Anisotropic Consolidation Behavior of Ariake Clay from Three Different CRS Tests

**Introduction**

For civil engineering projects related to soft clayey deposits, engineers inevitably have to consider the consolidation behavior of the deposit. According to the loading and drainage conditions, consolidation problems can be summarized into the following three types: (1) vertical loading and vertical drainage, which are normally considered in the design of an embankment (Fig. 1(a)) or a foundation on a clayey deposit; (2) vertical loading and horizontal drainage, normally considered important in the design of schemes for the improvement of clayey deposits utilizing prefabricated vertical drains (PVDs) and surcharge load and/or vacuum pressure (Fig. 1(b)); and (3) horizontal loading and horizontal drainage, such as in the design of a laterally loaded pile and the consolidation behavior of soil under the toe of the embankment (Fig. 1(b)). For sedimentary clays, consolidation properties in the horizontal direction are usually different from those in the vertical direction.

**REFERENCES:**


**ABSTRACT:** Three types of constant rate of strain (CRS) consolidation tests were conducted on samples of undisturbed Ariake clay, using a newly developed consolidometer, to investigate the anisotropic consolidation behavior of the clay. CRS tests conducted using vertically cut specimens (with respect to both the in situ condition) were designated as CRS-V-V tests. Specimens cut vertically but with radial drainage were designated CRS-V-R, whereas those cut horizontally and tested with vertical drainage were designated CRS-H-V. The test results show that the ratio of the consolidation yield stress of a horizontally cut specimen ($p_{ych}$) to that of a vertically cut specimen ($p_{ych}$) is in a range from 0.5 to 1.0. Both $p_{ych}$ and $p_{ych}$ increased by about 15% with a tenfold increase in strain rate, but there was no clear difference in the degree of strain-rate dependency for $p_{ych}$ and $p_{ych}$. Values of the coefficient of consolidation obtained from CRS-H-V ($c_{h}$) and CRS-V-R ($c_{r}$) tests are larger than those measured in CRS-V-V ($c_{v}$) tests, and it has been identified that these differences arise mainly from the anisotropy of hydraulic conductivity ($k$). The ratio of $k_h$ in the horizontal direction ($k_h$) measured in a CRS-V-R test to that in the vertical direction ($k_v$) from a CRS-V-V test is about 1.65, and the ratio of $c_h/c_v$ is about 1.54. The value of $k_h$ from a CRS-H-V test is generally smaller than that from a CRS-V-R test.

**KEYWORDS:** anisotropic consolidation behavior, constant rate of strain (CRS) consolidation, consolidation yield stress, strain-rate effect, coefficient of consolidation, hydraulic conductivity

behavior is therefore useful for designing civil engineering projects related to soft clayey deposits.

Regarding the horizontal consolidation properties, there are published incremental loading (IL) consolidation test results for soil samples cut horizontally with respect to in situ conditions (e.g., Park 1994; Bo et al. 2003) and Rowe cell tests conducted with radial drainage (Rowe and Barden 1966; Wong 2005). Comparing the test results of Ariake clay samples cut vertically and horizontally with respect to the in situ condition, Park (1994) reported that the ratio of hydraulic conductivity in the horizontal direction ($k_h$) to that of in the vertical direction ($k_v$) was about 1.5. Recently, constant rate of strain (CRS) consolidation tests with radial drainage have been used to obtain the horizontal consolidation properties, with radial drainage directed toward the periphery of the specimen (Yune and Chung 2005) or toward the center of the specimen (Seah and Juiraarongrit 2003; Moriwaki and Satoh 2009). Yune and Chung (2005) reported a value of $k_h/k_v$ of 1.7 for undisturbed Korea clay and 1.3 for reconstituted Korea clay from CRS test with both vertical and radial drainage. Seah and Juiraarongrit (2003) conducted CRS tests on Bangkok clay with vertical and radial drainage, and recorded a ratio of $k_h/k_v$ of 1.45. Further, Leroueil et al. (1990) investigated the anisotropy of hydraulic conductivity ($k$) of five marine clays by directly measuring the values of $k_h$ and $k_v$ and reported $k_h/k_v$ ratios of about 1.10 to 1.55. CRS test has several advantages in comparison with the traditional incremental loading consolidation test: for example, CRS tests can be used to study the strain-rate effect, 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 $\sigma'$.  

Manuscript received March 2, 2011; accepted for publication June 12, 2012; published online September 2012.

