DEGRADATION OF RESILIENT MODULUS OF SATURATED CLAY DUE TO PORE WATER PRESSURE BUILDUP UNDER CYCLIC LOADING

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By
Jun Huang, B. S.

* * * * *

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Master’s Examination Committee:

Approved by

Dr. William E. Wolfe, Adviser

Dr. Tien H. Wu

Dr. Frank M. Croft

Adviser

Graduate Program in Civil Engineering
ABSTRACT

Laboratory testing was carried out to find out the reason why some pavement sections have larger deflections than what they were designed to. The soil specimens were made from compacted clay and fully saturated before they were subjected to cyclic loading which was to simulate traffic loading. Pore water pressure buildup and degradation of resilient modulus were observed during the cyclic loading. The buildup of pore water pressure caused the decrease of the effective confining stress, which, in turn, caused the degradation of resilient modulus. The degradation of resilient modulus and pore water pressure buildup were related to some boundary conditions, including loading period. Laboratory tests showed that longer relaxing time between two spikes of loading cycles resulted in lower pore water pressure buildup and less degradation of resilient modulus. In the collected data, there is fluctuation in peak pore water pressure when the soil was subjected to cyclic loading. The reasons are to be determined if more tests can be done.
Dedicated to my father, mother and girlfriend
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VITA

March 5, 1978........Born Fujian, P. R. China
1999...................B.S. Civil Engineering, Zhejiang University, Zhejiang, P. R. China

FIELD OF STUDY

Major Field: Civil Engineering
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CHAPTER 1
INTRODUCTION

Many pavements fail in less than their design life, some even shortly after being subjected to traffic loadings (ODOT, 1997, Internet source 31). In the United States, some billions of dollars are allocated for surface transportation programs and the maintenance of interstate highways annually. According to the summary of FY 2001 apportionments pursuant to the TEA-21 restoration act and the president’s FY 2001 budget proposal, more than five billion dollars have been requested for interstate maintenance and more than seven billion dollars will be spent on surface transportation program for fiscal year 2001 (TEA-21, 2000, Internet source 32). It is obviously not a minor problem when pavements fail prematurely. As part of an overall program to modernize highway design, and reduce the amount of money spent on the roadways repair and maintenance, the Federal Highway Administration (FHWA) developed the Strategic Highway Research Program (SHRP) in 1993 (AASHTO, 1997, Internet source, 30).

As part of its support for SHRP, the Ohio Department of Transportation, with the Federal Highway Administration, constructed a comprehensive test road consisting of four experiments (40 sections) in the Specific Pavement Studies (SPS). In this project, some important pavement response parameters were measured, including strain,
displacement, pressure and joint opening. The purpose of this project was to monitor the performance of specific pavement designs under traffic loading, and through these studies, to find out the influence of moisture contents, loading period, and deviator stress levels on the stiffness of subgrade soils.

According to some previous research, for example, Seed 1962, Elliott, 1992, as long as the loading is not close to the shear strength of subgrade soil, the deflection of pavement structure largely depends on the recoverable deformation of subgrade soil, which is, in turn, dependent on the subgrade soil stiffness. The stiffness of the subgrade soil can be represented by the resilient modulus, $M_r$ (Seed, 1962). The resilient modulus represents the elastic behavior of the soil and the detailed definition of resilient modulus is given in Chapter 2. Thus, the resilient modulus of the subgrade soil is an important parameter of the pavement design for the accurate calculation of pavement deflection. The American Association of State Highway and Transportation Officials AASHTO Guide for Design of Pavement Structures 1986 adopted the resilient modulus as the subgrade soil property controlling the design of flexible pavements.

There are a lot of factors influencing the value of the resilient modulus, including moisture content, loading magnitude, confining stress, number of loading cycles. Many researchers, including Mohammad et al 1996 and Elliot 1992, have pointed out that high moisture content could lead to reduction in the resilient modulus of subgrade soil, especially in clay soils. With seasonal and environmental changes, the underground water level and the precipitation will change too, which, in turn, will change the moisture content of subgrade soil. Therefore, AASHTO incorporated seasonal effects on the resilient modulus, and considered the reduction of resilient modulus due to the increase of
moisture content (AASHTO Guide for Design of Pavement Structures, 1993). The “effective resilient modulus” is the weighted mean value of different resilient moduli obtained at different moisture contents.

However, this methodology doesn’t seem conservative according to the field data observation from the SPS studies. Instrumented sections, at which pore water pressure transducers were installed, showed high water pressures when the pavement was subjected to continuous loading sequences. This pore-water pressure build-up was thought to be the result of the dynamic nature of the load and the inability of the pressure to dissipate before the load is repeated. The build-up of pore water pressure reduces the confining stress, and thus, the stiffness of the subgrade soil. Therefore, the deformation of subgrade soil will be larger and the deflection of a pavement structure will increase accordingly.

According to some researchers, capillary effects in unsaturated soil tend to draw water into the soil increasing the degree of saturation (see Fredlund and Hernandez, 1993 for a detailed review). Most subgrade soils are partially saturated in the construction because they are compacted at or close to optimum moisture content. If the subgrade soils get fully saturated within a short period of time, it is possible that, due to the buildup of dynamic pore water pressure, the reduction in the resilient modulus of subgrade soil leads to excessive deflections of pavement layers. Therefore, we need to recalculate the resilient modulus considering the reduction caused by pore water buildup as well as the variation of moisture contents in order to correctly calculate the deflection of pavement layer.
Because static testing procedures are not adequate for characterizing the behavior of soils subjected to the impulse type repeated loading representative of moving wheel loads, data from repeated loading test are important inputs for flexible pavement analysis and design. The laboratory testing was designed based on the AASHTO Designation T 294-94 to reveal the relationship between the resilient modulus and buildup of pore water pressure under the simulated cyclic loading. The effects of loading amplitude, loading frequency, number of loading cycles were also investigated in the laboratory testing.

