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Strain-Rate Selection in the Constant-Rate-of-Strain Consolidation Test

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Strain-Rate Selection in the Constant-Rate-of-Strain Consolidation Test

by

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in cooperation with
DEPARTMENT OF TRANSPORTATION
Commonwealth of Kentucky

and

Federal Highway Administration
U. S. DEPARTMENT OF TRANSPORTATION

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June 1981
Abstract

Constant-rate-of-strain (CRS) consolidation tests were performed on remolded kaolinite specimens. The effect of strain rate on CRS test data is shown. A relation between soil parameters and strain rate was developed and used to formulate a strain-rate selection procedure. The final selection procedure is based on liquid limit and initial degree of saturation of the specimen and is presented in graphical form. The procedure is applicable to all types of soils.
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Introduction

The constant-rate-of-strain (CRS) consolidation test was introduced in 1959 (1); the theory for complete reduction of data was developed in 1971 (2). In 1976 (3), the CRS test was compared with the controlled-gradient and incremental-load consolidation tests. The 1976 report related the conclusion that the CRS test offers several advantages over other methods of consolidation testing. The CRS test requires the least time to complete, is the least difficult to perform, and does not require sophisticated equipment. A disadvantage is selecting a suitable strain rate which must be set prior to loading the soil specimen. Previous experimental research and theoretical developments (2, 4) have shown that the magnitude of pore pressure developed at the base of the specimen, $u_b$, must be maintained within certain limits to produce meaningful data. The value of $u_b$ may be raised or lowered by increasing or decreasing the strain rate. Specifically, the limits on $u_b$ have been based on the total vertical stress, $\sigma_v$, by limiting $u_b/\sigma_v$. Guidelines for selecting approximate strain-rates were given in the 1976 report (3) and were based on limited data then available. More specific guidelines are needed to fully develop a standard method of test and to implement the test on a routine basis. This report refines and extends the strain-rate selection guidelines.

Strain-rate selection is not an exact procedure. The usefulness of the data does not depend on a precise selection of strain rate. Engineering judgment and the guidelines presented herein are sufficient to select a strain rate inasmuch as a wide range of acceptable rates exist for a given soil. Engineering judgment must be based on a knowledge of the effects that different strain rates have on the parameters determined in the CRS test and used in settlement analyses – more specifically, the effect of strain rate on the coefficient of consolidation, $C_v$, apparent preconsolidation pressure, $P_c$, and compression ratio, CR.

Development of guidelines was based on a testing program using remolded kaolinite soil samples. A series of tests was performed to define the effect of strain rate on results for given soil conditions. The steps are outlined below:

1. The effect of different strain rates is discussed and the connection between these effects and $u_b/\sigma_v$ is established.
2. The soil parameters that primarily affect $u_b/\sigma_v$ for a given strain rate are defined and discussed. The primary parameters are $C_v$ and initial degree of saturation, $S$.
3. A relationship between strain rate and $u_b/\sigma_v$ is established using test results obtained from CRS tests performed on the remolded kaolinite samples.
4. The relationship obtained from the remolded kaolinite test series is extended to variable $C_v$ and checked against CRS test data for other soils.
5. An upperbound is selected for allowable $u_b/\sigma_v$, thereby eliminating $u_b/\sigma_v$ as a variable in the rate selection criteria. The variables in the rate selection criteria, at this point, are strain rate, $r$, and $C_v$.
6. A correlation between liquid limit and $C_v$ is used as a basis for replacing $C_v$ with liquid limit in the $r$-versus-$C_v$ relation.
7. Test data obtained from CRS tests on partially-saturated kaolinite specimens are used to extend the $r$-versus-liquid limit relationship and to develop final strain-rate selection guidelines based on liquid limit and initial degree of saturation.

Sample Preparation

A kaolinite clay, obtained from the Edgar Kaolin Co., Edgar, Florida, was selected for the testing program. The index properties of this soil are shown in Table 1.

