3. CONSOLIDATION TESTS

Constant rate of strain (CRS) testing was done to measure the consolidation and drainage properties of the Lake Bonneville Clays. CRS testing was chosen because of its advantages over conventional incremental loading (IL) consolidation testing methods. However, a few traditional IL consolidation tests were conducted to compare the results with those of CRS consolidation tests.

3.1 Constant Rate Strain (CRS) Consolidation Tests

3.1.1 CRS Consolidation Device

The CRS consolidation testing apparatus includes the axial loading frame, axial loading device (load cell), pore water transducer, deformation indicator (LVDT) and CRS consolidation cell (Figures 3.1 and 3.2).

The load frame manufactured by Geocomp Corporation was used as an axial loading device (LoadTrac system). The LoadTrac loading frame system uses a micro stepping motor set at 2000 steps per revolution to move the load platen. This translates to one step of the motor moving the load platen a distance of $2.1 \times 10^{-6}$ inch. A control box signals the micro motor and allows the load platen to run smoothly. Also, upper and lower limit switches prevent the load platen from exceeding its travel limits. A schematic diagram of the automated LoadTrac is shown in Figure 3.1 and the set-up of the LoadTrac with CRS consolidation cell can be seen in Figure 3.2.
Figure 3.1 Diagram of fully automated LoadTrac

Figure 3.2 LoadTrac with CRS consolidation cell
An axial loading device and an S shaped load cell, which has maximum capacity of 2000 lb, were used. Also the LVDT selected has a sensitivity of 0.00001 of an inch and total 0.8 inch stroke range, which is more than minimum recommended 50% of the specimen height. A pore pressure transducer, with a capacity of 50 psi (approximately 350 kPa), was used. Before the start of the CRS testing, in the initial set-up, the strain rate, final load, final displacement and sampling period have to be programmed in to the control box. The final load and final displacement options allow the operator to enter the values of load and displacement, which when either one is reached, the system will stop the test. As a final strain of 20% and a final load of 1500 lbs, which is approximately 2200 kPa were selected. For the most of the tests, 20% of the displacement controlled the end-of-test condition. A strain rate of 0.00009 inch/min, which is 0.54%/hr, was applied for all the CRS consolidation tests. A sampling period of 900 s, which is 15 minutes, was chosen for the load, displacement and pore pressure readings. To reach the final displacement of 20% with the 0.54%/hr strain rate, the total test duration was about 37 hours.

The control box provided by GeoComp was able to record load and LVDT readings with time, but it was not able to record pore pressure readings. The excess pore water pressure readings were made at the bottom of the sample using a Validyne’s pore pressure transducer and its own data collection system.

3.1.2 CRS Cell

The CRS consolidation cell was designed by the GeoComp so that pore pressure measurements can be taken during consolidation testing. It includes a stainless steel top cap with a piston passing through linear bearings and connected to the top pad. Take-off
ports allow for the application of confining a pressure up to 1400 kPa (200 psi). A bottom port is also provided for excess pressure monitoring. The disassembled and assembled CRS cell is shown in Figures 3.3, and 3.4, respectively.

Figure 3.3 CRS consolidation cell parts

Figure 3.4 CRS consolidation cell.
3.1.3 Sample Preparation and CRS Testing Procedure

All Shelby tubes collected from the research sites were stored in a humidity room to preserve their original water content. Prior the CRS testing, all tubes which contained Lake Bonneville Clays from S. Temple site were radiographed prior to consolidation testing. The radiographs were used to plan the laboratory test program and to select the best quality sample for CRS testing. A CT scan of B4/Tube #5, which sampling interval is from 1279.68m to 1279.07m, can be seen in Figure 3.5. Additional CT scan images of other tubes from the South and North Temple sites are found in Appendix C.

![B4/Tube #5: Scan#2](image)

Figure 3.5 CT scan image of B4/Tube #5 from S. Temple research site

Three-inch long sections of the Shelby tube were cut by a band saw prior to extrusion. This was done to minimize sample disturbance during extrusion. The samples were then extruded using a standard extruder. The consolidation ring used for the tests had a height of 1.0 inches and a diameter of 2.5 inches. Trimming of the specimen to fit the consolidation ring was carefully done using a wire saw to minimize sample disturbance. Before placing the soil in the consolidation ring, the inner circumference of the ring was lubricated with a low-friction lubricant to minimize disturbance. Excess soil was used for determining water content and soil classification. After soil specimens were placed in the ring, and the top and bottom of the specimens were trimmed flush with the
ring. Any small voids were carefully filled with remolded soil without disturbing the specimen. The ring and soil specimen was weighted for unit weight determination.

Prior to assembling the CRS cell, porous stones were saturated with de-aired water, then the ring with the specimen, porous stones and loading cap were assembled. The CRS cell was filled with water for application of the cell pressure and the bottom pore pressure transducer was also saturated. To saturate the specimen, the back pressure was applied thorough both the base and top porous stones. Prior to consolidation loading, the base drain valve was closed and the initial pore water pressure value was recorded to calculate the B value. A B value of 98% was achieved prior to loading. Details of the B value analysis are found in Head (1992). For all CRS consolidation tests, filter paper was used between the sample and the porous stones to prevent clogging of the stones and the loss of the solids during the test.

After the saturation was achieved, the strain rate, final load, final displacement and sampling period were entered into control box and the tests were started. The test was terminated after either the final load or final displacement had been achieved.

