost identical. The possible reason for slightly larger $p_{cv}$ value from CRS-V-R test may be the effect of the friction between the soil specimen and the central annular drainage porous stone.

5.3 Coefficient of consolidation ($c_v$, $c_{hh}$ and $c_{hv}$)

The results of $c_v$, $c_{hh}$ and $c_{hv}$ obtained from three types of CRS test for soil samples from BH-5, BH-6 and BH-7 are compared in Figs. 5.9 to 5.12. In the overconsolidated range the data are scattered, but in the virgin consolidation range $c_{hh}$ and $c_{hv}$ obtained from CRS-H-V and CRS-V-R test are obviously bigger than $c_v$ obtained from CRS-V-V test.
Fig. 5.9 Comparison of $c_v$, $c_{hh}$ and $c_{hv}$ (BH-5 15.5-16.3 m)

Fig. 5.10 Comparison of $c_v$, $c_{hh}$ and $c_{hv}$ (BH-6 17-17.8 m)

Fig. 5.11 Comparison of $c_v$, $c_{hh}$ and $c_{hv}$ (BH-7 9-9.85 m)
The values of $c_v$ under $\overline{\sigma}'_v = 300$ kPa (in the virgin compression range for all the soil samples tested) were chosen for investigating the relation between $c_{hh}$ and $c_v$, and the results are shown in Fig. 5.13 (excluding 3 extremely scattered points). It can be seen that $c_{hh}$ is larger than $c_v$ and the linear regression resulted in a ratio $c_{hh}/c_v$ of 1.38.

Figure 5.14 shows the relation between $c_{hv}$ and $c_v$ (excluding 4 extremely scattered points). It can be seen that $c_{hv}$ is larger that $c_v$ also, and the linear regression shows that the ratio of $c_{hv}$ to $c_v$ is about 1.54. CRS-V-R test resulted in a higher $c_{hv}$ value than $c_{hh}$ obtained from CRS-H-V test. From Figs. 5.13 and 5.14, there is no clear trend of difference on $c_{hh}/c_v$ or $c_{hv}/c_v$ values for different strain rate. Therefore, it can be said that for Ariake clay there is no anisotropic behavior of strain rate effect. It is interesting to know that the difference on $c_v$, $c_{hh}$ and $c_{hv}$ is from $m$ or $k$. Comparison of $m$ and $k$ will be made in the next sections.
5.4 Coefficient of volume compressibility ($m_v$ and $m_h$)

The comparison of $m_v$ and $m_h$ for soil samples from BH-5, BH-6 and BH-7 at different depth are presented in Figs. 5.15 to 5.18. In overconsolidated range, $m_v$ is obviously less than $m_h$. However, in virgin consolidation range the $m_v$ and $m_h$ are almost the same, especially for $\bar{\sigma}_v' > 200$ kPa cases.
Fig. 5.16 Comparison of $m_v$ and $m_h$ (BH-6 17-17.8 m)

Fig. 5.17 Comparison of $m_v$ and $m_h$ (BH-7 9-9.85 m)

Fig. 5.18 Comparison of $m_v$ and $m_h$ (BH-7 14-14.75 m)
5.5 Hydraulic conductivity ($k_v$ and $k_h$)

Figures 5.19 to 5.22 show the comparison of $k$ for soil samples from BH-5, BH-6 and BH-7 at different depth. In the overconsolidated range the data are scattered, but in the virgin consolidation range $k_h$ obtained from CRS-H-V and CRS-V-R test is obviously larger than $k_v$ obtained from CRS-V-V test.

Fig. 5.19 Comparison of $k_v$ and $k_h$ (BH-5 15.5-16.3 m)

Fig. 5.20 Comparison of $k_v$ and $k_h$ (BH-6 17-17.8 m)
The same as for the coefficient of consolidation, the ratio of $k_h$ to $k_v$ is investigated at $\sigma'_v = 300$ kPa. All the test results (excluding 3 extremely scattered points, as for $c_v$ in Fig. 5.13) of $k_h$ from CRS-H-V test and $k_v$ are shown in Fig. 5.23. The linear regression shows that the ratio of $k_h$ to $k_v$ is about 1.34. This value is very close to $c_{hh}/c_v$ of 1.38 in Fig. 5.13, which indicates that $k$ is the main influencing factor for anisotropic coefficient of consolidation.