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However, in the case of peripheral drainage, minimizing the effect of the friction between the specimen and the consolidation ring is a challenge.

In this study, a CRS consolidometer was developed that can conduct CRS tests with both vertical and radial drainage. In the case of radial drainage, the drainage boundary is at the center of the specimen the same as that by Seah and Juimarongrit (2003) as well as by Moriwaki and Satoh (2009). Three types of CRS tests were conducted using this consolidometer to investigate the anisotropic consolidation behavior of undisturbed Ariake clay from Saga, Japan. The three types of test were: (1) those on vertically cut specimens (with respect to the in situ condition) with vertical (or end) drainage (with respect to the test conditions) (CRS-V-V), which represent vertical loading and the vertical drainage conditions, (2) those on vertically cut specimens with radial drainage (CRS-V-R), which represent vertical loading and horizontal drainage conditions, and (3) those on horizontally cut specimens with vertical drainage (CRS-H-V), which represent horizontal loading and horizontal drainage conditions. The test results are compared in terms of the measured 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 $k_h$). The subscripts “v” and “h” are adopted here to indicate the values in the vertical and the horizontal directions with respect to the direction of deposition, respectively. The effects of strain rate on $p_{cv}$ and $p_{ch}$ were also investigated. The test procedures and the test results are presented, and some discussion is provided on a suitable method for obtaining the horizontal consolidation properties of a clayey deposit.

**Test Equipment and Method**

CRS-V-V and CRS-H-V tests were conducted using a conventional CRS device (Jia et al. 2010). The device was modified to conduct CRS-V-R tests and the setup of the modified device is illustrated in Fig. 2. The modification components (1–6) are explained as follows:

1. Central drainage porous stone: it is a cylindrical ring porous stone with an outer diameter of 8 mm and wall thickness of 3 mm, and 20 mm height. The porous stone is inserted in the middle of the specimen to provide drainage during consolidation.

2. Stainless steel loading cap: a hole 9 mm in diameter and 10 mm deep at the center of the loading cap was made to allow the central drainage porous stone to slide freely during consolidation. There are also five small holes through the top surface of the loading cap leading from the central hole to provide drainage paths.

3. A guide ring: a guide ring is set on top of the specimen. 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 to prevent leakage in the vertical direction. The friction between the “o”-ring and the loading cap was calibrated.

4. Consolidation ring: a consolidation ring was fabricated with an excess pore water pressure gauge built in at the mid-height of the ring wall.

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 to improve the contact condition between the guide ring and the consolidation ring. A rubber ring is placed between the acrylic fiber ring and the guide ring to ensure uniform pressure is exerted on the guide ring.

The specimen preparation procedure for CRS-V-V and CRS-H-V tests is exactly the same as that described by Jia et al. (2010). In case of CRS-V-R tests, annular soil specimens with an o.d. of 60 mm, inside diameter of 8 mm, and a nominal height of 20 mm were used. Each specimen was prepared in two steps. The first step is the same as that adopted for preparing specimens for CRS-V-V and CRS-H-V tests, and the second step involved making the central hole required for inserting the central drainage porous stone. The diameter of the central hole is 8 mm.

CRS-V-V and CRS-H-V tests were carried out according to ASTM D4186-06 (ASTM 2006). For the CRS-V-R tests, the excess pore water pressure at the outside perimeter of the specimen ($u_e$) was measured. For all tests, a backpressure of 200 kPa was applied throughout the tests to increase the degree of saturation.

### Experimental Program

#### Soil Samples

Undisturbed soil samples were obtained from three boreholes (BH) in Saga Plain, Kyushu, Japan. The Japanese standard thin-wall sampler was used to obtain these undisturbed samples. In all, 18 sample tubes each about 1.0 m long were obtained from depths varying from 2.0 m to 21.0 m from the ground surface. After the sample tubes had been transported to the laboratory, soil samples were extruded from the thin-wall tubes, sealed with wax, and stored under water.

Figure 3 shows the soil profile and some of the index properties of the soil samples from BH-1. In the figure, $w_p$ denotes the plastic limit, $w_L$ the liquid limit, and $w_n$ the natural water content. In the column labelled “grain size distribution,” C stands for clay, M for silt, S for sand, and G for gravel.

