A total of 114 constant rate of strain (CRS) consolidation tests and 15 incremental loading (IL) consolidation tests were conducted for undisturbed Ariake clay samples from three boreholes in the Saga Plain, Kyushu, Japan, to systematically investigate the strain-rate effect on the consolidation behaviour of Ariake clay. The test results show that the consolidation yield stress ($P_y$) of Ariake clays increased by about 15–16% with a tenfold increase in strain rate, and no clear correlation exists between the strain-rate effect and the clay content or plasticity index $I_p$ of the samples. For a given strain level, the strain rate does not influence the compression index $C_s$, which implies that an isotach model is applicable to Ariake clay. It has been newly found that, under a given effective vertical stress, the coefficient of consolidation $C_v$ increases with increase of the strain rate, resulting mainly from the increase of hydraulic conductivity $k$ with strain rate. Linear regression gives about a 33% increase in $C_v$, with a tenfold increase in the strain rate. For the Ariake clay tested, the $P_y$ and $C_v$ values of CRS tests with a strain rate of 0-0.2%/min are comparable with those of IL tests.

**NOTATION**

- $C_s$: compression index
- $C_i$: recompression index
- $c_v$: coefficient of consolidation
- $c_r$: void ratio
- $c_r$: void ratio of sample at time $t$
- $c_{r,t+\Delta t}$: void ratio of sample at time $t + \Delta t$
- $H$: thickness of sample
- $\Delta H$: change of sample thickness in time interval $\Delta t$
- $H_i$: thickness of sample at time $t$
- $H_{t+\Delta t}$: thickness of sample at time $t + \Delta t$
- $\Pi$: average thickness of sample between $t$ and $t + \Delta t$
- $i$: hydraulic gradient
- $k_v$: average hydraulic gradient
- $k_t$: threshold hydraulic gradient
- $k$: hydraulic conductivity
- $m_r$: coefficient of volume compressibility
- $P_y$: consolidation yield stress for a given strain rate $\varepsilon$
- $P_y(0.02/\min)$: consolidation yield stress under strain rate of 0.02%/min
- $q$: flow rate
- $RPC$: ratio of consolidation yield stress
- $SRE$: strain-rate effect
- $u_b$: excess pore water pressure at the bottom of the sample
- $u_{b,t}$: excess pore water pressure at bottom of sample at time $t$
- $u_{b,t+\Delta t}$: excess pore water pressure at bottom of sample at time $t + \Delta t$
- $u_b$: average excess pore water pressure at bottom of sample between $t$ and $t + \Delta t$
- $w_l$: liquid limit of sample
- $w_p$: natural water content of sample
- $w_f$: plastic limit of sample
- $\gamma_w$: unit weight of water
- $\varepsilon$: strain
- $\varepsilon$: strain rate
- $\sigma_v$: total vertical stress
- $\Delta \sigma_v$: total vertical stress increment in time interval $\Delta t$
- $\sigma_v^i$: effective vertical stress
- $\Delta \sigma_v^i$: effective vertical stress increment in a time interval of $\Delta t$
- $\sigma_v^{t+\Delta t}$: effective vertical stress at time $t$
- $\sigma_v^{t+\Delta t}$: effective vertical stress at time $t + \Delta t$
- $\sigma_v^{0.2/\min}$: effective vertical stress corresponding to strain rate of 0-2%/min
- $\sigma_v^{0.2/\min}$: effective vertical stress corresponding to strain rate of 0-0.2%/min
- $\bar{\sigma}_v$: average effective vertical stress

**1. INTRODUCTION**

For civil engineering projects related to soft clayey deposits, engineers invariably have to consider the consolidation behaviour of the deposit. This is because with regard to the natural viscosity of clayey soils, the consolidation behaviour of such soils is strongly influenced by strain rate. An understanding of the strain-rate effect on consolidation behaviour is useful for designing geotechnical projects such as embankments in areas of clayey deposits.

In geotechnical practice there are two test methods for determining the consolidation characteristics of a clayey soil: incremental loading (IL) and constant rate of strain (CRS) consolidation tests. The IL test is more convenient in use, and the equipment is relatively simple. However, the strain-rate effect can be investigated only by the CRS test. Also, in comparison with the IL test, the CRS test has several merits: for example, the test duration is shorter (about 1–2 days), and it...
provides continuous data points for a plot of void ratio $e$ against effective vertical stress $\sigma'_v$.