Park (1994) conducted the IL test with Ariake clay samples cut vertically and horizontally with respect to in-situ condition, and reported that $k_h/k_v$ ratio was about 1.5. This number is larger than 1.34 from this study but comparable. Leroueil et al. (1990) investigated the hydraulic conductivity anisotropy of several clayey soils by directly measuring the $k_h$ and $k_v$ values at different strain level, and concluded that permeability anisotropy does not vary significantly for strain up to 25%. Therefore, it is considered that $k_h/k_v$ and $c_{hh}/c_v$ ratios obtained for $\sigma'_v = 300$ kPa represent the general anisotropic consolidation behavior of Ariake clay.
Fig. 5.23 The ratio of $k_h$ from CRS-H-V test to $k_v$ from CRS-V-V test

Figure 5.24 shows the ratio of $k_h$ obtained from CRS-V-R test to $k_v$ for all the soil samples tested (excluding 4 extremely scattered points, as for $c_v$ in Fig. 5.14). The linear regression shows that the ratio of $k_h$ to $k_v$ is about 1.65. This number is slightly higher than $c_h/c_v$ of 1.54 in Fig. 5.14, but we consider that they are very close. It is not clear on why $k_h$ value obtained from CRS-V-R test is larger than that obtained from CRS-H-V test. A possible explanation is the difference on deformation pattern (direction). CRS-V-R test deforms vertically, but CRS-H-V test deforms horizontally. The later may destroy the micro-structure formed during the deposition process more easily.

From above comparisons, now we understand that: (1) Ariake clay has an anisotropic consolidation behavior and it is mainly due to the anisotropic hydraulic conductivities, i.e. $k_h > k_v$. (2) CRS-V-R tests result in larger $k_h$ values than that from CRS-H-V tests, and it may be because that the different compression directions for the soil sample in CRS-H-V and CRS-V-R test. The practical implication of (2) is: using the $c_h$ value obtained from the
test results of horizontally cut soil samples with vertical drainage to design PVD improvement, the $c_h$ value may be underestimated.

Most sedimentary clayey deposits exhibit anisotropic hydraulic conductivities. Leroueil et al. (1990) reported $k_h/k_v$ ratios of 5 marine clays of about 1.10 to 1.55 from directly measured $k_v$ and $k_h$ values. Yune and Chung (2005) reported $k_h/k_v$ value of 1.7 for undisturbed Korea clay and 1.3 for reconstituted Korea clay from both vertical and radial drainage CRS test results. Sea and Juimarongrit (2003) conducted CRS test with vertical and radial drainage for Bangkok clay, and $k_h/k_v$ of 1.45 was resulted. Therefore $k_h/k_v$ values obtained in this study are comparable with the values in the literature.

Figure 5.25 show the variations of $k_h/k_v$ ($k_h$ obtained from CRS-H-V test) with depth for the soil samples from three BHs at strain rate 0.02%/min.

![Variations of $k_h/k_v$ with depth](image)

**Fig. 5.25 Variations of $k_h/k_v$ with depth**

From Fig. 5.25, it can be seen that the soil samples tested in this study don’t show an obvious variations of anisotropy ($k_h/k_v$) with depth. A large number of tests need be conducted to investigate the variation of anisotropy with depth.

### 5.6 Excess pore water pressure ($u_b$ and $u_e$)

Figures 5.26 to 5.29 show the variation of excess pore water pressure of three types of CRS tests for soil samples from BH-5, BH-6 and BH-7 at different depth. At a given strain, the $u_b$ obtained from CRS-H-V test is lower than that from CRS-V-V test because of higher $k_h$ value. At a given strain, the $u_e$ is higher than $u_b$. The theoretical ratio of $u_e$ to $u_b$ can be expressed by the following equation:

$$
\frac{u_e}{u_b} = \frac{(r_e - r_w)^2}{2H^2r_w} \frac{k_h}{k_v}
$$

(5.2)

Figure 5.30 shows that the ratios of $u_e$ to $u_b$ for all the CRS-V-V and CRS-V-R test results at a vertical strain of 15%. The data points are in the range of 1.5-8, and the average value is about 3.57. At 15% vertical strain, $H = 17$ mm. Using $k_h/k_v = 1.65$ (Fig. 5.24), the calculated theoretical ratio of $u_e$ to $u_b$ is about 4.6, which is within the range of the
measured data but larger than the simple average value.

Fig. 5.26 Comparison of $u_b$ and $u_e$ (BH-5 15.5-16.3 m)

Fig. 5.27 Comparison of $u_b$ and $u_e$ (BH-6 17-17.8 m)
Fig. 5.28 Comparison of $u_b$ and $u_e$ (BH-7 9-9.85 m)

Fig. 5.29 Comparison of $u_b$ and $u_e$ (BH-7 14-14.75 m)

Fig. 5.30 The ratio of $u_e$ from CRS-V-R test to $u_b$ from CRS-V-V test
5.7 Strain rate effect on horizontal consolidation characteristics

5.7.1 Strain rate effect on $p_{ch}$

Figure 5.31 shows the strain rate effect on $p_{ch}$ obtained from CRS-H-V test. Majority of the values are in the range of 1.0 to 1.3, and the average value is about 1.15 (that is, about 15% increase in $p_{ch}$ with a tenfold increase in the strain rate). This is almost the same as the value of strain rate effect on $p_{cv}$ (Fig. 4.11).