Chapter 2 will present a review of the pertinent literature on the resilient behavior of soils in laboratory testing. In Chapter 3, the laboratory program followed to get necessary soil parameters and investigate the problems is presented. Chapter 4 will include the set-up and procedure of the modified resilient modulus test program and Chapter 5 will present the testing results and discussion of those results. Finally, Chapter 6 will summarize the findings of my research and make recommendations for modifying current practice with those findings.
CHAPTER 2

DEFINITION AND PREVIOUS INVESTIGATIONS

2.1 Introduction

The possibility of extensive cracking of pavements resulting from excessive recoverable deformations of the subgrade soils without significant plastic deformation has been well recognized for some time. Seed, in 1962, pointed out the modulus of resilient deformation was the key factor in determining the deflection of pavements. Many researchers analyzed different factors influencing the value of resilient modulus of subgrade soils, including Drumm et al. (1997), Elliott (1992), and Mohammad et al. (1994). Brown et al. (1975) identified several factors influencing the resilient modulus of cohesive soils. The major factors included number of loading cycles, deviator stresses, and overconsolidation ratio. Brodsky (1989) studied the effects of deviator stresses and confining pressure on the resilient modulus of cohesive soils. He found that the $M_r$ of clay increased with the increase of confining stress or decrease of deviator stress. In order to standardize the design of highway pavements, ASSHTO proposed T274-82, T292-91 and T294-94 as the testing procedures for measurement $M_r$ of cohesionless or cohesive soils.
used in the design of AASHTO Guide for Design of Pavement Structures 1986. In the design guides, fixed deviator stress, confining stress, loading frequency, loading cycles are specified to measure the resilient modulus of reconstituted or intact soil samples.

Partially saturated soil has negative pore water pressure and capillary suction will change the moisture content of soil. Researchers subsequent to the adoption of the standard design codes in 1986 noticed that soils with different moisture contents show significant differences in resilient modulus under cyclic loadings (Elliot et al. 1992, Elfino et al. 1989). AASHTO modified the design guide (1993) to accommodate these research results by using “effective resilient modulus” to incorporate resilient modulus reduction due to increase in moisture content. If the soil finally gets saturated, the pore water pressure will build up due to the low permeability of the clay resulting in a reduction in effective confining stress and possibly the resilient modulus. Some researchers did observe pore water pressure build-up in cohesive soil under dynamic loading (Ansai et al. 1989, Mendoza et al. 1994, Ogawa et al. 1977). They found out the pore water pressure buildup was related to the deviator stress levels and the number of loading cycles. Unfortunately, none of the above researchers commented on the relationship of buildup of pore water pressure and reduction of resilient modulus.

2.2 Definition of \( M_r \)

Resilient modulus is usually described as the unloaded phase of the stress-strain curve during wheel loading when vehicles pass over the pavement.
The definition of resilient modulus is the ratio of deviator stress and resilient (or recoverable strain), given in the following equation, according to the AASHTO DESIGNATION: T 294-94:

\[ M_r = \frac{\sigma_d}{e_r} \quad (2.1) \]

Where \( M_r \) is resilient modulus,

\[ \sigma_d = \bar{\sigma}_1 - \bar{\sigma}_3 \] is the repeated axial deviator stress, and is the difference between the major and minor principal stresses in a triaxial test,

\[ \bar{\sigma}_1 \] is the effective axial stress (major principal stress),

\[ \bar{\sigma}_3 \] is the effective radial stress (minor principal stress),

\( e_r \) is the resilient (recovered, or elastic) axial strain due to \( \sigma_d \),

\( e_1 \) is the total axial deformation due to \( \sigma_d \).

Figure 2.1 shows the resilient modulus graphically.
2.3 Behavior of Unsaturated Soil

2.3.1 Measurement of Mr using laboratory dynamic tests

In the design of the pavement layer, the proper choice for the resilient modulus of the subgrade soil is important to calculate the displacement of pavement sections correctly. Therefore, the method to evaluate the resilient modulus of soil specimens under dynamic loading was introduced in AASHTO Designation T 294-94. In T 294-94, loading frequency and loading cycles were specified in the testing designation. Barksdale
(1971) pointed out that vehicle speed and depth beneath the pavement surface are of great importance in selecting the appropriate axial compressive stress pulse time to use in dynamic loading testing. Barksdale found that a pulse time of 0.03 to 0.05 sec was appropriate for vehicle speeds of 50 to 60 mph. In order to standardize the design process, AASHTO Designation T294-94 chose loading frequency to be one cycle per second consisting 0.1-sec loading period and 0.9-sec relaxing period.

2.3.2 Influence of moisture content on $M_r$

Thompson (1989) studied factors affecting the resilient modulus of soils. He found the resilient modulus would decrease with an increase in moisture content. Elliot (1992) pointed out there was no single resilient modulus for a soil, rather, the resilient modulus depends on a number of factors, principally moisture content, confining and deviator stresses. He also noticed that resilient modulus decreased with an increase in moisture content or degree of saturation.
End system: LVDT was placed outside of the triaxial chamber
Middle system: LVDT was clamped to the soil system

Figure 2.2 Influence of moisture content on resilient modulus test results of silty clay
(From Mohammad, et al., 1996)
Elliot and Mohammad, et al. (1996) had similar results on the trend of variation of resilient modulus vs. moisture content. Figure 2.2 shows Mohammad's results. Drumm et al. (1997) demonstrated a decrease in resilient modulus with an increase of moisture content. They also pointed out that soils with the highest resilient modulus at optimum moisture content and maximum dry density were most susceptible to changes in resilient modulus because of changes of degree of saturation. Their results showed that the relationship between resilient modulus and degree of saturation was linear. However, in their experiments, none of the soil samples was fully saturated. The highest degree of saturation of any sample was 97.3%. None of the above researchers used fully saturated soil samples in their experiments to demonstrate the relationship between the degradation of resilient modulus and the buildup of pore water pressure buildup.