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. Crawford (1964) noticed the importance of strain-rate effects on the behaviour of clay, and the fact that consolidation test results depend on the testing method used. Subsequently, numerous CRS tests were performed on natural clays at various strain rates (Graham et al., 1983; Leroueil, 1988; Leroueil et al., 1985; Tanaka et al., 2006; Vaid et al., 1979) to investigate the effects of strain rate on the $e - \log \sigma'_v$ curve of clays. All the tests showed that the $e - \log \sigma'_v$ curves corresponding to different strain rates were very close to parallel, and that at a given strain $\sigma'_v$ increased with strain rate. All these findings indicate that the isotach model can be used to describe the relationship of stress–strain with strain rate, and model the strain-rate effect on the $e - \log \sigma'_v$ plot.

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; Znidaric et al., 1986), but there is no generally accepted conclusion regarding the effect of strain rate on $c_v$. 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 either.

Ariake clay is a very soft clayey soil deposited around the Ariake Sea in Kyushu, Japan. 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 the strain-rate effect on the consolidation behaviour of Ariake clay. In this paper the results of CRS tests, stepwise CRS tests (strain rate changed once) and IL tests for undisturbed soil samples from three boreholes in the Saga Plain, Kyushu, Japan, are presented and compared. The effects of strain rate on the consolidation yield stress $p_y$, compression index $c_v$, and the relationship between the degree of the strain-rate effect and the clay content and $I_p$ of the samples, are systematically investigated. The $p_y$ and $c_v$ values obtained from IL and CRS tests are compared.

2. EXPERIMENTAL PROGRAMME

2.1. Soil samples

Undisturbed soil samples were obtained from three boreholes (BH), as indicated in Figure 1, using the Japanese standard thin-wall sampler. In all, 20 sample tubes (1 m long, with depth varying from about 1·0 m to 16·0 m from the ground surface) were obtained. 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. In general, the natural water contents $w_h$ of the samples were more than 100%, and slightly larger than their corresponding liquid limits $w_L$. The main clay mineral of Ariake clay is smectite (Ohtsubo et al., 1995). The clay content ($<5 \mu$m) is in the range 40–70%, and $I_p$ is about 50. Figure 2 shows the soil profile and some of the index properties of the soil samples from BH-1. In the figure, C stands for clay, M for silt, S for sand, G for gravel, and $w_p$ for plastic limit.

2.2. Test equipment and method

The set-up of the CRS test is illustrated in Figure 3. The soil sample has a diameter of 60 mm and a nominal height of 20 mm. 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. 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. For the stepwise CRS tests the strain rate was changed once, when the vertical strain reached 10%. 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). For the CRS test, assuming that the distribution of excess pore pressure within a sample is parabolic (JSA, 2000b), the average effective vertical stress $\sigma'_v$ in a sample is calculated from

$$\sigma'_v = \sigma_v - \frac{1}{2} u_b$$

where $\sigma_v$ is the total vertical stress, and $u_b$ is the excess pore pressure at the bottom of the sample.

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.

2.3. Cases tested
A total of 114 CRS tests (of which 34 were stepwise CRS tests) and 15 IL tests were conducted for the undisturbed samples, as listed in Table 1. The strain rates used for the CRS tests were 0.02, 0.05, 0.1 and 0.2%/min.

3. RESULTS AND DISCUSSION
3.1. Strain-rate effect on $p_c$
Figures 4 and 5 are two 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, Figure 4 shows 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 Figure 5 does 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 et al., 2002). At present, we consider that the isotach model is applicable to natural Ariake clay.

To quantify the strain-rate effect on $p_c$, a ratio of $p_c$ (RPC) is defined as

$$\text{RPC} = \frac{p_{ij}}{p_{0.02/\text{min}}}$$

Table 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, 5, 6, 7, 9, 10, 11, 12, 13, 14, 16</td>
<td>CRS, Stepwise CRS, IL</td>
<td>0.02, 0.05, 0.1, 0.2</td>
</tr>
<tr>
<td>BH-2</td>
<td>3, 6, 14*</td>
<td>CRS, Stepwise CRS, IL</td>
<td>0.02, 0.05, 0.1, 0.2</td>
</tr>
<tr>
<td>BH-3</td>
<td>4, 6, 8, 10</td>
<td>CRS, Stepwise CRS, IL</td>
<td>0.02, 0.05, 0.1, 0.2</td>
</tr>
</tbody>
</table>

* No IL test.
† No stepwise CRS or IL test.
where \( \sigma_c \) is the consolidation yield stress for a given strain rate \( \varepsilon \) (0.05%/min, 0.1%/min or 0.2%/min), and \( \sigma_{c0} \) is the consolidation yield stress under a strain rate of 0.02%/min. The value of \( \sigma_c \) is obtained by Casagrande’s method.