Specimens were molded by two methods. The first method produced uniform samples, which were used to study strain-rate effects. A Vac-Aire extrusion machine produced solid cylindrical speci-
TABLE 1. ENGINEERING PROPERTIES OF KAOLINITE

<table>
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<th>Property</th>
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<tr>
<td>Atterberg Limits</td>
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</tr>
<tr>
<td>Liquid Limit (Percent)</td>
<td>59</td>
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<tr>
<td>Plastic Limit (Percent)</td>
<td>34</td>
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<tr>
<td>Gradation</td>
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<tr>
<td>Percent Silt</td>
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<tr>
<td>Percent Clay</td>
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<td>Moisture-Density Relation</td>
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<td>Optimum Moisture (Percent)</td>
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<td>Maximum Dry Density (kg/m³)</td>
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<tr>
<td>(pcf)</td>
<td>81.3</td>
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<td>Classification</td>
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<tr>
<td>Unified</td>
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<tr>
<td>AASHTO</td>
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<td>Kaolinite (Percent)</td>
<td>94</td>
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<td>Quartz (Percent)</td>
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<td>Talc (Percent)</td>
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TABLE 2. PROPERTIES OF VAC-AIRE EXTRUDED SPECIMENS

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<th>Property</th>
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<tr>
<td>Moisture Content (Percent)</td>
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<td>Degree of Saturation (Percent)</td>
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<tr>
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<tr>
<td>(pcf)</td>
<td>79.3</td>
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<tr>
<td>Void Ratio</td>
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TABLE 3. PROPERTIES OF MECHANICALLY COMPACTED SPECIMENS

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<tr>
<th>Moisture Content (%)</th>
<th>Degree of Saturation (%)</th>
<th>Void Ratio</th>
<th>Dry Density (kg/m³)</th>
<th>Dry Density (pcf)</th>
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<td>23</td>
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<td>1.16</td>
<td>1,240</td>
<td>77.4</td>
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<td>28</td>
<td>59</td>
<td>1.27</td>
<td>1,181</td>
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<tr>
<td>34</td>
<td>79</td>
<td>1.17</td>
<td>1,235</td>
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<tr>
<td>40</td>
<td>89</td>
<td>1.24</td>
<td>1,197</td>
<td>74.7</td>
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</table>

Testing

The first series of CRS tests were performed on uniform samples at 96 percent of saturation. All variables were constant except for strain rate. Five CRS tests were performed at five strain rates. Two incremental-load consolidation tests were also performed. The second series included tests on four partially-saturated specimens, shown in Table 3. These specimens were tested at approximately the same strain rate.

Some of the equations developed in this report were compared with data obtained previously (3). The tests were described in that report, and only pertinent data are presented here.

EQUIPMENT

A schematic of the CRS equipment is shown in Figure 1. The equipment was designed in-house and fabricated by the Machine Shop, University of Kentucky, College of Engineering. Several special features, Figure 2, are worthy of mention. The loading ram is guided into the chamber by a low-friction seal system described by Chan (5). The pressure transducer to monitor pore-water pressure at the specimen base is mounted directly in the base plate. This reduces the flexibility of the pressure measuring system and reduces the time lag in pore-pressure measurement. The steel ring, to laterally confine the specimen, is sealed to
Figure 1. Schematic of CRS Test Equipment.

Figure 2. CRS Test Equipment.

The CRS consolidometer was loaded in a Karol-Warner triaxial press. The adjustment of strain rate was by a gear reducer and variable speed electric motor. They are shown in Figure 3.

In addition to the CRS tests, two incremental-load consolidation tests were also performed. The Anteus back-pressure consolidometer shown in Figure 4 was used in these tests.