3.1.4. Data Reduction Method for CRS Consolidation Test

Data reduction of the CRS consolidation test results was done using Wissa’s et al. (1971) method, which has been standardized by ASTM D-4186. Wissa et al. (1971) developed both linear and nonlinear CRS consolidation theory based on assuming a constant $c_v$ but allowing $m_v$ and $k_v$ vary. Both theories were developed for both steady state and for transient conditions. For steady state conditions, the strain distribution remains constant with the dimensionless time factor, $T$. For transient conditions, which occurs at the start of loading, the strain distribution is not constant. For a nonlinear, but
constant compression index, $C_r$, the strain is proportional to the change in the logarithm of the effective stress. The factor $F_3$ is related to the total vertical and effective stresses as:

$$F_3 = \frac{\log(\sigma_v - u_h) - \log(\sigma_v @ t = 0)}{\log(\sigma_v) - \log(\sigma_v @ t = 0)} \quad (3.1)$$

$F_3$ can then be related to $T$ using an empirical equation. A regression analysis of the plot of $T$ versus $F_3$ gives a simplified equation for $T$:

$$T = 4.78(F_3)^3 - 3.21(F_3)^2 + 1.65F_3 + 0.0356 \quad (3.2)$$

When $T$ is greater than 0.5, steady state condition can be assumed. The steady state solution of Wissa et al. (1971) has been standardized by ASTM in D4186. ASTM requires that the excess pore pressure at the base of the CRS cell be limited to 30% of the total vertical stress at any time during the test. If steady conditions exist, then:

$$\sigma_v' = \left(\sigma_v^3 - 2\sigma_v^2u_h + \sigma_vu_h^2\right)^{1/3} \quad (3.3)$$

$$c_v = -\frac{H^2 \log\left(\frac{\sigma_{v2}}{\sigma_{v1}}\right)}{2\Delta t \log\left(1 - \frac{u_h}{\sigma_v}\right)} \quad (3.4)$$

$$k_v = \frac{0.434\gamma \gamma_w}{2\sigma_v' \log\left(\frac{\sigma_v - \Delta u_h}{\sigma_v}\right)} \quad (3.5)$$

where $r =$ constant rate of strain, $\sigma_{v1}$ and $\sigma_{v2} =$ total stress at the beginning and end of $\Delta t$, respectively, $u_h =$ excess pore pressure at the bottom of the sample, $\sigma_v' =$ effective stress (average value over $\Delta t$) and $H =$ current specimen height. For values of $T$ smaller
than 0.5 (i.e., transient conditions), $c_v$ and $k_v$ can be found by using Terzaghi’s 1-D consolidation theory.

3.1.5. Calculation of Preconsolidation Stress from Laboratory Data

There are different methods of determining the preconsolidation stress, $\sigma'_p$, from laboratory oedometer data. Some of the more common ones are:

1. Casagrande’s method (Casagrande, 1936)
2. $\ln(1 + e)$ vs. log P method (Butterfield, 1979)
3. $\sigma'_v$ vs. time method (Leroueil, and LeBihan, 1980)
4. Work method (Becker and Jeferies, 1987)
5. Log-Log method (log e vs. log P method, Babu, Sridharam, and Abraham, 1989)
6. $1+e$ vs. log P method (Sridharan, Abraham, and Jose, 1991)

Because Casagrande’s method is the most commonly used procedure for determining the preconsolidation stress, this method was used in this report. Figure 3.12 shows the Casagrande procedure as described in Holtz and Kovacs (1981). The following steps describe this procedure.

1. Choose by eye the point of minimum radius (or maximum curvature) on the consolidation curve (point A in Fig. 3.6)
2. Draw a horizontal line from point A.
3. Draw a line tangent to the curve at point A.
4. Bisect the angle made by steps 2 and 3.
5. Extend the straight-line portion of the virgin compression curve up to
where it meets the bisector line obtained in step 4. The point of intersection of these two lines is the preconsolidation stress (point B of Fig. 3.6).

Preconsolidation stresses for the North Temple site were determined using this method as seen in Table 3.1. Results for the other research sites are presented in Appendix D.

Figure 3.6 The Casagrande procedures for determining the preconsolidation pressure
3.1.6 Calculation of Field Consolidation Curve from Laboratory Data

The slope of the virgin compression curve must be corrected to adequately estimate the compression index, $C_c$, and the compression ratio, $CR$. This correction is necessary to account for disturbance introduced into the laboratory results by sampling and handling. Such effects tend to decrease the slope of virgin compression curve, hence it must be corrected to more adequately represent the field behavior. Schmertmann (1955) developed a graphical procedure to adjust the slope of the laboratory virgin compression curve to the field virgin compression curve. The Schmertmann procedure for an overconsolidated soil (Holtz and Kovacs, 1981) is as follows (Figure 3.7):

1. Perform the Casagrande (1936) construction and evaluate the
preconsolidation pressure, $\sigma_p'$.  
2. Calculate the initial void ratio, $e_o$. Draw a horizontal line from $e_o$ that parallels the log of the effective stress axis to the existing vertical overburden pressure $\sigma_{vo}'$. This establishes control point 1 as illustrated by triangle 1 in Figure 3.7.
3. From control point 1, draw a line parallel to the rebound-reload curve to the preconsolidation pressure $\sigma_p'$. This will establish control point 2, as shown by triangle 2 in Figure 3.7.
4. From a point on the void ratio axis equal to 0.42 $e_o$, draw a horizontal line to where the this line meets the extension of the laboratory virgin compression curve L. This establishes a third control point, as shown by triangle 3. The

3-10
coefficient of 0.42 times $e_o$ is not a “magic number,” but is a result of many observations on different clays as discussed by Schemertmann (1955).