![Fig. 5.31 Strain rate effect on $p_{ch}$](image)

5.7.2 Strain rate effect on $c_{ch}$ and $k_{ch}$

Figure 5.32 shows the strain rate effect on $c_{ch}$ obtained from CRS-H-V and CRS-V-R test (excluding 6 extremely scattered points). Majority of the values are bigger than 1, and the average value is about 1.25 (that is, about 25% increase in $c_{ch}$ with a tenfold increase in the strain rate). This is very close to the value of strain rate effect on $c_{cv}$ (Fig. 4.37).

![Fig. 5.32 Strain rate effect on $c_{ch}$](image)
Figure 5.33 shows the strain rate effect on $k_h$ obtained from CRS-H-V and CRS-V-R test (excluding 6 extremely scattered points). Majority of the values are bigger than 1, and the average value is about 1.25 (that is, about 25% increase in $k_h$ with a tenfold increase in the strain rate). This is very close to the value of strain rate effect on $c_h$, which means that the increase of $c_h$ with increasing strain rate is due mainly to the increase of $k_h$.

\[ k_{h25}\%/min / k_{h0.02}\%/min \]

\[ 0.0 \quad 0.5 \quad 1.0 \quad 1.5 \quad 2.0 \quad 2.5 \quad 3.0 \quad 3.5 \quad 4.0 \quad 4.5 \]

\[ 0 \quad 5 \quad 10 \quad 15 \quad 20 \quad 25 \]

- BH-5 CRS-H-V
- BH-5 CRS-V-R
- BH-6 CRS-H-V
- BH-6 CRS-V-R
- BH-7 CRS-H-V

**Fig. 5.33 Strain rate effect on $k_h$**

### 5.8 Summary

Based on the test and analysis results presented in this chapter, the following conclusions can be drawn.

1. The ratio of consolidation yield stress in the horizontal direction ($p_{ch}$) from CRS test using horizontally cut sample with vertical drainage (CRS-H-V) to the consolidation yield stress in the vertical direction ($p_{cv}$) from CRS test using vertically cut sample with vertical drainage (CRS-V-V) is in the range of 0.5 to 1.0, with an average value of about 0.7. $p_{cv}$ from CRS test using vertically cut sample with radial drainage (CRS-V-R) is almost same with that $p_{cv}$ from CRS-V-V test.

2. Ariake clay has an anisotropic consolidation behavior. The coefficient of consolidation in the horizontal direction ($c_{hh}$ from CRS-H-V test and $c_{hv}$ from CRS-V-R test) is larger than that in the vertical direction ($c_v$). This anisotropic consolidation behavior is mainly from the anisotropy of hydraulic conductivity ($k$). The ratio of $k$ in the horizontal direction ($k_h$) from CRS-V-R test to that in the vertical direction ($k_v$) from CRS-V-V test is about 1.65 and the ratio of $c_{hv}/c_v$ is about 1.54.

3. The ratio of $k_h$ from CRS-H-V test to $k_v$ is about 1.34 and the ratio of $c_{hh}/c_v$ is about 1.38. Therefore the $k_h$ and $c_{hh}$ from CRS-H-V test is smaller than $k_h$ and $c_{hv}$ from CRS-V-R test. It is considered that this difference may be resulted from different deformation pattern during consolidation. The micro-structure formed during deposition process may be destroyed more easily in CRS-H-V test. This implies that using the $c_{hh}$ obtained from CRS-H-V test to design PVD improvement, it may underestimate the degree of consolidation due to PVD.
(4) The degree of strain rate effect on horizontal consolidation characteristics ($p_{ch}$, $c_h$ and $k_h$) is almost the same with the strain rate effect on vertical consolidation characteristics ($p_{cv}$, $c_v$ and $k_v$).
6 NUMERICAL INVESTIGATIONS AND DISCUSSIONS

6.1 Introduction

There are two theories for interpreting CRS test results, i.e. small strain theory (Smith and Wahls, 1969; Aboshi et al., 1970; Wissa et al., 1971) and large strain theory (Umehara and Zen, 1980; Lee, 1981; Znidarcic et al., 1986). Small strain theory is simple and is adopted into the standard for CRS test (JSA, 2000b; ASTM, 2006). However, under a consolidation pressure of few hundreds kPa or even 1000 kPa, for a soft clay sample with an initial water content more than 100%, the strain can be as large as 30%. This kind of strain cannot be regarded as a small strain and large strain theory will be much more appropriate. Therefore, it is needed to clarify that under what kind condition the small strain theory can yield an acceptable result, and for what kind of condition, the large strain theory may result in a better or more realistic result.