PROCEDURE

Each specimen was trimmed to 2.5 inches (64 mm) in diameter by 1.0 inch (25 mm) in height using the cutting shoe shown in Figure 5. The specimen was weighed and set in a stainless steel ring which confined the specimen laterally. The ring and specimen were placed over the bottom porous stone, and the ring was clamped to the chamber base. A porous stone and cap were placed on top of the specimen. After assembling the chamber, the ram was brought into contact with the cap. A linear variable displacement transducer (LVDT) and strain-gage type transducer were installed. The chamber was filled with de-aired water. A back
A pressure of 10 psi (69 kPa) was applied to the top and bottom of the specimen. Swell was prevented by the fixed ram. Approximately 10-15 hours after applying the back pressure, the drainage line to the bottom of the specimen was closed, and the specimen was loaded at a constant rate of strain. When a total vertical stress of approximately 30 tsf (3 MPa) was reached, the loading phase was stopped. The specimen was then unloaded at the same strain rate used in the loading phase.

Pore-pressure transducer, load transducer, LVDT, and elapsed time readings were taken periodically by an automatic acquisition system and recorded onto magnetic tape for subsequent processing. The data acquisition system and tape drive are shown in Figure 6. Data reduction is shown in Equations 1 through 4:

\[ \varepsilon = \frac{\Delta H}{H_0}, \]  
\[ \tau = \frac{\Delta \varepsilon}{\Delta t}, \]  
\[ \sigma'_{v} = \sigma_{v} - 2u_{b}/3, \]  
\[ C_v = \frac{(\Delta \sigma_{v}/\Delta t)H^2/2u_{b}}, \]

in which \( \varepsilon \) = strain, \( \Delta H \) = change in height from initial height, \( H_0 \) = initial height, \( \tau \) = strain rate, \( \Delta \varepsilon \) = change in strain in time interval \( \Delta t \), \( \Delta t \) = time interval, \( \sigma_{v} \) = vertical stress, \( \sigma'_{v} \) = effective vertical stress, \( u_{b} \) = excess pore-water pressure developed at the base of the specimen, \( C_v \) = coefficient of consolidation, \( \Delta \sigma_{v} \) = change in vertical stress in time interval \( \Delta t \), and \( H \) = \( H_0 \cdot \Delta H \).

The equation for \( C_v \) is based on the assumption that initial transient conditions, as described by Wissa (2), have dissipated. All values of \( C_v \) reported herein were computed after \( u_{b} \) reached a value of 0.5 psi (3 kPa).
STRESS-STRAIN DATA

The results of five CRS tests at various strain rates and of two incremental-load consolidation tests are shown in Figure 7. Strain for the incremental-load tests were calculated from the deformation corresponding to 100 percent hydrodynamic consolidation, according to ASTM Standard Method of Test for One-Dimensional Consolidation Properties of Soils, Designation D 2435; and all load increments were maintained for a 24-hour period. The shifting of stress-strain curves in the direction of increasing stress at
higher strain rates is a well-documented phenomenon (1, 4, 6, 7, 8). Such shifting suggests that primary and secondary consolidation do not occur at separate time intervals during the consolidation process but, rather, occur simultaneously; and the amount of secondary consolidation is dependent on strain rate. It is important to note that this phenomenon is not unique to the CRS test but also occurs in incremental load tests. Crawford (6) noted this "... increasing contribution of secondary consolidation with long increments of loading ..." in a series of incremental-load and CRS tests. Lowe (8) explained the phenomenon with a new, more realistic mechanical model in his modified Terzaghi consolidation theory. The model is shown in Figure 8; his description of the model follows:

"In the model for the Modified Theory, the coiled springs (Terzaghi Model) are replaced with leaf springs having a filling of viscous material between the leaves. The load these leaf springs can carry depends upon the rate of compression imposed. If the rate of compression imposed is very fast, the viscous material between the leaves develops appreciable shear resistance and the stack of leaves acts as a unit. If the rate of compression imposed is very slow, the viscous filling material can deform readily and each of the leaves acts as an individual leaf. The resistance to compression when the stack of leaves acts as individuals is much less than the resistance when the stack acts as a combined beam as under the fast loading condition." Therefore, the effect of increasing strain rate on stress-strain data in the CRS test is not attributable to, or unique to, the CRS test itself. The effect is a result of the viscous nature of the particular soil being tested. The lower strain rates, which produce essentially the same stress-strain curves as incremental-load tests with 24-hour load-increments, are preferred in the development of strain-rate selection criteria. The lower range of strain rates is preferred because the stress-strain
curves seem less sensitive to strain rate in this range and because field conditions are more closely duplicated by slower rates.