5. Connect control points 1 and 2, and 2 and 3 by straight lines. The slope of line F joining control points 2 and 3 defines the compression index, $C_c$, for the field virgin compression curve. The slope of the line joining control points 1 and 2 represents the recompression index, $C_r$.

![Figure 3.7 Illustrations of Schmertmann Procedures to Obtain the Field Virgin Compression Curve for OC Soil](image)

The compression ratio, $CR$, can be calculated as:

$$CR = C_c / (1 + e_o)$$

(3.6)
3.1.7 CRS Consolidation Tests Results

A total of 42 CRS consolidation tests were conducted for the Lake Bonneville Clays from South and North Temple research sites. The test data are plotted in the form of vertical axial strain, $\varepsilon_v$, versus effective stress, $\sigma_v'$ (Figure 3.8); void ratio, $e$, versus effective stress, $\sigma_v'$ (Figure 3.9); and excess pore pressure at the bottom of the sample, $u$, versus effective stress, $\sigma_v'$ (Figure 3.10).

![Effective Stress vs. Vertical Strain](image)

**Figure 3.8 CRS test results of $\varepsilon_v$ vs. $\sigma_v'$ for North Temple 1278.22m**

As seen from Figure 3.8 and 3.9, it is easier to find the maximum curvature point using CRS test results when compared with results from traditional incremental oedometer tests. CRS consolidation curves reduce the subjective judgment required to identify the maximum curvature point and subsequently reduce the uncertainty in estimating the pre-consolidation pressure.
Figure 3.9 CRS test results of $e$ vs. $\sigma_v'$ for North Temple 1278.22m

Figure 3.10 CRS test results of $u$ vs. $\sigma_v'$ for North Temple 1278.22m
Wissa et al. method was used for data reduction of CRS consolidation tests. This method was chosen because the steady state solution of Wissa et al. (1971) has been standardized by ASTM D4186 with the base excess pore pressure restricted to 30% of the total vertical stress at any time of the test. As can be seen on Figure 3.11, the actual stress-strain curve from North Temple site compares well with both the linear and non-linear CRS theory of Wissa et al. (1971). This test produced 11% maximum $u/\sigma$ ratio, which is less than the recommended 30% maximum $u/\sigma$ ratio allowed by ASTM D4186.

![Effective Stress vs. Vertical Strain](image)

**Figure 3.11** $\varepsilon$ vs. $\sigma'_v$ for North Temple 1278.22m comparison with ASTM D4186
### Table 3.1 Summary of CRS test Results (North Temple Site, R=0.009%/min)

<table>
<thead>
<tr>
<th>Elevation of test (m)</th>
<th>Overburden (kPa)</th>
<th>Max. Stress (kPa)</th>
<th>Casagrande's Procedure (kPa)</th>
<th>OCR</th>
<th>CR</th>
<th>ku</th>
<th>or</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(kPa)</td>
<td>(%)</td>
<td>(kPa)</td>
<td>(%)</td>
<td></td>
<td>s/d</td>
<td>d0 to d0, d0 to rest of the test, d0 to d0, d0 to rest of the test</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1281.870</td>
<td>76</td>
<td>20</td>
<td>95</td>
<td>125</td>
<td>139</td>
<td>1.66</td>
<td>0.44</td>
</tr>
<tr>
<td>1280.303</td>
<td>86</td>
<td>9</td>
<td>82</td>
<td>135</td>
<td>183</td>
<td>1.57</td>
<td>0.20</td>
</tr>
<tr>
<td>1279.070</td>
<td>90</td>
<td>12</td>
<td>106</td>
<td>146</td>
<td>166</td>
<td>1.63</td>
<td>0.26</td>
</tr>
<tr>
<td>1279.520</td>
<td>93</td>
<td>17</td>
<td>103</td>
<td>130</td>
<td>137</td>
<td>1.40</td>
<td>0.40</td>
</tr>
<tr>
<td>1278.380</td>
<td>96</td>
<td>3</td>
<td>126</td>
<td>163</td>
<td>168</td>
<td>1.70</td>
<td>0.43</td>
</tr>
<tr>
<td>1278.220</td>
<td>101</td>
<td>11</td>
<td>122</td>
<td>164</td>
<td>189</td>
<td>1.62</td>
<td>0.32</td>
</tr>
<tr>
<td>1277.300</td>
<td>103</td>
<td>28</td>
<td>87</td>
<td>159</td>
<td>187</td>
<td>1.54</td>
<td>0.23</td>
</tr>
<tr>
<td>1274.520</td>
<td>134</td>
<td>54</td>
<td>133</td>
<td>203</td>
<td>256</td>
<td>1.56</td>
<td>0.21</td>
</tr>
<tr>
<td>1274.220</td>
<td>137</td>
<td>24</td>
<td>111</td>
<td>180</td>
<td>262</td>
<td>1.32</td>
<td>0.22</td>
</tr>
<tr>
<td>1273.520</td>
<td>143</td>
<td>49</td>
<td>122</td>
<td>195</td>
<td>264</td>
<td>1.36</td>
<td>0.26</td>
</tr>
<tr>
<td>1272.870</td>
<td>148</td>
<td>46</td>
<td>311</td>
<td>340</td>
<td>366</td>
<td>2.35</td>
<td>0.37</td>
</tr>
<tr>
<td>1272.220</td>
<td>154</td>
<td>43</td>
<td>116</td>
<td>167</td>
<td>234</td>
<td>1.22</td>
<td>0.24</td>
</tr>
<tr>
<td>1271.870</td>
<td>157</td>
<td>17</td>
<td>128</td>
<td>164</td>
<td>164</td>
<td>1.04</td>
<td>0.37</td>
</tr>
<tr>
<td>1271.540</td>
<td>169</td>
<td>30</td>
<td>130</td>
<td>181</td>
<td>238</td>
<td>1.13</td>
<td>0.30</td>
</tr>
<tr>
<td>1270.886</td>
<td>169</td>
<td>45</td>
<td>154</td>
<td>253</td>
<td>400</td>
<td>1.50</td>
<td>0.19</td>
</tr>
<tr>
<td>1270.483</td>
<td>170</td>
<td>38</td>
<td>112</td>
<td>228</td>
<td>322</td>
<td>1.34</td>
<td>0.18</td>
</tr>
</tbody>
</table>
However, the 30% maximum \( u/\sigma \) ratio recommendation was not followed for all tests performed herein. As can be seen in Table 3.1, which summarizes all CRS consolidation testing results for North Temple site, the maximum \( u/\sigma \) ratio varies between 3% and 54% and as can be seen in Appendix D for South Temple site, the maximum \( u/\sigma \) ratio varies between 3% and 65%. Nonetheless it was found that Wissa theory compared well with all tests conducted for Lake Bonneville clays, even when they produced more than 30% of the maximum \( u/\sigma \) ratio. This suggests that the 30% \( u/\sigma \) maximum of ASTM D4186 is conservatively selected and values greater than this up to 65% can be used without greatly affecting the results from Wissa et al. theory.