In this chapter, the strain distribution within the sample of CRS test obtained by small strain theory and large strain theory were compared with the results of numerical simulations. The strain rate effect on $c_v$ calculated by large strain theory was compared with that of small strain theory, and the applicable range of small strain theory is suggested.

6.2 Strain distribution within the sample of CRS test

6.2.1 Small strain theory

Assuming the strain is infinitesimal and $c_v$ is constant, Wissa et al. (1971) obtained the strain distribution within a sample subjected to a constant rate of strain as follows:

$$\varepsilon(z,t) = rt + \frac{rH^2}{6c_v}\left(3 \frac{z^2}{H^2} - 6 \frac{z}{H} + 2\right) - \frac{2rH^2}{\pi^2 c_v} \sum_{n=1}^{\infty} \cos n\frac{\pi z}{H} \exp\left(-n^2 \pi^2 T_v\right)$$

where $\varepsilon$ = vertical strain; $r$ = strain rate; $t$ = time; $H$ = height of the sample; $c_v = k/\gamma_w m_v$ ($k$ = the hydraulic conductivity; $\gamma_w$ = the unit weight of water; and $m_v$ = the coefficient of volume compressibility); $z$ = the vertical coordinate of a point; $T_v = \text{time factor } (T_v = c_v t/H^2)$.

If the transient period is passed ($T_v \geq 0.5$) (Wissa et al., 1971), the last part in Eq. (6.1) becomes negligible. Thus Eq. (6.1) becomes:

$$\varepsilon(z,t) = rt + \frac{rH^2}{6c_v}\left[1 - \frac{1}{6}\left(3 \frac{z^2}{H^2} - 6 \frac{z}{H} + 2\right)\right]$$

For $c_v/rH^2 = 100, 10$ and $1$, the results are shown in Figs. 6.1, 6.2 and 6.3. The strain distribution pattern varies with $r$. Also from Eq. (6.2) and Figs. 6.1, 6.2 and 6.3, it can be seen that the strain distribution pattern is parabolic. For a given soil sample, at any time, the difference between the strain at the top and at the bottom of the sample is a function of strain rate only and can be expressed as $\Delta \varepsilon = (1/2)rH^2/c_v$. At any point, the difference between the strains at any two times $t_1$ and $t_2$ is $r(t_2 - t_1)$. 

- 81 -
Fig. 6.1 Strain distributions (small strain theory $c_v/rH^2 = 100$)

Fig. 6.2 Strain distributions (small strain theory $c_v/rH^2 = 10$)

Fig. 6.3 Strain distributions (small strain theory $c_v/rH^2 = 1$)
6.2.2 Large strain theory

In engineering practice, under an embankment or a structure load the strain in clay deposit can up to 10 - 30%. Therefore the assumption of infinitesimal strain may not represent the actual situation. Mikasa (1963) proposed a large strain consolidation theory considering the change of the thickness of the sample during consolidation. The “original coordinate” $z_0$ and “consolidation ratio” $\zeta = (1 + e_0)/(1 + e)$ (where $e$ is void ratio $e_0$ is the initial void ratio) were introduced. $\zeta$ can be related to the axial strain $\varepsilon$ as $\zeta = 1/(1 - e)$. With the variable $\zeta$, the governing equation becomes:

$$\frac{\partial \zeta}{\partial t} = c_0 \frac{\partial^2 \zeta}{\partial z_0^2}$$  \hspace{1cm} (6.3)

Eq. (6.3) is a non-linear partial differential equation of second order and cannot be solved explicitly. Using finite difference method, Eq. (6.3) can be approximated as following:

$$\zeta(z_0, t + \Delta t) = \frac{c_0 \Delta t}{\Delta z_0^2} \bar{z}^2 \left[ \zeta(z_0 + \Delta z_0, t) - 2\zeta(z_0, t) + \zeta(z_0 - \Delta z_0, t) \right] + \zeta(z_0, t)$$  \hspace{1cm} (6.4)

where $\bar{z}$ is the average value of $z$ within a time increment $\Delta t$. To generalize the expression, the following dimensionless parameters are introduced.

$$T' = \frac{\Delta H_0}{H_0} = rt$$
$$Z_0 = \frac{z_0}{H_0}$$
$$n = \frac{1}{\Delta Z_0}$$

(6.5)

Using the dimensionless variables $(Z_0, T')$, Eq. (6.4) becomes:

$$\zeta(Z_0, T' + \Delta T') - \zeta(Z_0, T') = n \left( \frac{c_0}{rH_0^2} \right) \Delta T' \bar{z}^2$$  \hspace{1cm} (6.6)

$$\left[ \zeta(Z_0 + \Delta Z_0, T') - 2\zeta(Z_0, T') + \zeta(Z_0 - \Delta Z_0, T') \right]$$

The solutions of Eq. (6.6) can be obtained by solving the following two equations.