**Cv DATA**

Values of $C_v$ calculated from CRS test data were affected by strain rate. $C_v$'s for the CRS tests at different strain rates and for the incremental load tests are shown in Figure 9. A progressive increase in $C_v$ was noted in the early portions of the CRS tests for progressively increasing strain rates. Eventually, at higher values of $\sigma_v'$, all values of $C_v$ converged. The value of $\sigma_v'$ at convergence also increased with increasing strain rate.

This phenomenon is caused by two factors. The first is based on the decrease in permeability with decreasing void ratio. As the sample is strained, the void ratio decreases and permeability decreases; this tends to decrease $C_v$. Figure 10 illustrates this point at strain rates of 0.0174 and 0.0963 percent per minute. As the strain rate increases, strain, for a given value of $\sigma_v'$, decreases. Therefore, as the strain rate increases, $C_v$ at the given value of $\sigma_v'$ should increase. This is shown by the data only within a narrow range of $\sigma_v'$ values. For this reason, and because other factors obviously influence $C_v$, no attempt was made to quantify this trend.

The second factor, which produces the rate effect shown in Figure 9, concerns the assumption of a stress-strain relationship for the material being tested. To obtain Equation 4, it was necessary to assume a linear stress-strain relation of the form

$$m_v = \Delta e/\Delta \sigma_v',$$

5

in which $\Delta \sigma_v' = $ change in effective vertical stress and $m_v = $ coefficient of volume compressibility. Wissa (2) showed the consequences of an error in this assumption by assuming a nonlinear stress-strain relation of the form

$$C_v = -\Delta e/\Delta (\log \sigma_v').$$

6

Substitution of this assumption for the linear stress-strain assumption in the derivation of $C_v$ yields

$$C_v = H^2 \log(\sigma_v2/\sigma_v1) \div (2\Delta t \log (1 - u_b/\sigma_v)),$$

7

in which $\sigma_v1$ and $\sigma_v2$ = total vertical stress at times $t_1$ and $t_2$, respectively, and $\Delta t = t_2 - t_1$. The relationship between the linear theory $C_v$ and nonlinear theory $C_v$ is expressed by combining Equations 4 and 7, yielding

$$C_v \text{ linear}/C_v \text{ nonlinear} = -\log (1 - u_b/\sigma_v) \div (0.434 u_b/\sigma_v).$$

8

The difference between the two solutions is solely a function of $u_b/\sigma_v$ and is expressed graphically in Figure 11; the solutions diverge as $u_b/\sigma_v$ increases. The variation of $u_b/\sigma_v$ throughout the CRS tests is shown as a function of $\sigma_v'$ in Figure 12. The quantity reaches a maximum early in the test and varies over a wide range throughout the test. The way in which the
Figure 9. $C_v$ Data from CRS and Incremental-Load Consolidation Tests on Vac-Aire Extruded Specimens.

Figure 10. Comparison of Stress-Strain Data and $C_v$ Data from CRS Tests.
stress-strain assumption, and hence the magnitude of \( \frac{u_b}{a_v} \) affect \( C_v \) is clearly shown in Figures 11 and 12. These figures quantify the effect of an error in the stress-strain assumption and explain, in part, the phenomenon noted for the \( C_v \) data in Figure 9.

The obvious solution to this problem is to limit the value of \( \frac{u_b}{a_v} \) during the test, and this has been suggested \((2, 3, 4)\). However, some means of predicting \( \frac{u_b}{a_v} \) for a given soil and strain rate is necessary to incorporate \( \frac{u_b}{a_v} \) limits into the rate selection process. Furthermore, because \( \frac{u_b}{a_v} \) is not constant during the CRS test, it appears that a critical value of \( \frac{u_b}{a_v} \) must be expressed as a function of \( a_v \).