2.3.3 Loading frequency

The effects of different frequencies ranging from 0.01 to 10 Hz on the pore water pressure buildup were negligible according to Brown et al. (1975). Seim (1989) had similar conclusion in his comprehensive research. He indicated that changes in Mr due to varying the pulse duration between 0.1 and 1.0 sec are small for cohesive soils. He also referred to many other resources whose testing had drawn similar conclusions for a variety of soil types. Ansal et al. (1989) indicated that frequencies of cyclic loading had little influence on the buildup of pore water pressure at a large number of cycles of loading.
2.3.4 Influence of confining and deviator stresses on resilient modulus

Mohammad, et al (1996) showed the resilient modulus increased as the confining pressure increased. They explained the reason was stiffness increased at higher confining pressures. An increase in the deviator stress resulted in a reduction of resilient modulus of clay, which was attributed to a reduction in effective confining stress resulting from increased positive pore water pressure development. Since the permeability of clay is quite low, the pore water pressure cannot completely dissipate during dynamic loading, thus reducing the effective confining stress. However, Mohammad, et al. didn’t prove this point with experiments. Mohammad, et al (1994) showed the resilient modulus of sands increased with either the increase in confining stresses or an increase in deviator stresses apparently because the high permeability of sands allows for the fast dissipation of pore pressure even during dynamic loading, thus the effective bulk stress will increase with deviator stress. Drum, et al (1997) presented similar results except that the resilient modulus of fine-grained soils increased only slightly with increase of confining pressure. It is possible that, because of low permeability of fine-grained soils, the buildup of pore water pressure may counteract the effects of increase of confining pressure.

2.3.5 Water Suction in Partially Saturated Clay

In partially saturated soil, the pores act as capillary tubes that cause the soil water to rise above the underground water table into soil pores. According to the review by Fredlund (1993), matric suction, which is the difference between the reference air pressure and pore water pressure in a partially saturated soil, accounts for the capillary phenomenon in partially saturated soils. Higher matric suction means greater negative
pressure difference between soil pore water pressure and reference air pressure. More water will be attracted into subgrade soil within a period of time, which means subgrade soil may get fully saturated in a short time.

2.4 Behavior of Saturated Soil

2.4.1 Saturating clay with backpressure

Completely saturated soil has no air voids, which means the pore water will sustain all variation of total confining pressure if water is almost incompressible compared to the soil particles. In order to make sure the soil is fully saturated, one easy and accurate way is to measure the pore water pressure response with the variation of confining stress. Skempton (1954) introduced the pore pressure coefficient $B$. According to Skempton, the definition of $B$ is

$$B = \frac{\Delta u}{\Delta \sigma_c}$$  \hspace{1cm} (2.2)

Where $\Delta \sigma_c$ is the variation of confining pressure, and $\Delta u$ is the corresponding variation of pore water pressure inside the soil sample. $B = 1$ means full saturation.

Because of the low permeability of clay and compaction, it takes a long time to fully saturate compacted clay. Some original ways came out to speed up this process. The following research done by Black and Lee (1973) is to saturate soil with high backpressure. The procedure of using an elevated back pressure to produce complete saturation of soil samples is quite efficient as long as a high enough back pressure is provided to ensure almost all the air bubbles will dissolve into water. According to the above researchers, the saturation problem includes two major problems: permeability
problem and diffusion problem. To solve the permeability problem, a large water head difference between top and bottom of soil sample should be maintained in order to let water flush through the sample. And to solve the diffusion problem, de-aired water and high backpressure should be used to ensure more air bubbles could be dissolved into surrounding pore water. Black and Lee also showed a way to calculate how long it would take to completely saturate a soil sample at different backpressure level, which could be used to calculate the preferred backpressure in this research.

**2.4.2 Pore Water Pressure Build-up in Clay under Dynamic Loading**

In the previous investigations of measurement of pore water pressure buildup in clay under dynamic loading, different researchers had different observations. Brown et al. (1975) found out that there existed a threshold stress ratio. If the ratio of deviator stress to the undrained shear strength is less than the threshold stress ratio, the applied cyclic shear stresses did not cause the sharp build-up of pore water pressure under cyclic loading. These results coincided with those of Ansal and Erken (1989) (Figure 2.3) and Medndoza and Hernandez (1994), but were contradictory to those of Ogawa et al. (1977).
Figure 2.3 Cyclic Stress Ratio-Pore Pressure Relationship for Different Number of Cycles (From Ansal, A. M., Erken, A., 1989)

Ogawa et al. (1977) had a totally different observation, which was the pore pressure increased with an increase in the number of loading cycles no mater how small the deviator stresses were. Ansal et al. (1989) observed the buildup of pore water pressure in clay under cyclic loading too. They investigated the effects of frequencies, amplitudes, and numbers of cycles of dynamic loading on pore water pressure buildup. They found the existence of threshold deviator stress level like Brown et al. (1975). The required number of cycles to create high pore water pressure would decrease with an increase in the deviator stress. Anderson et al. (1990) had similar observation in their comprehensive
experiments. They showed that different amplitudes of deviator stress created different levels of residual pore water pressure under dynamic loading. Mendoza and Hernandez (1994) observed that pore water pressure increased almost linearly with the increase of deviator stress up to a certain threshold shear stress and thereafter, pore water pressure grew much faster.