$$\zeta(Z_0, T' + \Delta T') = \zeta(Z_0, T') + \frac{\zeta(Z_0, T') \Delta \zeta''(Z_0, T')}{1 - \zeta(Z_0, T') \Delta \zeta''(Z_0, T')}$$  \hspace{1cm} (6.7)

$$\Delta \zeta''(Z_0, T') = n \left( \frac{c_0}{rH_0^2} \right) \Delta T' \left[ \zeta(Z_0 + \Delta Z_0, T') - 2\zeta(Z_0, T') + \zeta(Z_0 - \Delta Z_0, T') \right]$$  \hspace{1cm} (6.8)

So the strain distributions under different average strain $(rt)$ and $c_0/rH_0^2$ value can be obtained, and the results for $c_0/rH_0^2 = 100, 10$ and 1 are shown in Figs. 6.4, 6.5 and 6.6.
Fig. 6.4 Strain distributions (large strain theory $c_v/rH_0^2 = 100$)

Fig. 6.5 Strain distributions (large strain theory $c_v/rH_0^2 = 10$)

Fig. 6.6 Strain distributions (large strain theory $c_v/rH_0^2 = 1$)
From Figs. 6.4, 6.5 and 6.6, it can be seen that at higher strain rate (Fig. 6.6) the pattern of strain distribution changes with time. At an early stage ($\bar{\varepsilon} = 0.1$), the strain at the top of the sample is about 25 times of that at the bottom of the sample. At a later stage ($\bar{\varepsilon} = 0.5$), the strain at the top of the sample is about 1.26 times of that at the bottom of the sample. This indicates that during the test, the higher strain increment zone was located first in the upper part of the sample and then moved to the lower part of the sample.

Figure 6.7 shows the distributions of a strain increment of 0.001 at 3 different strain levels for $c_v/rH_0^2 = 1$. At an early stage ($\bar{\varepsilon}: 0.099-0.1$), the strain increment at the top of the sample is larger than that at the bottom of the sample. At a later stage ($\bar{\varepsilon}: 0.499-0.5$), the strain increment at the bottom becomes larger. So the strain distribution pattern becomes less non-uniform with the increase of average strain.

6.2.3 Comparison of the theoretical strain distribution with numerical results

To investigate the strain distribution pattern using numerical simulation, a correct void ratio ($e$) versus consolidation stress ($p'$) relationship is important in numerical modeling. For most natural clay deposits, in $e - \ln p'$ plot, the compression curves are non-linear (Burland, 1990). Chai et al. (2004) found that in $\ln(e + e_c) - \ln p'$ plot ($e_c$ is a constant and depends on soil type), the compression curves are very close to linear for most natural deposits. In numerical simulation, modified Cam clay model (MCC) (Roscoe and Burland, 1968) with both linear $e - \ln p'$ and linear $\ln(e + e_c) - \ln p'$ relationships were used to model the mechanical behavior of Ariake clay and the results were compared.

(1) Simulating CRS consolidation test

For the CRS test with the sample from BH-1 at 5.0-5.9 m depth with a strain rate of 0.02%/min were simulated. The program used is a modified version of CRISP ( Britto and Gunn, 1987), M-CRISP. The values of model parameters adopted are given in Table 6.1, which were determined from test results.
Table 6.1 Parameters for finite element analysis

<table>
<thead>
<tr>
<th>Description</th>
<th>ν</th>
<th>κ</th>
<th>κ₁</th>
<th>λ</th>
<th>λ₁</th>
<th>M</th>
<th>( p'_y ) (kPa)</th>
<th>( e_0 )</th>
<th>( e_c )</th>
<th>( c_k )</th>
<th>( k_0 ) (m/day)</th>
<th>( \sigma'_{v_0} ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( e - \ln p' )</td>
<td>0.3</td>
<td>0.14</td>
<td>—</td>
<td>0.67</td>
<td>—</td>
<td>1.2</td>
<td>42.5</td>
<td>3.51</td>
<td>—</td>
<td>1.4</td>
<td>4.0E-04</td>
<td>20</td>
</tr>
<tr>
<td>( \ln (e + e_c) - \ln p' )</td>
<td>0.3</td>
<td>—</td>
<td>0.05</td>
<td>—</td>
<td>0.30</td>
<td>1.2</td>
<td>42.5</td>
<td>3.51</td>
<td>-0.63</td>
<td>1.4</td>
<td>4.0E-04</td>
<td>20</td>
</tr>
</tbody>
</table>

Note: ν, Poisson’s ratio; κ, slope of unloading-reloading line in \( e - \ln p' \) plot; κ₁, slope of unloading-reloading line in \( \ln (e + e_c) - \ln p' \) plot; λ, slope of virgin consolidation line in \( e - \ln p' \) plot; λ₁, slope of virgin consolidation line in \( \ln (e + e_c) - \ln p' \) plot; \( M \), slope of critical state line in \( q-p' \) plot (\( q \) is deviator); \( p'_y \), size of the yield locus; \( e_0 \), initial void ratio; \( k_0 \), initial hydraulic conductivity; \( \sigma'_{v_0} \), initial effective vertical stress.