PARAMETERS AFFECTING \( \frac{u_b}{a_v} \)

The way in which strain rate affects CRS consolidation test results, specifically \( C_{v}, P_c \), and \( C_v \), and the ability of \( \frac{u_b}{a_v} \) to serve as an indicator of when results may be considered reasonable has been shown. Sufficient data exist to establish the relationship between strain rate and \( \frac{u_b}{a_v} \) for the particular kaolinite soil used in the tests herein. To extend this relationship to cover a wide range of soils, the relationship must be expressed in terms of the parameters to which it is most sensitive.

Figure 11. Comparison of \( C_v \) from Linear and Nonlinear Theory (after Wissa \((2)\)).

The parameters which most affect \( \frac{u_b}{a_v} \) are initial degree of saturation, \( S \), coefficient of consolidation, \( C_u \), and initial height, \( H \). \( H \) is a soil-independent parameter. Theoretically, \( \frac{u_b}{a_v} \) should decrease as a function of the decrease in \( H \) to the \( 1/2 \) power.

Figure 12. \( \frac{u_b}{a_v} \) versus \( a_v \) from CRS Tests on Vac-Aire Extruded Specimens.
Figure 13. Results of CRS Tests on Partially-Saturated Kaolinite.
Initial height was not varied in the testing program and was not considered as a variable in the development of strain-rate selection criteria. All tests were performed on specimens 25.4 mm (1.0 inch) high. If H reduction is used as a method of reducing $u_b/\sigma_v$, a minimum of H of 0.5 inch (12.7 mm) and a minimum diameter to height ratio of 2.5 should be maintained.

S and $C_v$ are soil-dependent and will be used in the development of strain-rate selection criteria. A general discussion of their effect on $u_b/\sigma_v$ follows.

By definition,

$$C_v = k/m \gamma_w,$$

in which $k$ = permeability and $\gamma_w$ = unit weight of water. Therefore, the term $C_v$ couples permeability with compressibility, two properties which govern the rate of consolidation. The higher the permeability, the faster the soil will consolidate; the higher the compressibility, the slower it will consolidate. When the soil is loaded at a constant rate of strain, the higher the permeability the slower the pore pressure will rise; the higher the compressibility, the faster it will rise. The modified Terzaghi model shown in Figure 8 is helpful in visualizing this pore-pressure response. The term $u_b/\sigma_v$ is, therefore, inversely proportional to $C_v$.

The fact that the parameter $C_v$ alone is sufficient to describe the consolidation rate process reveals that it is the single most important parameter in determining $u_b/\sigma_v$ for a given strain rate.

Before considering S as a parameter which affects $u_b/\sigma_v$, it should be noted that complete saturation of the soil, $S = 100$ percent, is a basic and necessary assumption in all consolidation theories. This, together with the difficulty in determining the effective stress in partially saturated soils, should eliminate consideration of soils not completely saturated. However, many partially saturated soils exist and are tested in practice. Such test results have been used successfully. Therefore, although it is somewhat of a contradiction, the effect of the parameter S on $u_b/\sigma_v$ must be studied.

CRS test results for the partially-saturated remolded specimens previously described are shown in Figure 13. The plots of $u_b/\sigma_v$ versus $\sigma_v$ for these tests show increasing $u_b/\sigma_v$ with increasing S. As S increases, in the different tests, the value of $\sigma_v$ at which $u_b/\sigma_v$ begins to build up decreases. As the initial degree of saturation decreases, an increasingly greater amount of compression is needed to force any entrapped air into solution. The average pore pressure will rise very slowly until the pore water flows into and fills the voids. Data from the CRS tests shown in Figure 13 will be used later in this report to incorporate the effect of S into the strain-rate selection process.