All CRS test results and their respective \( e \) vs. \( \log \sigma' \) curves are presented in Appendix D. Vertical permeability, \( k \), versus average effective stress, \( \sigma'_v \), and coefficient of consolidation, \( c_v \), versus average effective stress, \( \sigma'_v \), plots shown in Figures 3.12 and Figure 3.13, respectively. These plots were also produced using Wissa et al. (1971) non-linear theory.
Figure 3.12 CRS test results of $k$ vs. $\sigma_v'$ for North Temple 1278.22m

Figure 3.13 CRS test results of $c_v$ vs. $\sigma_i'$ for North Temple 1278.22m
3.2 Incremental Loading (IL) Tests

To analyze the validity of the selected strain rate, the CRS test results were compared with results from IL oedometer tests. IL oedometer tests were chosen as the reference method because this test method has widespread use in geotechnical engineering practice. IL consolidation test results from North Temple site are presented in Table 3.2. As it can be seen, stress-strain curves from these two tests are very similar in the over consolidated part of the consolidation curve, but the IL consolidation curve shows a more stiff response in the normal consolidation part. The CR values for these tests are 0.2556 and 0.2372 for the CRS and IL methods, respectively. Of the 4 IL consolidation tests performed, the compression curves for 3 of them showed slightly stiffer response than the CRS test results.

Table 3.2 IL Consolidation Test Results (North Temple Site)

<table>
<thead>
<tr>
<th>Elevation of test (meter)</th>
<th>CR</th>
<th>Casagrande's Procedure Effective Pre-Cons. Pressure</th>
<th>Effective Overburden Stress (kPa)</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Min. Possible (kPa) Most Probable (kPa) Max. Possible (kPa)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1279.790</td>
<td>0.2385</td>
<td>110.40  169.37  269.07</td>
<td>90.01  1.88</td>
<td></td>
</tr>
<tr>
<td>1278.298</td>
<td>0.2651</td>
<td>100.00  148.59  234.42</td>
<td>100.44  1.48</td>
<td></td>
</tr>
<tr>
<td>1274.596</td>
<td>0.2311</td>
<td>103.35  236.81  387.41</td>
<td>133.33  1.78</td>
<td></td>
</tr>
<tr>
<td>1273.596</td>
<td>0.2372</td>
<td>130.22  213.98  331.78</td>
<td>141.44  1.51</td>
<td></td>
</tr>
</tbody>
</table>

Figures 3.15 and 3.16 show the $c_v$ and $k$ data, respectively, versus $\sigma'_v$. Values of $c_v$ for IL consolidation tests were calculated from the log$t$ method and the resulting $k$ values calculated from:

$$k = \gamma_w m_v c_v$$  \hspace{1cm} (3.7)
Figure 3.14 Comparison of compression curves from CRS and IL tests

Figure 3.15 Comparison of \( c_v \) values from CRS and IL tests
Figure 3.16 Comparison of $k$ values from CRS and IL tests

Additional IL consolidation compression curves and comparison of both $c_v$ and $k$ versus $\sigma'_v$ are found in Appendix E. Figures 3.17, 3.18, 3.19, 3.20 and 3.21 show the $\sigma'_p$, OCR, $c_v$, $k$, and CR data, respectively, versus elevation for the research sites. Table 3.3 gives the average consolidation properties for both upper and lower Bonneville Clay zones for the research sites.
Table 3.3 Summary of the average consolidation properties for the Bonneville Clay