2.4.3 Reduction of $M_r$ occurred after some loading cycles

In their experiments, Brown et al. (1975) also observed the reduction of resilient modulus with the increase of loading cycles. However, a reasonably constant value was

![Graph showing the variation of resilient modulus with cycles](image)

Figure 2.4 Typical variation of resilient modulus during repeated load tests (From Brown, et al., 1975)
achieved after some $10^5$ cycles (Figure 2.4). The stable resilient modulus was a function of confining stress as well as the cyclic deviator stress.

The researchers didn't discuss the relationship between the pore water pressure build-up and the reduction of resilient modulus furthermore. Anderson et al. (1990) came to similar conclusions from their research on the reduction of resilient modulus under cyclic loading. The reduction of Secant Deformation Moduli occurred after some loading cycles. They used secant deformation moduli instead of resilient modulus. Basically, both moduli are quite similar to each other in reflecting stiffness of soil.

2.5 Conclusion

Behavior of saturated clays is different from that of unsaturated clays. For example, different frequencies of loading cycles have few effects on the measurement of the resilient modulus of unsaturated clay (Seim, 1989), while loading cycles of different frequencies create different pore water pressure buildup in saturated soil samples at first few loading cycles (Ansal et al., 1989). Many researchers observed pore water pressure buildup, although some researchers came to different conclusions on the concept of a threshold stress ratio (Ansal and Erken, 1989, Ogawa et al. 1977, Brown et al. 1975). Some researchers carried out continuous cyclic loading experiments on clay samples and observed the reduction of resilient modulus after some large loading cycle numbers, as well as the buildup of pore water pressure (Ansal and Erken 1989). None of the above researchers related the buildup of pore water pressure to the reduction of resilient modulus. Mohammad et al. (1994) speculated that the buildup of pore water pressure
might reduce the resilient modulus of fine-grained soils. However, he didn't show this relationship by experiments.
CHAPTER 3

SOIL PROPERTIES LAB TESTS AND STANDARD M₅ TEST SETUP

3.1 Introduction

Basic soil properties include soil particle distribution, soil texture, particle shape, and moisture content. According to AASHTO Designation T 294-94, different values for loading deviator stress and confining stress, as well as sample preparation methods are specified for different soil groups. Therefore, knowledge of basic soil properties are important for the determination of resilient modulus test methods.

Maximum dry density and optimum moisture content are also important for the preparation of soil specimens in the standard resilient modulus tests. In order to simulate the field situation, it is important to compact soil specimens at optimum moisture content and maximum dry density, which is usually how the subgrade soil is compacted in the field.

As mentioned in Chapter 2, the resilient modulus of unsaturated soil has been shown to be independent of loading patterns and loading frequency. In order to
standardize the procedure of determining resilient modulus of unsaturated soil, AASHTO Designation T 294-94 specified fixed loading frequency and loading pattern for the testing. However, some factors, like loading frequency, which may not be important in the determination of the resilient modulus of unsaturated soil, may become influential during modified resilient modulus tests of saturated soil specimens. Based on this hypothesis, modified resilient modulus testing was carried out to reveal the relationship between pore water pressure buildup and degradation of resilient modulus, and the influence of loading frequency on the degradation of resilient modulus of saturated soil.

3.2 Engineering Indices of Soil Samples

Soils may be classified by using basic laboratory tests as specified by the AASHTO or ASTM standards. In this thesis, AASHTO standards will govern the classification if the differences between AASHTO and ASTM exist. Engineering indices of soil determine the soil types and the engineering properties.

There are two major soil classification systems: AASHTO Soil Classification System and Unified Soil Classification System (USCS). The basis for the USCS is that coarse-grained soils can be classified according to their grain size distributions, whereas the engineering behavior of fine-grained soils is primarily related to their plasticity. Therefore, only a sieve analysis and the Atterberg Limits are necessary to completely classify a soil in this system. The fine-grained soils, having more than 50% passing the No. 200 sieve, are subdivided into silts (M) and clays (C) based on their liquid limit and plasticity index. According to Casagrande’s plasticity chart (Figure 3.1), the fine-grained soils can be subdivided into different soils types.
Figure 3.1 Casagrande’s Plasticity Chart, showing several representative soil types (Modified from “An Introduction to Geotechnical Engineering”, 1981)

AASHTO Designation M 145-91 describes a procedure for classifying soils into seven groups based on laboratory determination of particle-size distribution, liquid limit, and plasticity index. The granular materials, having fewer than 35% particles passing No. 200 sieve, are subdivided into A-1, A-2, A-2 types of soils using particle distribution and Atterberg limits. The silt-clay materials, having more than 35% particles passing No. 200 sieve, are subdivided into A-4, A-5, A-6, A-7 types of soils using only Atterberg limits (Table 3.1).
<table>
<thead>
<tr>
<th>General Information</th>
<th>Granular Materials (≤35% passing No. 200)</th>
<th>Silt-Clay Materials (&gt;35% passing No. 200)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A-1</td>
<td>A-2</td>
</tr>
<tr>
<td>Sieve analysis, percent passing:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.00 mm (No. 10)</td>
<td>≤50</td>
<td>--</td>
</tr>
<tr>
<td>0.425 mm (No. 40)</td>
<td>≤30</td>
<td>≤50</td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
<td>≤15</td>
<td>≤25</td>
</tr>
<tr>
<td>Characteristics of fraction passing 0.425 mm (No. 40)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Liquid Limit Plasticity Index</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>≤6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Usual types of significant constituent materials</td>
<td>Stone Fragments, gravel and sand</td>
<td>Fine sand</td>
</tr>
<tr>
<td>General ratings as subgrade</td>
<td>Excellent to Good</td>
<td></td>
</tr>
</tbody>
</table>