It was stated earlier that the critical value of $u_b/\sigma_v$ must be expressed as a function of $\sigma_v$· Study of the parameters affecting $u_b/\sigma_v$ implies that $u_b/\sigma_v$ should be limited at all values of $\sigma_v$ above $P_c$. The low compressibility below $P_c$ and, in some cases, the effect of partial saturation inhibit pore-pressure buildup. This can lead to erratic values of $u_b/\sigma_v$ below $P_c$. These values will, therefore, be discounted.

Development of the Strain-Rate Selection Procedure

**DEVELOPMENT OF THE RELATIONSHIP BETWEEN $r$, $C_v$, AND $u_b/\sigma_v$.**

Four variables will be considered in the development of a strain-rate selection procedure: $r$, $C_v$, $u_b/\sigma_v$, and S. At this step in the development, S is not included and, for that reason, all data are from tests on saturated soils. The variables $r$, $C_v$ and $u_b/\sigma_v$ will be related in a way that describes both the CRS tests on kaolinite, presented herein, and the CRS tests on soil types presented elsewhere. All values of $u_b/\sigma_v$ reported in this development, are the maximum values of $u_b/\sigma_v$ within the range of effective stress above $P_c$.

The relationship between $r$ and $u_b/\sigma_v$ for the kaolinite is shown graphically in Figure 14. Theoretically, $u_b/\sigma_v$ should equal zero for a zero strain rate and increase as the strain rate is increased. A maximum $u_b/\sigma_v$ of one is finally approached as $r$ becomes very large. The value of $u_b/\sigma_v$ from Test No. 23 did not fit this trend because the high strain rate caused a shift in the stress-strain curve which raised $P_c$, changing the selection point for maximum $u_b/\sigma_v$. This test was discounted. The general shape of the remaining points

**Figure 14. Relationship between Strain Rate and $u_b/\sigma_v$ from CRS Tests on Vac-Aire Extruded Specimens.**
and the boundary conditions for the $u_b/\sigma_v$ versus $r$ relationship may be expressed by an equation of the form

$$y = 1 \cdot e^{-0.22 \cdot t/C_v},$$

in which $y$ and $x$ are the ordinate and abscissa. An equation of this form was fit to valid CRS test data. The final form of the equation, which best described remolded kaolinite behavior and undisturbed soil sample behavior, was

$$u_b/\sigma_v = 1 - e^{-0.22 \cdot t/C_v},$$

in which $t$ is expressed in percent per minute and $C_v$ is expressed in inches per minute. Equation 11 is shown in Figure 14 for the kaolinite ($C_v = 0.02 \text{ in.}^2/\text{min}$ (13 mm$^2$/min)) and is used to predict $u_b/\sigma_v$ for tests on undisturbed specimens in Table 4. Equation 11 is solved in graphical form, for the variable $C_v$, in Figure 15. This chart may be used to select strain rate for saturated soils if $C_v$ is known prior to testing. The dashed lines on the chart illustrate the procedure. To use the chart in this way, a maximum allowable value of $u_b/\sigma_v$ must also be selected; $u_b/\sigma_v = 0.30$ was arbitrarily chosen for the example shown on the chart.

**STRAIN-RATE SELECTION GUIDELINES**

The selection of the maximum value of $u_b/\sigma_v$ is an important step in the strain-rate selection process. If the $u_b/\sigma_v$ limit is set too low, strain rates will be chosen which are too low to generate any significant pore pressure in the specimen, thereby rendering Equation 4, for $C_v$, meaningless. In addition, a low $u_b/\sigma_v$ limit will add significantly to the time required to complete a test. If the $u_b/\sigma_v$ limit is set too high, the consequences already outlined will prevail. Based on the following observations, a limiting value of $u_b/\sigma_v$ equal to 0.20 seems reasonable:

1. The pore pressure gradient was established within a reasonable length of time for the CRS test on kaolinite at this limit.
2. The maximum theoretical error in $C_v$, due to an error in the stress-strain relationship assumption, is approximately 12 percent.
3. The CRS test on kaolinite at this limit produced stress-strain data in agreement with incremental-load tests and was completed within 32 hours.