Figure 6 shows RPC plotted against the strain rate obtained from CRS tests for all the Ariake clay samples tested 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 \( \sigma_c \) to increase with the strain rate. The linear regression line gives an increase of \( \sigma_c \) by about 16% with a tenfold increase in the strain rate.

One of the reasons for the scatter of the data is that different samples had to be used for different strain rates. Stepwise CRS tests were conducted, which used the same sample for two different strain rates. Figure 7 shows typical results of stepwise CRS tests. From the figure it can be seen that, when the strain rate was increased or decreased abruptly, the effective vertical stress showed an obvious increase or decrease. 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

\[
SRE = \frac{\sigma_{0.2/\text{min}}}{\sigma_{0.02/\text{min}}} - 15\%
\]

where \( \sigma_{0.2/\text{min}} \) and \( \sigma_{0.02/\text{min}} \) are the effective vertical stresses corresponding to strain rates of 0.2%/min and 0.02%/min, respectively.
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 8 shows the SRE from stepwise CRS tests and normal CRS tests for the soil samples from BH-1. 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), and the test results from BH-2 and BH-3 showed a similar tendency. 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 SRE value. 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 Figure 8, an average SRE value of about 1.15 can be obtained (i.e. about a 15% increase in effective vertical stress with a tenfold increase in the strain rate), and the test results from BH-2 and BH-3 also gave an average SRE value of about 1.15, which is smaller but very close to the increased rate for \( p_c \) of about 16% given in Figure 6.

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.

Conceptually, the SRE is the result of the natural viscosity of a clayey soil, and there may therefore be some kind of correlation between SRE and clay content or \( I_p \), or both.

Figure 9 is the SRE–\( I_p \) plot and Figure 10 is the SRE–clay content plot using stepwise CRS test results for soil samples from BH-1. It can be seen that there is no clear correlation between SRE and \( I_p \) or clay content.

Graham et al. (1983) concluded that the influence of strain rate on the stress–compressibility curves of the consolidation test is independent of \( I_p \). Tanaka et al. (2000) also found that no correlation exists between the strain-rate effect and \( I_p \) or clay content.

In engineering practice, IL tests are more widely used than CRS tests, and there is a question as to the strain rate at which the \( p_c \) values from CRS tests will be comparable with those from IL tests.

IL tests were also conducted for most of the samples. The values of \( p_c \) from IL and CRS tests are compared in Figures 11, 12 and 13 for BH-1, BH-2 and BH-3 respectively. Iribe (2006) also conducted both IL and CRS tests for undisturbed Ariake
clay samples from the Saga plain and the test results are included in Figure 12. 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. 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 Figure 14. 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 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).

3.2. Strain-rate effect on compression index ($C_c$)

The compression index $C_c$ and recompression index $C_r$ from CRS tests corresponding to different strain for soil samples from BH-1 at 5–5.9 m depth are depicted in Figure 15. The value of $C_c$ or $C_r$ is calculated from

$$C_c(C_r) = \frac{\epsilon_t - \epsilon_{t+\Delta t}}{\log(\sigma_{c(t+\Delta t)} - \sigma_{c(t)})}$$

The value of $C_c$ or $C_r$ is calculated from

![Figure 14](image)

![Figure 15](image)

clay samples from the Saga plain and the test results are included in Figure 12. 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. Although the
where \( e_t \) and \( e_{t+\Delta t} \) are the void ratios of the sample at time \( t \) and \( t + \Delta t \), and \( \sigma_{c,t} \) and \( \sigma_{c,t+\Delta t} \) are 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 \( C_t \) (\( \sigma_{c,t} < \sigma_{c,t+\Delta t} \)) and \( C_t \) show a clear trend of strain-rate independence. This result supports the isotach model, which has been used to interpret the stepwise CRS test results in this study.