<table>
<thead>
<tr>
<th>Cons. Properties</th>
<th>Research Sites</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>North Temple</td>
<td>South Temple</td>
<td>South Temple Embankment</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Upper Bonneville</td>
<td>Lower Bonneville</td>
<td>Upper Bonneville</td>
<td>Lower Bonneville</td>
<td>Upper Bonneville</td>
</tr>
<tr>
<td>$\sigma_p$ (kPa)</td>
<td>149.03</td>
<td>217.80</td>
<td>135.76</td>
<td>223.95</td>
<td>453.19</td>
</tr>
<tr>
<td>OCR</td>
<td>1.61</td>
<td>1.46</td>
<td>1.82</td>
<td>1.48</td>
<td>1.08</td>
</tr>
<tr>
<td>$c_v$ ($m^2/\text{year}$)</td>
<td>25.51</td>
<td>7.49</td>
<td>23.71</td>
<td>7.54</td>
<td>10.55</td>
</tr>
<tr>
<td>$k$ ($m/s$)</td>
<td>$4.84 \times 10^{-9}$</td>
<td>$4.01 \times 10^{-10}$</td>
<td>$2.73 \times 10^{-9}$</td>
<td>$6.25 \times 10^{-9}$</td>
<td>$3.79 \times 10^{-10}$</td>
</tr>
<tr>
<td>CR</td>
<td>0.31</td>
<td>0.26</td>
<td>0.37</td>
<td>0.25</td>
<td>0.27</td>
</tr>
</tbody>
</table>
Figure 3.17 Preconsolidation Pressure vs. Elevation
Figure 3.18 OCR vs. Elevation
Figure 4.19 $c_v$ vs. Elevation
Figure 4.20 $k_v$ vs. Elevation
Figure 4.21 CR vs. Elevation
Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading

This standard is issued under the fixed designation D 4186; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This test method covers the determination of the rate and magnitude of consolidation of soil when it is restrained laterally and drained axially and subjected to controlled-strain loading.

Note 1—The determination of the rate and magnitude of consolidation of soil when it is subjected to incremental loading is covered by Test Method D 2435.

1.2 The values stated in SI units are to be regarded as the standard. The values stated in inch-pound units are approximate.

1.3 This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

2. Referenced Documents

2.1 ASTM Standards:
D 422 Test Method for Particle-Size Analysis of Soils
D 653 Terminology Relating to Soil, Rock, and Contained Fluids
D 854 Test Method for Specific Gravity of Soils
D 1587 Practice for Thin-Walled Tube Sampling of Soils
D 2216 Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures
D 2435 Test Method for One-Dimensional Consolidation Properties of Soils
D 3550 Practice for Ring-Lined Barrel Sampling of Soils
D 4220 Practices for Perserving and Transporting Soil Samples
D 4318 Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

3. Significance and Use

3.1 Information concerning rate and magnitude of consolidation settlement of soil is essential in the design of earth and earth-supported structures. The results of this method may be used to analyze or estimate one-dimensional consolidation settlements and rates.

3.2 Strain Rate:

3.2.1 It is recognized that consolidation test results are strain-rate dependent. Strain rates recommended in this standard are within the range usually encountered in Test Method D 2435.

3.2.2 Field strain rates vary greatly with time, depth below the loaded area, and radial distance from the loaded area. Because field rates cannot be accurately determined or predicted, it is not feasible to relate the laboratory-test strain rate to the field strain rate. However, it may be feasible to relate field pore pressure ratios (u, e) to laboratory pore pressure ratios. Further research is needed in this area.

3.2.3 The constant-rate-of-strain consolidation test does not address the problem of strain-rate effects but does provide a means for studying strain rate effects.

3.3 This method is not applicable to soils of high permeability, such as sands and other coarse-grained soils, or to partially saturated soils.

3.4 This method makes the following assumptions:

3.4.1 The ratio of soil permeability to soil compressibility is constant.

3.4.2 Flow of soil pore water occurs only in the vertical direction.

3.4.3 Darcy's law for flow through porous media applies.

3.4.4 The soil is saturated.

3.4.5 The soil is homogeneous.

3.4.6 The compressibility of the soil grains and water is negligible.

3.4.7 The log stress versus strain relationship is linear during a short-time interval of loading, and

3.4.8 The distribution of excess pore-water pressure across the specimen is parabolic.

4. Terminology

4.1 Definitions—The definitions of terms used in this method shall be in accordance with Terminology D 653.

4.2 Descriptions of Terms Specific to this Standard:

4.2.1 back pressure—the pore-water pressure at the drainage boundary.

4.2.2 excess pore-water pressure, \( \Delta u \), the pore-water pressure developed at the impervious end of the specimen (usually the base of the specimen) in excess of the back pressure.

4.2.3 applied vertical stress, \( \sigma \), the axial stress applied at the drainage boundary in excess of the back pressure.

4.2.4 pore pressure ratio—the excess pore water pressure divided by the applied vertical stress.

5. Apparatus

5.1 Axial Loading Device—The axial compression device may be a geartronic with a screw loading device capable of being a part measuring load to an accuracy of ± 5.2. Axial hydraulic device capable of pressure transducers for measuring the load on the specimen.

5.3 Port water pressure transducer, with both actuated

5.4 B1. ying and This device, connected partially filled reservoirs or gas pressure measured hydraulic piston or a applying at prescribed change valving device This valve or to prevent the pore-water back press

5.5 Def:

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may be a screw jack driven by an electric motor through a geared transmission, a platform weighing scale equipped with a screw-jack activated yoke, a hydraulic or pneumatic loading device, or any other compression device with sufficient capacity and control to axially compress the specimen at the constant rate of strain prescribed in 9.6. If the axial loading device is outside the consolidometer, see 5.8.

5.2 Axial Load-Measuring Device—The axial load-measuring device may be a load ring, strain-gage load cell, hydraulic load cell, or any other load-measuring device capable of the accuracy prescribed in this subparagraph and may be a part of the axial loading device. The axial load-measuring device shall be capable of measuring the axial load to an accuracy of 0.25% of the maximum load applied to the specimen.

NOTE 2—For a constant rate of deformation to be transmitted from the axial loading device through the load-measuring device, it is important that the load-measuring device be relatively stiff. Some hydraulic load cells or proving rings may not provide sufficient stiffness.