A reference for the method of calculating κ₁, λ₁ and \( e_c \) is given (Chai et al., 2004). The value of \( p'_y \) is calculated from:

\[
p'_y = \left[ 1 + \frac{\left( p'_y - K_0 p' \right)^2}{\left( 1 + 2 K_0 \right) p'_y / 3} \right] \cdot \left( 1 + 2 K_0 \right) p'_y / 3 \]

(6.9)

where \( p'_c \) = the consolidation yield stress of the simulated sample; \( K_0 = \) the coefficient of earth pressure at rest; \( M = \) the slope of critical state line in Cam clay model. The hydraulic conductivities listed in Table 6.1 are initial values, and during the consolidation it varied with \( e \) according to Taylor’s equation (Taylor, 1948):

\[
k = k_0 10^{(e - e_0)/c_k} \]

(6.10)

where \( k_0 \) and \( k = \) hydraulic conductivities corresponding to \( e_0 \) and \( e \), respectively; and \( c_k = \) a constant, which can be estimated as 0.4\( e_0 \) (Tavenas et al., 1983).

The finite element mesh and the boundary conditions used are shown in Fig. 6.8. The thicknesses of the element varied from 1 to 4 mm. Eight-node quadrilateral elements with 4 pore pressure nodes were adopted. Further, in FEM analysis, the node coordinates were updated at the end of each step to approximately consider the effect of large deformation.

---

Fig. 6.8 Mesh used in the FEM simulation
The measured and simulated vertical effective stress versus vertical strain curves and excess pore water pressures at the bottom of sample are compared in Figs. 6.9 and 6.10 respectively.

Using linear $\ln(e - 0.63) - \ln p'$ relation resulted in a better simulation than that using linear $e - \ln p'$ relation. Then, the linear $\ln(e - 0.63) - \ln p'$ relationship was used in simulating strain distribution pattern under CRS test condition.

![Fig. 6.9 Comparison of the stress-strain curves](image1)

**Fig. 6.9 Comparison of the stress-strain curves**

![Fig. 6.10 Comparison of excess pore water pressure variation](image2)

**Fig. 6.10 Comparison of excess pore water pressure variation**

(2) Simulated strain distribution patterns

In both the small strain and large strain theories, $c_v$ is assumed as a constant. In FEM analysis, the $c_v$ is not a direct input parameter. Instead the hydraulic conductivity ($k$) and the parameter for compressibility are inputted. With a linear $\ln(e + e_c) - \ln p'$ relationship, $m_v$ can be expressed as:

\[
m_v = (e + e_c) \lambda_v / (1 + e) p'
\]

(6.11)

using Eqs. (6.10) and (6.11), $c_v$ can be calculated.
Although by considering the change of $m_v$ and $k$ during consolidation process, the variation of $c_v$ can be reduced, but there is no guarantee that $c_v$ is a constant. As an approximation, $c_v$ value corresponding to the average effective vertical stress of whole virgin consolidation process is used as the representative value for obtaining the different $c_v/rH_0^2$ values.

The simulated strain distributions are compared with those from the small and large strain theories in Figs. 6.11, 6.12 and 6.13 for $c_v/rH_0^2 = 100$, 10 and 1 respectively.

![Fig. 6.11 Comparison of strain distributions ($c_v/rH_0^2 = 100$)](image)

![Fig. 6.12 Comparison of strain distributions ($c_v/rH_0^2 = 10$)](image)
Fig. 6.13 Comparison of strain distributions ($c_v/rH_0^2 = 1$)

From Figs. 6.11, 6.12 and 6.13, it can be seen that the strain distributions obtained by the large strain theory compare well with the numerical simulations. The strain distributions obtained by the small strain theory at higher strain rate ($c_v/rH_0^2 = 1$) is quite different from that of numerical results, and they are more non-uniform.