If the average effective stress \( \bar{\sigma}_v \) within a time interval \( \Delta t \) is defined as

\[
\bar{\sigma}_v = \sqrt{\sigma_{c,t} \sigma_{c,t+\Delta t}}
\]

then, if the values of \( C_t \) and \( C_t \) are plotted against \( \bar{\sigma}_v \), it can be seen that there is a tendency for \( C_t \) to increase with strain rate (Figure 16).

In the IL test, the stress is applied in a stepwise fashion. In most cases the value of \( \sigma_c \), 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 \( C_t \) from the IL and CRS tests occurs around the \( \sigma_v \) value, as shown in Figure 17 for soil samples from BH-1 at 5–5.9 m depth.

The results given in Figures 15, 16 and 17 indicate that for microstructured Ariake clay, in the virgin compression range, the \( e \)–log \( \sigma_c \) 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 non-linear \( e \)–log \( \sigma_c \) relationships, in a plot of log(\( e + c_t \)) against log \( \sigma_c \), where \( c_t \) is a constant, they are very close to linear, and a model is proposed to consider the non-linear \( e \)–log \( \sigma_c \) behaviour in engineering design and analysis.

3.3. Strain-rate effect on coefficient of consolidation \( c_v \)

Figure 18 shows the variation of \( c_v \) with average effective vertical stress \( \sigma_c \) under different strain rates for soil samples from BH-1 at 5–5.9 m depth. For the CRS test the value of \( c_v \) is calculated from (JSA, 2000b)

\[
c_v = \frac{\Delta \sigma_v \Pi}{2 \Delta \bar{\sigma}_v \Delta t}
\]

where \( \Delta \sigma_v \) is the total vertical stress increment in a time interval \( \Delta t \). \( \Pi \) and \( \bar{\sigma}_v \) are the average sample thickness and excess pore water pressure at the bottom of the sample, which can be calculated as

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

\[
\bar{\sigma}_v = \frac{u_{h,t} + u_{h,t+\Delta t}}{2}
\]

where \( H_t \) and \( H_{t+\Delta t} \) are the thickness of the sample at time \( t \) and \( t + \Delta t \), and \( u_{h,t} \) and \( u_{h,t+\Delta t} \) are the excess pore water
pressure at the bottom of the sample at times \( t \) and \( t + \Delta t \) respectively. The values of \( c_v \) from the IL test are determined by the \( \sqrt{t} \) method (where \( t \) is time), and are also shown in Figure 18. It can be seen that the values of \( c_v \) obtained from CRS tests with a strain rate of 0–0.2%/min are comparable with those obtained from the IL test. For the results from the CRS tests, 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. The values of \( c_v \) under \( \sigma^c_v = 100 \) kPa (in the virgin compression range) were chosen for a detailed investigation of the strain-rate effect on \( c_v \), and the results are given in Figure 19 (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 0–0.2%/min. From Figure 19, 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 33% increase for a tenfold increase in the strain rate.

c_v \text{ is a function of coefficient of the volume compressibility } m_v \text{ and hydraulic conductivity } k, \text{ and can be expressed as}

\[
c_v = \frac{k}{m_v \gamma_w}
\]

where \( \gamma_w \) is the unit weight of water. It is interesting to know the degree of the strain-rate effect on \( m_v \) and \( k \).

### 3.4. Strain-rate effect on coefficient of volume compressibility \( m_v \)

Figure 20 shows the variation of \( m_v \) with \( \sigma^c_v \) under different strain rates for soil samples from BH-1 at 5–5.9 m depth. The value of \( m_v \) is calculated from

\[
m_v = \frac{\Delta H}{H \Delta \sigma^c}
\]

where \( \Delta H \) is the change of sample thickness and \( \Delta \sigma^c \) is the increment of effective vertical stress in a time interval \( \Delta t \). Although the data are scattered before \( \sigma^c \) increases, after this the values of \( m_v \) are almost the same for different strain rates on a logarithmic scale. \( m_v \) can be calculated from \( C_v, e \) and \( \sigma^c \) as

\[
m_v = 0.434 C_v (1 + e) \sigma^c_v
\]

As shown in Figure 21, even though the \( e - \log \sigma^c \) curves for different strain rates are parallel, for a given \( \sigma^c \) value the slope of the curve of \( C_v \) corresponding to different strain rates is different, owing to the non-linearity of the curve: for example, \( C_v^2 > C_v^1 \) (see also Figure 16 for the strain-rate effect on \( C_v \) for a given \( \sigma^c \)). The corresponding value of \( e \) is also different: for example, \( e_2 > e_1 \). In Equation 11, if \( \sigma^c \) is the same, both \( e \) and \( C_v \) increase with an increase in strain rate. As a result, \( m_v \) does not change much with the strain rate.