By limiting the value of $u_b/\sigma_v$ to less than 0.20 in the effective stress range above $P_c$, $u_b/\sigma_v$ was eliminated as a variable in the strain-rate selection process. The strain-rate selection process for saturated soils was reduced to a function of $C_v$ only.

However, $C_v$ usually is not known or not easily determined prior to testing. An indirect method of determining $C_v$ that has met with some success is a...

![Figure 15. Strain-Rate Selection Chart for Saturated Soils Based on $C_v$ and $u_b/\sigma_v$.]

**TABLE 4. PREDICTION OF $u_b/\sigma_v$ FOR CRS TESTS ON UNDISTURBED SPECIMENS USING EQUATION 11**

<table>
<thead>
<tr>
<th>CRS Test No.</th>
<th>$C_v$ (min$^{-2}$/sec)</th>
<th>$C_v$ (in.$^2$/min)</th>
<th>Strain Rate (%/min)</th>
<th>Predicted $u_b/\sigma_v$</th>
<th>Actual $u_b/\sigma_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRS-07</td>
<td>0.60</td>
<td>0.060</td>
<td>0.015</td>
<td>0.05</td>
<td>0.07</td>
</tr>
<tr>
<td>CRS-15</td>
<td>0.085</td>
<td>0.0085</td>
<td>0.017</td>
<td>0.35</td>
<td>0.14</td>
</tr>
<tr>
<td>CRS-18</td>
<td>0.07</td>
<td>0.007</td>
<td>0.0065</td>
<td>0.18</td>
<td>0.04</td>
</tr>
<tr>
<td>CRS-19</td>
<td>0.04</td>
<td>0.004</td>
<td>0.0050</td>
<td>0.24</td>
<td>0.14</td>
</tr>
<tr>
<td>KY-320</td>
<td>1.0</td>
<td>0.10</td>
<td>0.016</td>
<td>0.03</td>
<td>0.02</td>
</tr>
</tbody>
</table>
TABLE 5. ADJUSTED STRAIN RATES FOR SATURATED SOILS

<table>
<thead>
<tr>
<th>CRS Test No.</th>
<th>$C_v$ (mm$^2$/sec)</th>
<th>$C_v$ (in.$^2$/min)</th>
<th>Actual Strain Rate (%)/min</th>
<th>Predicted Strain Rate (%)/min</th>
<th>Liquid Limit (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRS-07</td>
<td>0.60</td>
<td>0.06</td>
<td>0.015</td>
<td>0.061</td>
<td>27</td>
</tr>
<tr>
<td>CRS-15</td>
<td>0.08</td>
<td>0.008</td>
<td>0.017</td>
<td>0.0081</td>
<td>62</td>
</tr>
<tr>
<td>CRS-18</td>
<td>0.07</td>
<td>0.007</td>
<td>0.0065</td>
<td>0.0071</td>
<td>84</td>
</tr>
<tr>
<td>CRS-19</td>
<td>0.04</td>
<td>0.004</td>
<td>0.0050</td>
<td>0.0041</td>
<td>62</td>
</tr>
<tr>
<td>KY 320 07</td>
<td>1.0</td>
<td>0.10</td>
<td>0.016</td>
<td>0.101</td>
<td>29</td>
</tr>
<tr>
<td>07</td>
<td>0.20</td>
<td>0.02</td>
<td>0.033</td>
<td>0.020</td>
<td>59</td>
</tr>
</tbody>
</table>

1. The strain rate necessary, as predicted by Equation 12, to produce a value of $u_b/a_v$ equal to 20 percent.

The results of Equation 12 are shown as predicted strain rate in Table 5.