Table 3.1 Classification of Soils and Soil-Aggregate Mixtures (AASHTO Designation: M 145-91 (1995))
Most soils in Ohio are classified as either A-2, A-4, A-6, or A-7 in the AASHTO classification system. The soil samples used in the tests performed in this work were A6 soils retrieved from the upper soil layer of the subgrade soil at the construction site of US 23 in Delaware County, Ohio. A-6 soils have been widely used in the construction of highway subgrades. However, A-6 soils contain significant fraction of clay particles, therefore the permeability is very low, for example, $10^{-7} \sim 10^{-8}$ cm/s (Robert and William, 1981). The in-situ moisture content obtained from the lab tests was 10.7% (Table A. 1), which indicated the subgrade soil was compacted on the “dry” side of the optimum moisture content. The basic soil property lab tests are discussed in detail in the following sections.

3.3 Laboratory tests

As discussed in the previous section, two basic soil property tests, Atterberg limits, and particle size analysis (sieve analysis and hydrometer tests), were required to classify the US 23 soil. Atterberg limits tests were carried out to obtain the liquid and plastic limits. Particle size analysis was to obtain particle distribution curve of the sample. These results were necessary to classify the soil. In addition, Standard Proctor tests and unconfined compression tests were carried out. The Standard Proctor testing was performed to obtain the maximum dry density and optimum moisture content. Unconfined compression testing was to get the stress-strain curve and maximum unconfined compressive strength.
3.3.1 Atterberg Limits Test

According to AASHTO designation T 89-81, T 90-81, and ASTM D 4318-95a, liquid limit (LL) and plastic limit (PL) and plasticity index (PI) were obtained.

The Liquid Limit (LL) of this soil sample was 38 and Plastic Limit (PL) was 21, then the Plasticity Index (PI) was 17. The difference between two plastic limit tests was 1.4% of the mean value, which was within the range of 2.6% provided by ASTM D 4318-95a.

![Graph of Atterberg Limits Test](image)

Figure 3.2 Atterberg Limits
The following is the Atterberg limits for this soil sample (refer to Table 3.2 and Figure 3.2):

<table>
<thead>
<tr>
<th>Liquid Limit (LL)</th>
<th>Plastic Limit (PL)</th>
<th>Plasticity Index (PI)</th>
</tr>
</thead>
<tbody>
<tr>
<td>38</td>
<td>21</td>
<td>17</td>
</tr>
</tbody>
</table>

Table 3.2 Atterberg Limits of soil sample

3.3.2 Sieve Analysis and Hydrometer Tests

The soil sample used in sieve analysis test satisfied the minimum requirements of AASHTO T 88-81 designation and ASTM D 422-63 designation. Test results are listed in Table 3.3. Since Specific Gravity is required for Hygrometer tests, Specific Gravity testing was performed. The specific gravity of the soil sample was 2.75 (Table 3.4).

According to AASHTO T88-81 and ASTM D422-63, hydrometer test was carried out on a soil sample, which passes No. 200 (0.075-mm) sieve to get the size distribution curve of fine fraction in the soil sample. The test results are listed in Table 3.5. The particle distribution curve is in Figure 3.3.
### SIEVE ANALYSIS

Location: B3 5+50 (278+00)  
Description: BE03 embankment  
Sample  
Depth:  
Sample No.: 390101  
Date: 03/01/2000  
Tested by: Jun Huang

Total Soil Weight (g): 999.03

<table>
<thead>
<tr>
<th>Sieve No.</th>
<th>Sieve Opening (mm)</th>
<th>Wt. Sieve (g)</th>
<th>Wt. Sieve + Soil (g)</th>
<th>Wt. Soil Retained (g)</th>
<th>Percent Retained (%)</th>
<th>Accumulative Percent Retained (%)</th>
<th>Percent Finer</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>0.075</td>
<td>303.81</td>
<td>412.81</td>
<td>109.00</td>
<td>11%</td>
<td>17%</td>
<td>83%</td>
</tr>
<tr>
<td>40</td>
<td>0.42</td>
<td>394.89</td>
<td>424.78</td>
<td>29.89</td>
<td>3%</td>
<td>6%</td>
<td>94%</td>
</tr>
<tr>
<td>14</td>
<td>1.41</td>
<td>583.28</td>
<td>591.06</td>
<td>7.78</td>
<td>1%</td>
<td>3%</td>
<td>97%</td>
</tr>
<tr>
<td>8</td>
<td>2.38</td>
<td>550.00</td>
<td>560.47</td>
<td>10.47</td>
<td>1%</td>
<td>2%</td>
<td>98%</td>
</tr>
<tr>
<td>4</td>
<td>4.76</td>
<td>513.30</td>
<td>523.10</td>
<td>9.80</td>
<td>1%</td>
<td>1%</td>
<td>99%</td>
</tr>
<tr>
<td>3/8 in.</td>
<td>9.52</td>
<td>795.03</td>
<td>796.71</td>
<td>1.68</td>
<td>0%</td>
<td>0%</td>
<td>100%</td>
</tr>
<tr>
<td>1 in.</td>
<td>25.4</td>
<td>795.03</td>
<td>795.03</td>
<td>0.00</td>
<td>0%</td>
<td>0%</td>
<td>100%</td>
</tr>
<tr>
<td>2 in.</td>
<td>50.8</td>
<td>795.03</td>
<td>795.03</td>
<td>0.00</td>
<td>0%</td>
<td>0%</td>
<td>100%</td>
</tr>
<tr>
<td>3 in.</td>
<td>76.2</td>
<td>795.03</td>
<td>795.03</td>
<td>0.00</td>
<td>0%</td>
<td>0%</td>
<td>100%</td>
</tr>
</tbody>
</table>