### Cases Tested

All cases tested are listed in Table 1. The specimen cutting direction, loading and drainage conditions are illustrated in Fig. 4. From the CRS-V-V tests values of $p_{cv}$, $c_v$, $m_v$, and $k_v$ were obtained, from the CRS-H-V tests values of $p_{ch}$, $c_{hh}$, $m_h$, and $k_h$ were obtained, and from the CRS-V-R tests values of $p_{cv}$, $c_{hv}$, $m_v$, and $k_h$ were obtained. The symbol $c_{hh}$ indicates the coefficient of consolidation for horizontal drainage and horizontal loading, and $c_{hv}$ is used to indicate the coefficient of consolidation for horizontal drainage and vertical loading.

### Method for Interpreting the Test Results

#### CRS Test with the Vertical Drainage

For the CRS tests with the vertical drainage, assuming the distribution of excess pore water pressure within a specimen is parabolic with depth (ASTM 2006), the average vertical effective stress $\sigma'_v$ in a specimen is calculated as:

$$\sigma'_v = \sigma_v - \frac{2}{3} u_h$$  \hspace{1cm} (1)
where $\sigma_v$ is the total vertical stress and $u_b$ is the excess pore water pressure at the bottom of the specimen. The value of $c_v$ can be calculated as follows:

$$c_v = \frac{\Delta \sigma_v \cdot \bar{H}^2}{2 \cdot \bar{u}_0 \cdot \Delta \sigma^v}$$

(2)

where $\Delta \sigma_v$ is the total vertical stress increment in a time interval $\Delta t$; $\bar{H}$ is the average thickness of the specimen between $t$ and $t + \Delta t$ and $\bar{u}_0$ is the average excess pore water pressure at the bottom of the specimen between $t$ and $t + \Delta t$. The coefficient of volume compressibility ($m_v$) is calculated by the following equation:

$$m_v = \frac{\Delta H}{H \cdot \Delta \sigma^v}$$

(3)

where $\Delta H$ is the change of the thickness of the specimen and $\Delta \sigma^v$ is the increment of vertical effective stress in a time interval $\Delta t$. The hydraulic conductivity ($k$) is directly calculated from the test results as follows:

$$k = \frac{\gamma_w \cdot \Delta H \cdot H \cdot \Delta \sigma^v}{2 \cdot \bar{u}_0 \cdot \Delta \sigma^v}$$

(4)

**CRS Test with the Radial Drainage**

Consolidation with radial flow is governed by the following differential equation:

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

(5)

where $r$ is the radial distance from the center of the specimen and $u_r$ is excess pore water pressure at $r$. With central radial drainage and assuming equal vertical strain throughout the annular soil specimen, the solution for ideal conditions (no smear and no well resistance (Hansbo 1981)), which is applicable to the test conditions in this study, is as follows (Barron 1948):

$$u_r = \frac{\bar{u}_0 \cdot e^{-\lambda}}{r_w^2 \cdot F(n)} \left[ r_w^2 \cdot \ln(r/r_w) - (r^2 - r_w^2)/2 \right]$$

(6a)

Let $B(t) = \frac{\gamma_w}{\gamma_w \cdot \bar{H} \cdot \bar{m}_v}$, then

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

(6b)

where $\bar{u}_0$ is the initial excess pore water pressure (uniform); $r_w$ is the radius of the specimen; $r_c$ is the radius of the central drain; $\lambda = 8 \cdot T_h / F(n)$; $T_h$ is the time factor for radial drainage, which is defined as $T_h = c_h / (4 \cdot r_c^2)$; and $F(n) = \frac{n^2}{\pi} \cdot \frac{2}{\ln(n) - 3 \cdot \frac{n^2}{4} - 1} (n = r_c / r_w)$

Juinarongrit (1996) derived the equations based on Barron’s theory for determining values of $c_h$ and $k_h$ from the results of CRS tests with radial drainage. By applying the boundary condition $u_r = u_e$ at $r = r_c$, to Eq (6b), then

$$B(t) = \frac{u_e}{r_c^2 \cdot \ln(n) - (r^2 - r_w^2)/2}$$

(7)

Differentiating Eq (6b) with respect to $r$ gives:

$$\frac{\partial u_r}{\partial r} = B(t) \left( r_c^2 - r^2 \right) / r$$

(8)

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

$$q = k_h \frac{\partial u_r}{\partial r} \cdot 2 \cdot \pi \cdot r \cdot H$$

(9)