5.3 Pore-Water Pressure-Measuring Device—The pore-water pressure-measuring device shall be a differential pressure transducer. Separate pressure transducers for measuring pore-water pressure at the base of the specimen and back pressures may be used if both have the required accuracy and both are monitored during the test. The device shall be constructed and located such that the pore-water pressure at the base of the specimen can be measured with negligible drainage of pore water from the base of the specimen on one side of the transducer. The other side of the transducer measures the back pressure applied to the specimen. Negligible drainage of pore water from the base of the specimen can be attained if the coefficient of volume change of the pore-pressure-measuring device and de-aired, water-filled cavities connecting the device to the base of the specimen is less than 10^-5 m^3/psa (10^-4 m^3/Pa). The pore-pressure-measuring device shall be capable of measuring the pore-water pressure at the base of the specimen to an accuracy of 0.25% of the maximum anticipated pore pressure.

5.4 Back Pressure-Maintaining Device, capable of applying and controlling the back pressure to within ±2.0%. This device may consist of a reservoir, it may have reservoirs connected to the top and bottom of the specimen and partially filled with de-aired water; the upper part of the reservoir shall be connected to a compressed gas supply, the gas pressure being controlled by a pressure regulator and measured by a pressure gage. (See Note 3.) However, a hydraulic system pressurized by a deadweight acting on a piston or any other pressure-maintaining device capable of applying and controlling the back pressure to the tolerance prescribed in this paragraph may be used. A low volume-change valve shall be provided in the back-pressure-maintaining device as near as possible to the base of the specimen. This valve, when open, shall permit the application of back pressure to the base of the specimen; when closed, shall prevent the drainage of water from the specimen base and pore-water pressure-measuring device to the reservoir of the back-pressure-maintaining device.

NOTE 3—All gas-water interfaces should be small in area relative to the area of the specimen and should be in reservoirs connected to the consolidometer by a length of small diameter tubing.

5.5 Deformation Indicator—The deformation indicator shall be a dial indicator or displacement transducer having a sensitivity of 0.002 mm (0.0001 in.) and a range of at least 50% of the specimen height, or other measuring device meeting these requirements for sensitivity and range.

5.6 Timer, indicating the elapsed testing time to the nearest 1 s for establishing the rates of strain application prescribed in 9.6.

5.7 Balances, devices for determining the mass of the soil specimens as well as portions of the apparatus. All measurements of mass should be accurate to 0.1%.

5.8 Consolidometer, to hold the specimen in a ring that is fixed to a rigid base, with porous stones on each face of the specimen. Any potentially submerged parts of the consolidometer shall be made of a material that is noncorrosive in relation to the soil or other parts of the consolidometer. The bottom of the ring shall form a leak-proof seal with the rigid base capable of withstanding internal pressures of 1400 kPa (200 psi). The consolidometer shall be constructed such that placement of the specimen into the ring and consolidometer will not entrap air at the base of the specimen. The axial loading device and back pressure-maintaining device may be an integral part of the consolidometer. If the design of the consolidometer is such that back pressures affect axial load readings (due to pressure pushing the piston from the consolidometer), the change in readings in changes in back pressure shall be determined by calibration. The consolidometer shall conform to the following requirements:

5.8.1 Minimum Specimen Diameter shall be 50 mm (2.0 in.) and shall be at least 6 mm (0.25 in.) less than the diameter of the sample tube if using undisturbed samples, except as indicated in 7.2.

5.8.2 Minimum Specimen Thickness shall be 20 mm (0.75 in.) but shall be not less than 10 times the maximum grain diameter as determined in accordance with Method D 422.

5.8.3 Minimum Specimen Diameter-to-Thickness Ratio shall be 2.5.

5.8.4 Thickness of the Ring shall be such that, under assumed hydrostatic stress conditions in the specimen, the change in diameter of the ring will not exceed 0.03% under the greatest load applied.

5.8.5 Ring shall be made of a material that is noncorrosive in relation to the soil and pore fluid being tested. The inner surface shall be highly polished or shall be coated with a low-friction material.

5.9 Porous Disk:

5.9.1 The porous stones shall be of silicon carbide, aluminum oxide, metal, or other suitable material that is not attacked by the soil or soil moisture and shall be of medium grade. For soft fine-grain soils, a fine-grade porous stone shall be used. The stone shall be fine enough that the soil will not extrude into the pores, but have sufficient permeability so as not to impede the flow of water from the specimen. (Exact criteria have not been established.)

5.9.2 The diameter of the top stone shall be 0.2 to 0.5 mm (0.01 to 0.02 in.) less than that of the ring.

5.9.3 The stone shall be thick enough to prevent breaking. The top stone shall be loaded through a corrosion-resistant plate of sufficient rigidity to prevent breakage of the stone.

5.10 Moist Room—In climates where moisture loss
during preparation exceeds 0.1 %, the specimen shall be prepared in a moist room.

5.11 Trimmer or Cylindrical Cutter, for trimming the specimen down to the inside diameter of the consolidometer ring with a minimum of disturbance.

5.12 Specimen-Measuring Device, capable of measuring specimen height and diameter to the nearest 0.02 mm (0.001 in.).

5.13 Drying Oven, that can be maintained at 230 ± 9°F (110 ± 5°C).

5.14 Miscellaneous Equipment—Specimen trimming and carving tools, including spatulas, knives, and wire saws, moisture content cans, and data sheets as required.

6. Sampling

6.1 Sampling and field investigation shall be conducted in accordance with Practice D 1587. Specimens cut from block samples may also be used.