### 3.5. Strain-rate effect on hydraulic conductivity \( k \)

Figure 22 shows the variation of \( k \) with \( \sigma^c \) under different strain rates for soil samples from BH-1 at 5–5.9 m depth. The value of \( k \) is directly calculated from the test results as
Again, the data are scattered in the overconsolidated range, and in the virgin consolidation range k obviously increases with an increase of strain rate. As for c_v, the values of k at \( \sigma'_v = 100 \text{kPa} \) were chosen to study the strain-rate effect on k, and the results are shown in Figure 23 (12 extremely scattered points are excluded, as for c_v in Figure 19). 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 to increase with an increase in strain rate. Linear regression gives about a 29% increase of k with a tenfold increase in the strain rate, which means that the increase of c_v with increasing strain rate (Figures 18 and 19) is due mainly to the increase in 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 \( i_i \) for water flow in Na-clay, and when \( i < i_i \) there would be no flow (Figure 24). 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 increasing strain rate. Figure 25 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

\[
i_{av} = \frac{u_b}{Y_w H}
\]

where \( H \) is the thickness of the sample. Figure 26 shows the variation of \( i_{av} \) with strain rate at an effective vertical stress \( \sigma'_v = 100 \text{kPa} \). It can be seen that \( i_{av} \) increases with increasing strain rate. For 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 Figure 24, qualitatively it can be said that k apparently increases with an increase in i, as shown by the dashed lines in the figure (\( k_2 > k_1 \)).

3.6. Practical implications of the findings
The findings of strain-rate effect on the consolidation behaviour of clay from this study can help to improve our...
The strain-rate effect on \( p \), had been understood for a long time (Graham et al., 1983; Leroueil, 1988; Leroueil et al., 1985; Tanaka et al., 2006; Vaid et al., 1979). The systematic research from the current study indicates that for Ariake clay deposits the value of \( p \), increases by about 15–16% for a tenfold increase in strain rate. This value can be used in selecting a suitable value of \( p \), for settlement calculations or numerical analysis according to an estimated field strain rate. The finding that, for Ariake clay, the value of \( p \), from the IL test is comparable to that from the CRS test for a strain rate of 0-02%/min implies that the CRS test results for a strain rate of 0-02%/min can be used in the same way as those of the IL test, for which engineers have more experience and confidence.

The fact that \( k \) and therefore \( c_v \), increase with increasing strain rate is a new finding from this study. It implies that the rate of consolidation is a function of strain rate. Although more research may be needed, there is a possibility that a consolidation theory considering the strain-rate effect may be developed. In civil engineering design there is a tendency to adopt 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.

### 4. CONCLUSIONS

Based on the test and analysis results, the following conclusions can be drawn.

(a) Strain-rate effect on consolidation yield stress \( p \). The \( p \) value of Ariake clays increased by about 15–16% with a tenfold increase in the strain rate. There is no correlation between the strain-rate effect and the clay content or plasticity index \( (I_p) \) of the samples. Comparing the \( p \) values from IL and CRS tests, it is suggested that the \( p \) values from CRS tests with a strain rate of 0-02%/min are comparable to those from IL tests.

(b) Strain-rate effect on compression index \( C_v \). For a given strain level, the strain rate does not influence \( C_v \) values in the range of strain rates tested, and the isotach model is applicable to Ariake clay. 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 \( C_v \) value from the CRS and IL tests occurred around the value of \( p \).

(c) Strain-rate effect on coefficient of consolidation \( c_v \). Although the strain-rate effect on \( c_v \) had not been a subject of investigation in the literature, the test data from this study show that, under a given effective vertical stress, \( c_v \) increased with the increase of strain rate resulting mainly from the increase of hydraulic conductivity \( k \), and the coefficient of volume compressibility \( n_v \) was almost the same for different strain rates. Although the data are scattered, linear regression results in about a 33% increase of \( c_v \) for a tenfold increase in the strain rate. Comparing the \( c_v \) values from the IL and CRS tests suggests that the \( c_v \) values from CRS tests for a strain rate of 0-02%/min are comparable to those of IL tests.

### REFERENCES


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