Table 3.3 Soil Particle Distributions by Sieve Analysis
Specific Gravity Test

Location: B3 5+50 (278+00)  
Date: 03/02/2000  
Description: BE03 embankment  
Tested by: Jun Huang  
Depth:  
Sample No.: 390101  

Volume of flask at 20°C: 500 ml

<table>
<thead>
<tr>
<th>Test No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature (°C)</td>
<td>23</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>Mass of flask + water, W₁ (g)</td>
<td>689.44</td>
<td>661.43</td>
<td>667.50</td>
</tr>
<tr>
<td>Mass of flask + water + soil, W₂ (g)</td>
<td>761.86</td>
<td>730.18</td>
<td>736.20</td>
</tr>
<tr>
<td>Mass of dish (g)</td>
<td>203.81</td>
<td>190.03</td>
<td>164.87</td>
</tr>
<tr>
<td>Mass of dish + dry soil (g)</td>
<td>317.05</td>
<td>298.20</td>
<td>272.92</td>
</tr>
<tr>
<td>Mass of dry soil, W₄ (g)</td>
<td>113.24</td>
<td>108.17</td>
<td>108.05</td>
</tr>
<tr>
<td>Mass of equal volume of water, W₆ (g) = (W₁ + W₄) - W₂</td>
<td>40.82</td>
<td>39.42</td>
<td>39.35</td>
</tr>
<tr>
<td>Gₛ(T°C) = W₆/W₆</td>
<td>2.77</td>
<td>2.74</td>
<td>2.75</td>
</tr>
<tr>
<td>K (correction factor)</td>
<td>0.99930</td>
<td>0.99930</td>
<td>0.99930</td>
</tr>
<tr>
<td>Gₛ(20°C) = Gₛ(T°C)*K</td>
<td>2.77</td>
<td>2.74</td>
<td>2.74</td>
</tr>
</tbody>
</table>

Average Gₛ: 2.75

Table 3.4 Specific Gravity Results
Hydrometer (ASTM 152H) Analysis

Location: B3 5+50 (278+00)
Description: BE03 embankment
Sample No.: 390101

Dry weight of soil, \( W_s \): 50g
\( G_s \): 2.75

Hydrometer type: ASTM 152H
Meniscus correction, \( F_m \): 1
Zero correction, \( F_z \): 5
Correction factor, \( a \): 0.98

<table>
<thead>
<tr>
<th>Elapsed Time (min)</th>
<th>Temperature (°C)</th>
<th>Hydro-meter Reading, ( R )</th>
<th>( R_{cp} ) (( R + F_T - F_Z ))</th>
<th>Percent finer(%) ((a*R_{cp}*100)/50)</th>
<th>( R_{ct.} ) (( R + F_m ))</th>
<th>L (cm)</th>
<th>A</th>
<th>D (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>21.0</td>
<td>48.0</td>
<td>43.20</td>
<td>84.54</td>
<td>49.0</td>
<td>8.30</td>
<td>0.01309</td>
<td>0.02667</td>
</tr>
<tr>
<td>5</td>
<td>21.0</td>
<td>43.0</td>
<td>38.20</td>
<td>74.75</td>
<td>44.0</td>
<td>9.10</td>
<td>0.01309</td>
<td>0.01766</td>
</tr>
<tr>
<td>15</td>
<td>21.0</td>
<td>34.0</td>
<td>29.20</td>
<td>57.14</td>
<td>35.0</td>
<td>10.60</td>
<td>0.01309</td>
<td>0.01100</td>
</tr>
<tr>
<td>30</td>
<td>21.0</td>
<td>31.0</td>
<td>26.20</td>
<td>51.27</td>
<td>32.0</td>
<td>11.10</td>
<td>0.01309</td>
<td>0.00796</td>
</tr>
<tr>
<td>60</td>
<td>21.0</td>
<td>25.0</td>
<td>20.20</td>
<td>39.53</td>
<td>26.0</td>
<td>12.00</td>
<td>0.01309</td>
<td>0.00585</td>
</tr>
<tr>
<td>250</td>
<td>22.0</td>
<td>19.0</td>
<td>14.40</td>
<td>28.18</td>
<td>20.0</td>
<td>13.00</td>
<td>0.01294</td>
<td>0.00295</td>
</tr>
<tr>
<td>480</td>
<td>22.0</td>
<td>13.0</td>
<td>8.40</td>
<td>16.44</td>
<td>14.0</td>
<td>14.00</td>
<td>0.01294</td>
<td>0.00221</td>
</tr>
<tr>
<td>1440</td>
<td>20.0</td>
<td>12.0</td>
<td>7.00</td>
<td>13.70</td>
<td>13.0</td>
<td>14.20</td>
<td>0.01325</td>
<td>0.00132</td>
</tr>
</tbody>
</table>

Table 3.5 Soil Particle Distributions by Hydrometer Testing

28
Figure 3.3 Soil Particle Distribution Curve
3.3.3 Soil classification

With the USCS, the results are as shown in Table 3.6. Therefore, the soil sample is CL (Refer to Figure 3.1). According to AASHTO classification system, the results are as Table 3.7. Therefore, the soil sample belongs to A-6 soil type (Refer to Table 3.1).