Substituting Eq (8) into Eq (9) gives:

$$q = \frac{2 \cdot \pi \cdot k_h \cdot H}{\gamma_w} B(t) \left( r_c^2 - r^2 \right)$$

(10)

At $r = r_w$, the rate of water flow can be written as:

$$q = \frac{2 \cdot \pi \cdot k_h \cdot H \left( r_c^2 - r_w^2 \right) - \gamma_w}{\gamma_w} B(t)$$

(11)

Assuming the specimen is fully saturated and the soil skeleton material is virtually incompressible, the rate of water flow can also be expressed in terms of the vertical displacement rate of the specimen $v_p$ as:

$$q = v_p \cdot \frac{\gamma_w \cdot \left( r_c^2 - r_w^2 \right)}{u_w \cdot H}$$

(12)

Combining Eq (11) and Eq (12), and if $r_c = 30 \text{ mm}$ and $r_w = 4 \text{ mm}$ (conditions adopted in this study), then

$$k_h = \frac{v_p \cdot \gamma_w \cdot \left( r_c^2 - r_w^2 \right)}{u_w \cdot H \cdot m_v} = 0.762 \cdot r_c^2 \cdot u_p \cdot \gamma_w$$

(13)

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

$$c_h = \frac{k_h}{\gamma_w} \frac{\gamma_w \cdot \left( r_c^2 - r_w^2 \right)}{u_w \cdot H \cdot m_v}$$

(14)

The average excess pore pressure in a specimen is:

$$u_e = \int_{r_w}^{r_c} \frac{2 \cdot \pi \cdot r \cdot u \cdot dr}{A} = \left[ \frac{r_c^4 \ln(n) - r_w^4 \ln(n) - (r_c^2 - r_w^2)^2}{r_c^2 - r_w^2} \right] u_e = 0.857 \cdot u_c$$

(15)

where $A = \pi \left( r_c^2 - r_w^2 \right)$. So the average vertical effective stress can be expressed as:

$$\sigma_v' = \sigma_v - u_e = \sigma_v - 0.857 \cdot u_c$$

(16)

**Test Results and Discussion**

**Consolidation Yield Stress ($p_{cv}$ and $p_{ch}$)**

In the following presentation, $p_{cv}$ represents the consolidation yield stress of a vertically cut specimen (CRS-V-V and CRS-V-R tests), whereas $p_{ch}$ represents the consolidation yield stress of a horizontally cut specimen (CRS-H-V test). Figure 5 shows typical stress–strain relationships for the three types of CRS test, for soil samples from BH-1 at 15.5–16.3 m depth. It can be seen that the value of $p_{ch}$ obtained from the CRS-H-V test is smaller than the values of $p_{cv}$ obtained from the CRS-V-V and CRS-V-R tests.

For a normally consolidated natural soil sample, if the vertical effective stress is $\sigma_v' = p_{cv}$, the horizontal effective stress will be $\sigma_h' = K_0 p_{cv}$ ($K_0$ is the coefficient of earth pressure at rest). Figure 6
shows the ratio of $p_{ch}$ to $p_{cv}$ for all the soil samples tested. The majority of the data are in the range from 0.5 to 1.0, with an average value of about 0.7. For undisturbed Ariake clay, the angle of internal friction ($\phi'$) is about 30° (Hino et al. 2010). Considering that at the yield stress condition, over consolidation ratio (OCR) is 1.0 and using $K_0 = 1 - \sin \phi'$ (Mayne and Kulhawy 1982), $K_0$ value of 0.5 can be estimated. The test data show that generally $p_{ch} > K_0 p_{cv}$.

The effective stress path experienced by the soil samples in CRS-H-V test is different from that of CRS-V-V and CRS-V-R tests, and $p_{ch} > K_0 p_{cv}$ may be a result of the different stress paths of the soil sample experienced and anisotropic yielding behavior of the samples. Wood (1990) reported that most natural clays exhibit anisotropic yielding behavior.

Figure 7 shows the ratio of values of $p_{cv}$ obtained from CRS-V-R tests to those from CRS-V-V tests, for all soil samples tested. The linear regression shows that on average, values of $p_{cv}$ obtained from CRS-V-R tests are about 1.04 times those from CRS-V-V tests. In the figure, $R^2$ means the square of correlation coefficient of the linear regression. Although numerically there is some difference in the values of $p_{cv}$ deduced from the two types of test, for most practical purposes they are almost identical.