The predicted strain rates are shown as a function of liquid limit in Figure 16. An arithmetic scale was used for liquid limit, replacing the logarithmic scale for $C_v$ used in the previous strain-rate selection chart. The data should approximate a straight line when plotted to these scales. A reasonable straight-line representation of the data is shown on the figure. This chart forms the strain-rate selection criteria for saturated soils. The final step in the development of the strain-rate selection chart was the correction for partially saturated soils.

Data for partially-saturated remolded kaolinite, shown in Figure 13, were used to develop criteria for partially-saturated soils. The approach was similar to that for saturated soils. The strain rate used in these tests did not produce a $u_b/a_v$ of 0.20 as required in the selection criteria. For this reason, the actual strain rates in these tests were used to calculate the rate required to produce $u_b/a_v$ equal to 0.20.

This adjustment was based on solutions to Equation 11 for the actual maximum value of $u_b/a_v$ above $P_c$ and for the value of $u_b/a_v$ equal to 0.20 yielding

$$r_{20} = r_{\text{actual}} \ln (1 - 0.20)$$

$$+ \ln \left(1 - \frac{u_b}{a_v (\text{actual})}\right),$$

![Figure 16. Strain-Rate Selection Criteria for Saturated Soils.](image)
in which \( r_{0.20} \) = the adjusted strain rate required to produce \( u_b/\sigma_y \) equal to 0.20,
\[ r_{\text{actual}} = \] the actual strain rate in the CRS test, and
\[ u_b/\sigma_y(\text{actual}) = \] the actual maximum value of \( u_b/\sigma_y \) in the effective stress range above \( P_c \).

Values of \( r_{0.20} \) were plotted versus the liquid limit of kaolinite on the strain-rate selection chart in Figure 17. Assuming that the same straight-line relationship applies to partially saturated soils as applies to the saturated soils, lines parallel to the line for saturated soils were drawn through the points for partially saturated soil in Figure 17. To quantify the relationship between \( S \) and strain rate, the perpendicular distance between the parallel lines was plotted versus \( 100 - S \) in Figure 18. The equation describing the best straight-line fit to the data was

\[ y = 1.82x, \quad 14 \]

in which \( y \) = the perpendicular distance from the 100-percent saturation line and
\[ x = 100 - S \] in percent.

Given this function, lines may be drawn for any initial degree of saturation. This was done for various values of \( S \) in Figure 19.

Finally, an arbitrary maximum strain rate of 0.05 percent per minute was established; this permits com-
pletion of a CRS test within approximately one working day. When strain rates near or above the maximum are used, transient conditions described by Wissa (2) should be expected. Figure 19 allows the pre-selection of strain rate for CRS tests based on liquid limit and initial degree of saturation. As mentioned previously, considerable latitude exists in the choice of a proper strain rate; therefore, the value given by the chart form guidelines for the strain-rate selection process.

Conclusions

1. The appropriate rate of strain in the CRS consolidation test is one which will limit the value of $u_g/\sigma_v$ to less than 0.20 in the effective stress range above $P_c$.
2. Some latitude exists in the selection of an appropriate strain rate.
3. The general range of appropriate strain rates for CRS consolidation testing is 0.0001 to 0.05 percent per minute.
4. If the value of $C_v$ for a particular saturated soil is known prior to testing, Figure 15 can be used to select the rate of strain.
5. If the value of $C_v$ for a particular soil is not known prior to testing, or if the soil is partially saturated, Figure 19 can be used to select the rate of strain based on the liquid limit and the initial degree of saturation.

Summary

A schema based on theoretical considerations and experimental data has been proposed for strain-rate selection in the CRS consolidation test. The strain-rate selection chart presented in Figure 19 not only establishes numerical guidelines but, more importantly, establishes a framework for strain-rate selection techniques based on the more important variables in the CRS test. As the CRS consolidation test becomes more widely used, the strain-rate selection chart may be modified based on new data that will be generated. This approach, together with sound engineering judgment, should eliminate the uncertainty in the strain-rate selection process.

References

9. Terzaghi, K.; and Peck, R. B.; Soil Mechanics