<table>
<thead>
<tr>
<th>Percentage of particles passing No. 200 Sieve</th>
<th>Plasticity Index</th>
<th>Liquid Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>83%</td>
<td>17</td>
<td>38</td>
</tr>
</tbody>
</table>

Table 3.6 Soil classification with USCS

<table>
<thead>
<tr>
<th>Percentage Passing</th>
<th>Properties of fraction passing No.40</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 10</td>
<td>No. 40</td>
</tr>
<tr>
<td>97%</td>
<td>94%</td>
</tr>
<tr>
<td>No. 200</td>
<td>LL</td>
</tr>
<tr>
<td>83%</td>
<td>38</td>
</tr>
<tr>
<td></td>
<td>PI</td>
</tr>
<tr>
<td></td>
<td>17</td>
</tr>
</tbody>
</table>

Table 3.7 Sieve Analysis according to AASHTO classification
3.3.4 Proctor Compaction Test

According to AASHTO T 99-81, a Proctor compaction test was carried out to get the optimum moisture content and maximum dry density of the soil sample. From the test results, the maximum dry density and optimum moisture content are 17.78 (kN/m³) and 16.6%, respectively. Table A. 2 and Figure A. 1 are the results of Proctor compaction tests.

The Ohio Department of Transportation (ODOT) proposed the minimum compaction requirements, as shown in Table 3.8, for the subgrade soil of all roads constructed in Ohio. Based on the maximum dry density, a different degree of compaction is required.

<table>
<thead>
<tr>
<th>Maximum lab dry density kg/m³</th>
<th>1440-1680 (90-104.9)</th>
<th>1681-1920 (105-119.9)</th>
<th>&gt;1921 (&gt;120)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum compaction percent lab maximum %</td>
<td>102</td>
<td>100</td>
<td>98</td>
</tr>
</tbody>
</table>

Table 3.8 Embankment soil compaction requirements recommended by ODOT
3.3.5 Unconfined Compression Test

Knowledge of the unconfined compressive strength was important prior to conducting the modified resilient modulus tests, since the deviator stress in modified resilient modulus tests must be kept below the soil strength and an increase in moisture content will likely reduce the unconfined compressive strength of the same soil.

The tests were carried out according to AASHTO T 208-70. Six soil samples were tested using a strain-controlled compression machine, three immediately after compaction and three after saturation. Test results are listed in Figure A. 2 and Figure A. 3.

3.3.6 Soil Property Determinations

Table 3.9 shows the soil property determination by different testing designations.

<table>
<thead>
<tr>
<th>Property</th>
<th>Values</th>
<th>Test Procedure</th>
</tr>
</thead>
<tbody>
<tr>
<td>LL, as a percentage</td>
<td>38</td>
<td>AASHTO T 89</td>
</tr>
<tr>
<td>PI, as a percentage</td>
<td>17</td>
<td>AASHTO T 90</td>
</tr>
<tr>
<td>Grain size analysis</td>
<td>---</td>
<td>AASHTO T 88</td>
</tr>
<tr>
<td>Maximum dry density</td>
<td>17.78 (kN/m³)</td>
<td>AASHTO T 99</td>
</tr>
<tr>
<td>OMC</td>
<td>16.6%</td>
<td>AASHTO T 99</td>
</tr>
</tbody>
</table>

Table 3.9 Soil Property Determinations
3.3.7 Saturation of soil samples

Saturation was achieved by applying a backpressure of 33-psi. The cell pressure was 36-psi, for an effective confining pressure of 3-psi, which was almost the same effective pressure of the subsoil which was subjected to at the depth of 2 feet below the pavement.

The soil specimen was extruded from the steel cylindrical compaction mold and put into to triaxial chamber for saturation. The cell pressure, backpressure at the bottom and backpressure at the top are imposed sequentially. Table 3.10 lists the applied pressures.

<table>
<thead>
<tr>
<th>Cell Pressure (psi)</th>
<th>Backpressure at the bottom (psi)</th>
<th>Backpressure at the top (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>33.0</td>
<td>32.0</td>
<td>30.0</td>
</tr>
</tbody>
</table>

Table 3.10 Pressure used in the saturation process

Saturation was determined by measuring the B-value (Black and Lee, 1973). Before the process of saturation with backpressure, a calibrated pore water pressure transducer was connected to the triaxial chamber (Figure 3.4).
3.4 Standard Resilient Modulus Test Setup

3.4.1. Resilient Modulus Test Equipment

According to AASHTO designation T294-94, the suggested resilient modulus tests generally consist of a servo-hydraulic material testing system, triaxial cell, computer, load cell of a certain capacity, linear variable differential transformer (LVDT),
and data acquisition and analysis system. Figure 3.5 shows a possible schematic of testing apparatus. Figure 3.6 shows one suggested triaxial chamber with LVDT and load cell for dynamic loading tests.

Figure 3.5 Schematic of Testing Apparatus (from Mohammad et al., 1994)
Figure 3.6 Triaxial Chamber with External LVDT’s and Internal Load Cell (from AASHTO Designation: T294-94)
3.4.2 Soil Samples Preparation and Standard Resilient Modulus Testing

The T294-94 testing designation divides all soils into two types: Material Type 1 and Material 2. Different types of soil have different compaction methods, different mold sizes, different resilient modulus testing methods, etc. Many soils used for subgrades in Ohio are fine-grained soils according to AASHTO soil classification, therefore this study focuses on properties of fine-grained soils, that is, generally, Material Type 2. According to T 294-94, the general method of compaction of Type 2 soils will be static loading (also known as the double plunger method).