A possible reason for the slightly larger $p_{cv}$ values from the CRS-V-R tests may be the effect of the friction between the soil specimen and the central porous stone used for drainage.

To quantify the effect of strain rate on $p_{cv}$, strain-rate parameter $\rho_{0.02}$ (Sheahan et al. 1996) is defined as:

$$\rho_{0.02} = \left[ \frac{\Delta p_{cv}}{p_{cv}}/\log(0.2/0.02) \right] \cdot 100 \quad (18)$$

where $p_{cv}$ = value of $p_{cv}$ at the reference strain rate, 0.02 %/min; and $\Delta p_{cv} = \text{increase in } p_{cv} \text{ for the strain-rate increase from 0.02 %/min to 0.2 %/min}$. Figure 8 shows the values of $\rho_{0.02}$ obtained from three types of CRS test. Most of the $\rho_{0.02}$ values are larger than 0, with an average value of about 0.15, which implies that both $p_{cv}$ and $p_{ch}$ increase with strain rate. There is no obvious difference in the degree of strain-rate dependency for $p_{cv}$ and $p_{ch}$. The data are scattered because different soil samples were used for the different strain rates and there might be spatial variation of soil samples within a sample tube.

**Coefficient of Consolidation ($c_{cv}, c_{chh}, \text{and } c_{chv}$)**

The results for $c_{cv}, c_{chh}$, and $c_{chv}$ obtained from the three types of CRS test for soil samples from BH-1 at 15.5–16.3 m depth are compared in Fig. 9. In the virgin consolidation range values of $c_{chh}$ and $c_{chv}$ obtained from CRS-H-V and CRS-V-R tests, respectively,
are obviously larger than the corresponding values of \(c_v\) obtained from CRS-V-V tests. The coefficient of consolidation is a function of the hydraulic conductivity \((k)\) and the coefficient of volume compressibility \((m)\). Even if the values of \(k_h\) obtained from CRS-H-V and CRS-V-R tests were the same, the differences in the values of \(m\) will result in different values of \(c_h\) and \(c_v\). When \(\sigma'_v\) is larger than about 200 kPa, there is no obvious difference in the values of \(c_h\) and \(c_v\).

The values of \(c_v\) corresponding to \(\sigma'_v = 300\) kPa (in the virgin compression range for all the soil samples tested) were chosen to investigate the relation between \(c_h\) and \(c_v\), and the results are shown in Fig. 10(a) (excluding three extremely scattered points). It can be seen that \(c_h\) is larger than \(c_v\) and there is a trend of correlation between \(c_h\) and \(c_v\). Linear regression was conducted to provide a representative trend and resulted in a ratio of \(c_h/c_v\) of 1.38.

Figure 10(b) shows the relation between \(c_h\) and \(c_v\) (excluding four extremely scattered points). It can be seen that \(c_h\) is also generally larger than \(c_v\), and linear regression shows that the ratio of \(c_h/c_v\) is about 1.54. CRS-V-R tests resulted in larger values of \(c_h\) than the values of \(c_h\) obtained from CRS-H-V tests.

Regarding the effect of strain rate on the coefficient of consolidation, it is observed that the values of \(c_v\), \(c_h\), and \(c_v\) all increase with an increase in strain rate. However, because the data are more scattered than for \(p_c\), a definite rate of increase in the coefficient of consolidation cannot be reliably defined.

It is also of interest to investigate whether the differences in the values of \(c_v\), \(c_h\), and \(c_v\) arise because of differences in the values of \(m\) or \(k\). An investigation of this issue is presented in the following section.

**Coefficient of Volume Compressibility (\(m\) and \(m_h\))**

A comparison of values of \(m\) and \(m_h\) for soil samples from BH-1 at 15.5–16.3 m depth is presented in Fig. 11. In the overconsolidated range, \(m\) is obviously less than \(m_h\) However, in the virgin consolidation range the values of \(m\) and \(m_h\) are almost the same, especially for cases where \(\sigma'_v > 200\) kPa.

**Hydraulic Conductivity (\(k\) and \(k_h\))**

Figure 12 shows a comparison of values of \(k\) obtained for soil samples from BH-1 at 15.5–16.3 m depth. In the virgin consolidation range the \(k_h\) values obtained from CRS-H-V and CRS-V-R tests are obviously larger than the values of \(k\) obtained from CRS-V-V test.
The ratio of $k_h$ to $k_v$ was investigated at $\sigma'_v = 300$ kPa. All the test results (excluding three extremely scattered points) for values of $k_h$ obtained from CRS-H-V tests and the values of $k_v$ are shown together in Fig. 13(a). It can be observed that there is a trend of correlation between $k_h$ and $k_v$. Linear regression shows that the ratio of $k_h$ to $k_v$ is about 1.34. This value is very close to the number reported by Park (1994) of about 1.5 for the soil in the same region, but still comparable.