6.2 If suitable specimens can be obtained using Practice D 3550, then they may be used.

6.3 Transport and handling of samples shall be conducted in accordance with Practice D 4220.

7. Specimens

7.1 Prepare the specimen so moisture loss is less than 0.1 %; if necessary, prepare the specimen in a moist room. Trim the specimen to the inside diameter of the consolidometer ring. Fill with remolded soil any minor indentations in the specimen that would leave voids between the specimen and the ring. Place the specimen in the ring and trim it flush with the plane surface of the ring. The surface must be smooth. A specimen ring with the cutting edge attached provides the most accurate fit in most soils.

7.2 Organic soils, such as peat, and those soils that are easily damaged, may be transferred directly from the sampling tube to the ring where the ring and tube sizes have been selected for this purpose, provided that the cutting edge of the ring has the same diameter as the sample.

7.3 Determine the mass and height of the specimen. Record the specimen mass, height, and diameter.

Note 4—Precautions should be taken to minimize disturbance of the soil or changes in moisture and density during specimen preparation; vibration, distortion, and compression must be avoided.

7.4 Use the material trimmed adjacent to the specimen (see 7.1) to determine the natural moisture content (based on dry mass) in accordance with Method D 2216 and the specific gravity in accordance with Test Method D 854. Determine initial wet weight of the specimen and its volume from the mass, diameter, and height of the specimen ring. A more accurate determination of the specimen dry weight and moisture is found by drying the specimen at the end of the test (see 9.11). The value determined from the trimmings is approximate but permits determining the void ratio before the test is complete. The specific gravity can be estimated where an accurate void ratio is not needed.

7.5 The liquid and plastic limits as determined by Test Method D 4318 are useful in identifying the soil and in correlating the results of tests on different soils. These tests may also be performed on the trimmings.

8. Preparation of Apparatus

8.1 De-air the water in the back pressure-maintaining device and pore-water pressure-measuring system (see 5.3 and 5.4).

8.2 Saturate the porous stones with de-aired water.

8.3 Place the bottom porous stone in the consolidometer so as not to entrap any air in the pore water pressure-measuring system.

Note 5—Guidelines for system saturation may be found in one or more of the references listed at the end of this method.

9. Procedure

9.1 Assemble the specimen, ring, porous stones, and loading plate in the consolidometer. Avoid entrapping any air between the bottom porous stone and the specimen.

9.2 Place the consolidometer in the axial loading device, adjust the deformation indicator for the initial or zero reading, and apply a seating pressure of 5 kPa (100 lb/ft²). The axial loading device may be set to maintain constant seating pressure or maintain a constant specimen height. If a constant specimen height is desired, the seating pressure required to maintain constant specimen height must be recorded. (For very soft soils, a seating pressure of 2.5 kPa (50 lb/ft²), or less, is desirable.)

9.3 Check to ensure that the system formed by the water reservoirs of the back pressure-maintaining device and the consolidometer is completely de-aired. Open the valve connecting the consolidometer base to a de-aired water source and fill the reservoir to the appropriate level. Apply the appropriate value of back pressure simultaneously to the top and bottom of the specimen for the appropriate length of time to ensure complete saturation of the specimen, or to ensure as near complete saturation of the specimen as practical. The back pressure shall be applied slowly to specimens having low initial degrees of saturation to minimize deformation of the specimen and prestressing.

9.4 If necessary (see 5.8), adjust the axial load-measuring device to compensate for the load produced by the back pressure, or record the axial load produced on the axial load-measuring device by the back pressure (no volume change), and subtract this value from all load readings.

9.5 If the axial loading device is set to maintain constant seating pressure, record the amount of consolidation or swell that occurs prior to controlled-strain loading. If the axial loading device is set to maintain constant specimen height, record the decrease or increase in axial load that occurs prior to controlled-strain loading.

9.6 Strain Rate Selection—It is desirable to select a strain rate that will cause the absolute value of the excess pore-water pressure to be between 3 and 30 % (See Note 6) of the applied vertical stress at any time during the test.

Note 6—To achieve this, it is good practice to target a maximum value of 20 % and in no case may the maximum value exceed 30 %. Guidelines for strain rate selection may be found in one or more of the references listed at the end of this method. The excess pore-water pressure may be limited to values less than 30 % for purposes of getting results consistent with incremental loading tests (Test Method D 2435).

9.7 Axial Loading—Close the valve connecting the specimen base to the back pressure-maintaining device and begin to apply the axial load so as to produce axial strain at the constant rate selected in 9.6. Record axial load, excess pore-water pressure, and time to failure as desired.
poor-water pressure, deformation, and elapsed time values  
at approximately 1-min intervals for the first 10 min, 5-min  
intervals for the next 1 h, and 15-min intervals thereafter.  
Take-sufficient readings to define the stress-strain curve;  
more frequent readings may be required when significi-  

tant changes in test parameters occur. Continuous recording  
of plotting, or both, may be used to obtain necessary data.  
Continue the loading until the desired stress or strain is  
obtained. When axial loading is complete, allow the excess  
poor-water pressure to dissipate at constant axial load or  
constant deformation and monitor axial load, deformation,  
and excess poor pressure.

9.8 Secondary Compression may be evaluated at any time  
during the test. To obtain secondary compression data,  
interrupt the controlled-strain axial loading at any pre-  
selected axial load and maintain the axial-load constant.  
Continue to record axial load, excess poor-water pressure,  
deforestation, and elapsed time as suggested in 9.7. In  
addition, record deformation and elapsed time at time  
intervals of 0.1, 0.25, 0.5, 1, 2, 4, 8, 15, and 30 min and  
1, 2, 4, 8, etc. h, measured from the time of interruption  
of controlled-strain loading. Readings shall continue at  
least until the slope of the characteristic linear secondary portion  
of the deformation versus log of time plot is apparent. If  
additional axial loading is required, resume the controlled-strain  
axial loading at the previous constant strain rate and record  
axial load, excess poor-water pressure, deformation, and  
elapsed time at the 1, 5, and 15-min intervals described in  
9.7. The procedure in this paragraph may be repeated  
at subsequent higher stress levels, when necessary.