6.3 Comparing interpreted $c_v$ values from CRS test

6.3.1 Comparing interpreted $c_v$ values

The comparison of $c_v$ values calculated by small and large strain theories for soil samples from BH-1 at 5.0-5.9 m and 6.0-6.9 m depths are shown in Figs. 6.14 to 6.17. Figures 6.14 and 6.15 are for strain rate of 0.02%/min, and Figs. 6.16 and 6.17 are for strain rate of 0.2%/min.
Fig. 6.15 Comparison of $c_v$ (BH-1 6-6.9 m, 0.02%/min)

Fig. 6.16 Comparison of $c_v$ (BH-1 5-5.9 m, 0.2%/min)

Fig. 6.17 Comparison of $c_v$ (BH-1 6-6.9 m, 0.2%/min)
It can be seen that $c_v$ values calculated by the large strain theory are smaller than the values of from the small strain theory. For a CRS test, at a given time $t$, the average strain ($\Delta H_0 / H_0$) and the strain ratio $\varepsilon_B / \varepsilon_T$ can be easily calculated. The ratio of the strain at the bottom to that at the top of a sample ($\varepsilon_B / \varepsilon_T$) under different strain rate by large strain theory is shown in Fig. 6.18. For the purpose of comparison, the similar graph of small strain theory is calculated and shown in Fig. 6.19.

Comparing Figs. 6.18 and 6.19, it can be found that for a given value of $\Delta H_0 / H_0$ and $\varepsilon_B / \varepsilon_T$, a larger $c_v / rH_0^2$ value can be interpolated from Fig. 6.19 (small strain theory) than that from Fig. 6.18 (large strain theory), and therefore a larger $c_v$ value. In other words, for a given $\Delta H_0 / H_0$ and $c_v / rH_0^2$ value, the small strain theory will yield a smaller $\varepsilon_B / \varepsilon_T$ (more non-uniform strain distribution) and it does not represent the actual case (comparing with numerical simulation results).

The values of $c_v$ in the virgin compression range were chosen for a detailed investigation of the interpreted $c_v$ by small strain and large strain theory. BH-1, 2, 3 and 4
under a given average effective vertical stress $\bar{\sigma}_v' = 100$ kPa and 200 kPa, and BH-5, 6, 7 $\bar{\sigma}_v' = 300$ kPa. The comparison of the $c_v$ obtained by small strain and large strain theory are shown in Fig. 6.20 (0.02%/min) and Fig. 6.21 (0.2%/min). From Figs. 6.20 and 6.21, we can see that the $c_v$ values calculated by the large strain theory is about 0.5 times smaller than that by the small strain theory and the difference of $c_v$ calculated by the large strain theory and small strain theory increase with the increase of strain rate.

![Fig. 6.20 Comparison of the $c_v$ (0.02%/min)](image)

![Fig. 6.21 Comparison of the $c_v$ (0.2%/min)](image)

### 6.3.2 Comparing interpreted strain rate effect on $c_v$

The variation of strain rate effect on $c_v$ ($c_{v0.2\%/min} / c_{v0.02\%/min}$) in virgin compression range at different depth is shown in Fig. 6.22 for values both interpreted by small strain and large strain theory. Although the data are scattered, there is a tendency that the large strain theory results in less strain rate effects.
The $c_v$ values interpreted by the large strain theory are normalized by the value of strain rate of 0.02%/min, and plotted in Fig. 6.23. The linear regression gives about 14% increase for a tenfold increase in the strain rate, and it is less than the value interpreted by small strain theory (30% in Fig. 6.24). So the strain rate effect on $c_v$ is depend on which strain theory is used. For high strain rate, large strain theory should be used, it result in less strain rate effect on $c_v$. So the value (14%) may be more appropriate to use in the numerical analysis. The $c_v$ increased with the increase of strain rate resulting from the increase of hydraulic conductivity ($k$) due to the coefficient of volume compressibility ($m_v$) was almost same for different strain rates (Figs. 4.38 to 4.48).
6.4 Summary

Based on the analytical and numerical results in this chapter, the following conclusions can be drawn.

(1) Small strain theory and large strain theory result in quite different strain distribution in a soil sample under constant rate of strain (CRS) test condition and the difference increases with the increase of strain rate.

(2) Comparing the results of numerical analysis with that of small strain theory and large strain theory, indicates that the small strain theory can only be used for small strain rate ($r/c_v H^2 \geq 10$, $c_v$ is the coefficient of consolidation, $H$ is the thickness of the soil sample) and the large strain theory can be used for a larger range of strain rates.

(3) The theoretical strain distribution patterns influence the calculated values of $c_v$ from CRS test results. It has been clarified that the large strain theory results in smaller $c_v$ value and less strain rate effect (14% increase of $c_v$ for a tenfold increase in the strain rate).
7 CONCLUDING REMARKS

7.1 Conclusions

This dissertation mainly investigates the strain rate effect on consolidation behavior and anisotropic consolidation behavior of Ariake clay by constant rate of strain (CRS) tests. The theories for interpreting CRS test results are compared and their applicable conditions are also suggested. Based on the test, numerical and analysis results, the following conclusions can be drawn.

(1) About the strain rate effect on consolidation behavior

(a) The consolidation stress–strain curves are parallel in \( e - \log \sigma_v' \) plot (\( e \) is void ratio and \( \sigma_v' \) is effective vertical stress), which indicates that isotach model is applicable to Ariake clay. Consolidation yield stress \((p_c)\) of Ariake clays increased by about 14% with a tenfold increase in the strain rate.