Figure 13(b) shows the ratio of $k_h$ obtained from CRS-V-R tests to values of $k_v$ for all the soil samples tested (excluding four extremely scattered points). Linear regression analysis shows that the ratio of $k_h$ to $k_v$ is about 1.65. This value is slightly larger than the ratio $c_{bh}/c_v$ of 1.54 shown in Fig. 10(a), which indicates that $k$ is the main factor influencing the anisotropy of the coefficient of consolidation. The value of 1.34 is smaller than the number reported by Park (1994) of about 1.5 for the soil in the same region, but still comparable.

Deformation patterns (directions). In a CRS-V-R test, the specimen deforms vertically, but in a CRS-H-V test it deforms horizontally, both directions being considered with respect to their in situ orientations. It is considered hypothetically that there might be more de-structuring effect in CRS-H-V test and thus this results in a lower $k_h$ value.

From the comparisons presented above it is understood that: (1) Ariake clay exhibits anisotropic consolidation behavior and this is mainly because of anisotropic hydraulic conductivity, i.e., $k_h > k_v$; (2) CRS-V-R tests result in larger values of $k_h$ than those obtained from CRS-H-V tests, and this may be because of the different compression directions for the specimen in CRS-H-V and CRS-V-R tests. A practical implication of (2) is that data obtained from a consolidation test using horizontally cut soil specimens with vertical drainage is likely to result in underestimation of the value of $c_{bh}$.

Excess Pore Water Pressure ($u_b$ and $u_e$)

Figure 14 shows the variation of measured excess pore water pressure in the three types of CRS test, for soil samples from BH-1 at 15.5–16.3 m depth. At a given strain, the values of $u_b$ obtained from CRS-H-V tests are lower than those from CRS-H-V tests because of the higher value of $k_h$. At a given strain, $u_e$ is generally smaller than the number reported by Park (1994) of about 1.5 for the soil in the same region, but still comparable.
Figure 15 shows 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 indicate that the ratio of \( u_e \) to \( u_b \) is 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. 13(b)), 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.

### Conclusions

Three types of constant rate of strain (CRS) consolidation test were conducted using undisturbed Ariake clay samples in a newly developed consolidometer to investigate the anisotropic consolidation behavior of the clay. Vertically cut soil specimens with the vertical (or end) drainage were designated as CRS-V-V samples, horizontally cut specimen with vertical drainage were designated CRS-H-V and vertically cut specimens with radial drainage were designated CRS-V-R test results at a vertical strain of 15%. There is no obvious difference in the degree of strain-rate dependency for all the CRS-V-V and CRS-V-R test results. The following conclusions can be drawn:

1. The ratio of the consolidation yield stress of horizontally cut specimens \( (p_{ch}) \) to those of vertically cut specimens \( (p_{cv}) \) is in the range of 0.5 to 1.0, with an average value of about 0.7.
2. Both \( p_{cv} \) and \( p_{ch} \) increased with strain rate. For a tenfold increase in strain rate, both \( p_{cv} \) and \( p_{ch} \) increased by about 15%. There is no obvious difference in the degree of strain-rate dependency for \( p_{cv} \) and \( p_{ch} \).
3. Ariake clay exhibits anisotropic consolidation behavior. The coefficient of consolidation in the horizontal direction \( (c_{ch}) \) from CRS-V-V tests and \( c_{ch} \) from CRS-V-R tests) is larger than that in the vertical direction \( (c_v) \). This anisotropic consolidation behavior arises mainly from the anisotropy of hydraulic conductivity \( (k) \). The ratio of \( k \) in the horizontal direction \( (k_h) \) from CRS-V-R tests to that in the vertical direction \( (k_v) \) measured in CRS-V-V tests is about 1.65 and the ratio of \( c_{ch}/c_v \) is about 1.54.

### Acknowledgments

The writers are grateful to Prof. J. P. Carter at The University of Newcastle, Australia, for his valuable comments and suggestions in preparing this paper.

### References