Note 7—Interuption of the controlled-strain test to obtain  
secondary compression data under constant load may affect the  
void ratio-effective stress relationship. Further research is needed to define  
these effects.

9.9 Rebound—When rebound or unloading characteris-  
tics are desired, unload the specimen at a constant strain rate  
so that a positive total vertical stress is maintained. (See Note  
8.) The excess poor-water pressure will become negative.  
Back pressures must be sufficiently high or strain rates  
sufficiently low to maintain the pressure at the base of the  
specimen greater than atmospheric pressure. If the coeffi-  
cient of consolidation for the rebound portion is desired,  
back pressures must be sufficiently high or strain rates  
sufficiently low to maintain a greater pressure at the base of the  
specimen than the back pressure required for saturation.  
Record axial load, excess poor-water pressure, deformation,  
and elapsed time at the 1, 5, and 15-min intervals described in  
9.7. When rebound is complete, allow the excess poor-  
water pressure to dissipate at constant axial load or constant  
deforestation.

Note 8—Some have found that unloading strain rates on the  
order of one-tenth the loading strain rates are sufficient to  
maintain positive total vertical stress on the specimen and reason-  
able values of poor water pressures at the base.

10. Alternative loading, unloading, or reloading  
schedule may be employed that reproduces the construction  
stress changes, or obtains better definition of some part of the  
stress-strain (or stress-void-ratio) curve, or aids in inter- 
preting the field behavior of the soil. This shall be indicated  
clearly on the test results.

11. At the completion of the test, remove the entire  
specimen from the consolidometer, weigh, oven-dry, and  
reweigh to obtain the weight of solids.

10. Calculation

10.1 Calculate the initial void ratio, water content, unit  
weight, and degree of saturation, based on the dry weight of the  
total specimen. Specimen volume is computed from values  
measured in 7.3. Compute volume of solids by divid- 
ing the dry weight of specimen by the specific gravity of the  
solids. The volume of voids is assumed to be the difference  etween the specimen volume and the volume of the solids.

10.2 Calculate void ratio, $c$, (or alternatively, axial strain,  
$\epsilon$), total vertical stress, $\sigma_v$, and average effective vertical stress,  
$\sigma'_v$, for each set of values recorded in 9.7 through 9.9.

10.2.1 Calculate the void ratio as follows:

$$ c = c_0 - \frac{\Delta H}{H_0} $$

where:

- $c_0$ = initial void ratio,
- $\Delta H$ = deformation,
- $H_0$ = height of solids; volume of solids divided by the  
cross-sectional area of the specimen.

10.2.2 Calculate the axial strain as follows:

$$ \epsilon = \frac{\Delta H}{H_0} $$

where:

- $H_0$ = initial height of the specimen as measured in 7.3.

10.2.3 Calculate the applied axial stress as follows:

$$ \sigma_v = \frac{P}{A} $$

where:

- $P$ = applied axial load (see Note 9), and  
- $A$ = cross-sectional area of the specimen.

10.2.4 Calculate the average effective vertical stress (see  
3.4.8) as follows:

$$ \sigma'_v = (\epsilon - \epsilon_0)^2 \sigma_v $$

where:

- $\epsilon_0$ = excess poor-water pressure measured at the base  
of the specimen.

10.3 When the excess poor-water pressure measured at the  
base of the specimen exceeds 3 kPa (0.5 psi), calculate the  
coefficient of consolidation, $C_v$, for the interval between two  
sets of readings (see 3.4.7), recorded in 9.7 through 9.9, as follows:

$$ C_v = \frac{H^2 \log \left( 1 + \frac{\epsilon_v}{\epsilon_1} \right)}{2 \Delta \tau \log \left( 1 - \frac{\epsilon_0}{\epsilon_v} \right)} $$

where:

- $\epsilon_1$ = applied axial stress at time $t_1$,  
- $\epsilon_2$ = applied axial stress at time $t_2$,  
- $H$ = average specimen height between $t_1$ and $t_2$,  
- $\Delta \tau$ = elapsed time between $t_1$ and $t_2$ and $t_1$,  
- $\epsilon_0$ = average excess poor pressure between $t_1$ and $t_2$,  
- $\epsilon_v$ = average total applied axial stress between $t_1$ and $t_2$.  

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If strain rates are changed at any time during the test, the values of $C$, calculated at those times may be inaccurate.

Note 10—The above averages are obtained from one-half the sum of the two values.

10.3.1 It is best to compute $C$, between consecutive readings and assign the value of $C$, to the average value of $\sigma'$, between the two readings.

10.3.2 If the values of effective vertical stress do not change significantly between consecutive readings, the time interval may be increased.

11. Report

11.1 The report shall include the following information: including whether soil is undisturbed, remolded, compacted, or otherwise prepared,

11.1.2 Initial moisture content,

11.1.3 Initial wet unit weight,

11.1.4 Initial percent saturation,

11.1.5 If void ratio calculations are made, value of specific gravity of solids in the calculations,

11.1.6 Condition of test (value of back pressure, swell or consolidation during backpressure or settling pressure necessary to maintain constant height, strain rate(s) during loading and unloading),

REFERENCES


(3) Gorman, C. T., "Strain-Rate Selection in the Constant-Rate-of-Strain Consolidation Test," Research Report 556, Division of Research, Kentucky Department of Transportation, October 1980.


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