(b) For a given strain level, the strain rate does not influence the compression index \((C_c)\), as the isotach model implies. While for a given average effective stress, there is a tendency for \(C_c\) to increase with strain rate due to the non-linearity of \( e - \log \sigma_v' \) relation.

(c) Under a given effective vertical stress, the coefficient of consolidation \((c_v)\) increased with the increase of strain rate resulting mainly from the increase of the hydraulic conductivity \((k)\). Linear regression results in about a 30% increase of \(c_v\) for a tenfold increase in the strain rate when using small strain theory to interpret the CRS test results.

(d) Comparing the \(p_c\) and \(c_v\) values from the incremental loading (IL) and CRS tests suggests that the \(p_c\) and \(c_v\) values from CRS tests for a strain rate of 0.02%/min are comparable with those of IL tests. In \( e - \log \sigma_v' \) plot, the compression curve from the CRS test is more non-linear than that from the IL test, and the largest difference in \(C_c\) value from the CRS and IL tests occurred around the value of \(p_c\).

(e) There is a clear relationship between the strain rate effect (SRE) and clay content. Increase of clay content, increases SRE.

(2) About the anisotropic consolidation behavior

(a) The ratio of consolidation yield stress in the horizontal direction \((p_{ch})\) to that in the vertical direction \((p_{cv})\) is in the range of 0.5 to 1.0, and the average value is about 0.7. \(p_{cv}\) value from radial drainage CRS test is almost same with that from vertical drainage CRS test.

(b) The coefficient of consolidation in the horizontal direction \((c_h)\) is larger than that in the vertical direction \((c_v)\), and it is mainly from the anisotropy of hydraulic conductivity \((k)\). The ratio of \(k\) in the horizontal direction \((k_h)\) from radial drainage CRS test to that in the vertical direction \((k_v)\) is about 1.65.

(c) The \(k_h\) value from CRS test using horizontally cut sample with vertical drainage is smaller than that from CRS test using vertically cut sample with radial drainage. The possible reason is the different deformation pattern of the two types of tests. The micro-structure formed during deposition process may be destroyed more easily in CRS test using horizontally cut sample with vertical drainage.
(d) The degree of strain rate effect on horizontal consolidation characteristics \((p_{ch}, c_h \text{ and } k_h)\) is almost the same with that of vertical consolidation characteristics \((p_{cv}, c_v \text{ and } k_v)\).

(3) About the theories for interpreting CRS test results

(a) Small strain theory and large strain theory result in quite different strain distribution in a soil sample under constant rate of strain (CRS) test condition and the difference increases with the increase of strain rate.

(b) Comparing the results of numerical analysis with the theoretical results indicates that the small strain theory can only be used for small strain rate \(r\) with \(c_v/rH^2 \geq 10\) (\(H\) is the thickness of soil sample) and the large strain theory can be used for a larger range of strain rate.

(c) The strain distribution patterns influence the calculated values of \(c_v\) from CRS test results. It has been clarified that the large strain theory results in smaller \(c_v\) value and less strain rate effect on \(c_v\) value.

7.2 Future works

The following tasks are suggested for future study on the consolidation behavior of soft clay under constant rate of strain.

(a) In this study, the strain rate adopted for CRS test is in the range of 0.02%/min to 0.2%/min. The CRS test with a strain rate less than 0.02%/min needs to be conducted to further investigate the strain rate effect on the consolidation behavior of Ariake clay.

(b) The correlation between strain rate effect and fundamental soil properties (e.g. plasticity index \(I_p\)) need to be investigated. In this study, it was found that no correlation exists between the strain rate effect and \(I_p\). However, the sample for CRS test and liquid and plastic limit test were different. Same sample for CRS test and liquid and plastic limit test need be used to investigate the strain rate effect related to \(I_p\).

(c) \(k\) and therefore \(c_v\) increase with increasing strain rate implies that the rate of consolidation is a function of strain rate. More researches need to be done for developing a consolidation theory considering the strain rate effect. In civil engineering design there is a tendency to adopting performance design rather than prescribed design. The key point of performance design is accurate prediction of the performance of the target structure. If a consolidation model considering the strain rate effect can be established, it can certainly help to increase the accuracy of consolidation analysis.

(d) Large strain theory is more appropriate for interpreting CRS test result under large strain rate. However only small strain theory is adopted into the standard for CRS test due to there is no explicit equation for calculating \(c_v\) by the large strain theory. Currently, the diagram method is used and it is inconvenient. A simple and easy to use method need be developed.
REFERENCES


JSA (2000b) JIS A 1227. Test method for one-dimensional consolidation properties of soils using constant rate of strain loading. JSA, Tokyo.