As indicated in testing designation, the first step of tests is to condition soil specimens by applying 1000 cycles of a deviator stress of 28 kPa (4 psi) using a haversine shaped load pulse consisting of a 0.1-second load followed by a 0.9-second rest period. Conditioning is very important because it could minimize the effects of initially imperfect contact between samples and platens. Table 3.11 lists the specific loading sequence. A minimum load of 10% of the maximum applied load should be maintained throughout the all the loading sequence in order to eliminate the gaps between soil samples and platens.
<table>
<thead>
<tr>
<th>Sequence No.</th>
<th>Confining Pressure $S_3$ (kPa)</th>
<th>Deviator Stress $S_d$ (kPa)</th>
<th>Number of Load Applications</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>41</td>
<td>28</td>
<td>1000</td>
</tr>
<tr>
<td>1</td>
<td>41</td>
<td>14</td>
<td>100</td>
</tr>
<tr>
<td>2</td>
<td>41</td>
<td>28</td>
<td>100</td>
</tr>
<tr>
<td>3</td>
<td>41</td>
<td>41</td>
<td>100</td>
</tr>
<tr>
<td>4</td>
<td>41</td>
<td>55</td>
<td>100</td>
</tr>
<tr>
<td>5</td>
<td>41</td>
<td>69</td>
<td>100</td>
</tr>
<tr>
<td>6</td>
<td>21</td>
<td>14</td>
<td>100</td>
</tr>
<tr>
<td>7</td>
<td>21</td>
<td>28</td>
<td>100</td>
</tr>
<tr>
<td>8</td>
<td>21</td>
<td>41</td>
<td>100</td>
</tr>
<tr>
<td>9</td>
<td>21</td>
<td>55</td>
<td>100</td>
</tr>
<tr>
<td>10</td>
<td>21</td>
<td>69</td>
<td>100</td>
</tr>
<tr>
<td>11</td>
<td>0</td>
<td>14</td>
<td>100</td>
</tr>
<tr>
<td>12</td>
<td>0</td>
<td>28</td>
<td>100</td>
</tr>
<tr>
<td>13</td>
<td>0</td>
<td>41</td>
<td>100</td>
</tr>
<tr>
<td>14</td>
<td>0</td>
<td>55</td>
<td>100</td>
</tr>
<tr>
<td>15</td>
<td>0</td>
<td>69</td>
<td>100</td>
</tr>
</tbody>
</table>

(preconditioning)

6.9 kPa = 1 psi

Table 3.11 Testing Sequence for Material Type 2 soils (From ASSHTO Designation: T 294-94)

Resilient moduli at different confining and deviator stress levels are calculated using equation 2.1. The recovered deformations are calculated from the average of the recovered deformations of the last five cycles of each loading sequence.
3.5 Conclusions

The soil used in this study was classified as A-6 or CL with its particle size distribution curve and Atterberg limits by AASHTO and USCS classification systems, respectively. The soil specimens were prepared as Material type 2 soils according to AASHTO Designation T 294-94 for modified resilient modulus testing. High backpressure was provided to aid the saturation process. Saturated samples were ready for the modified resilient modulus testing that was based on the standard resilient modulus testing designation T 294-94.
CHAPTER 4

RESILIENT MODULUS TEST PROGRAM

4.1 Introduction

The current standard resilient modulus test is carried according to the AASHTO T294-94 Designation. The loading period is one second, and consists of a 0.1-second haversine load pulse and 0.9-second time of relaxation. At each loading sequence, the number of loading cycles is specified as 100 and the response of the soil to the last five cycles of loading is used to calculate the resilient modulus. However, different highways have different traffic patterns and traffic load frequencies. It is obvious that 1-second period of haversine loading pattern cannot cover all loading possibilities. At dynamic loading, the time interval between last cycle of loading and next cycle of loading will be short. Therefore, the dynamic pore water pressure in the saturated soil sample might not be able to dissipate fully during this interval, and thus the residual water pressure will reduce the effective confining stress and the resilient modulus. The 100-cycle sequence of loading might not be enough to reflect of the reduction of resilient modulus due to residual pore water pressure build-up.
The focus of modified resilient modulus is to concentrate on loading frequencies and loading cycles, which may have a profound effect on the measured resilient modulus of saturated subgrade soil. Therefore, loading periods, different deviator stress levels, and different loading cycles were imposed in modified resilient modulus testing, and the measured pore water pressure response of soil specimens should disclose the influence of these factors.

4.2 Equipment for resilient modulus

The major components of resilient modulus tests are axial-torsion loading system (Figure 4.1) from MTS company, personal computer (including Data Acquisition Board -- PC+ card and LabView® program, Figure 4.2), load cell (Figure 4.5), LVDT (Figure 4.6), pore water pressure transducer (Figure 4.7), triaxial chamber (Figure 4.8), signal conditioner (Figure 4.4), etc.

4.2.1 Loading frame and Controller

The loading system includes the controller and the loading frame. The controller is a servohydraulic system with a 458.20 MicroConsole and a 458.13 AC controller (Figure 4.2). The loading frame has a force capacity of 250kN.
Figure 4.1 Axial-Torsion Measuring System
4.2.2 DAQ card and LabView® Program

The IBM compatible PC is equipped with an internal DAQ board and LabView® software. The DAQ board (manufactured by National Instrument, model PC+) is to collect testing data from the loading system. The DAQ board is configured to collect 4 sets of data, including, load cell data, LVDT data, pore water pressure data, and wave signal data.

The data collection program was written in the LabView® software which is also used to control the loading process. The program is designed to generate a cyclic loading wave signal (Figure 4.3) which is sent to the actuator controller through the DAQ board. The loading cycles can be preset and the loading frequency can be changed through the
front panel of the program. During the modified resilient modulus tests, three sets of data will be collected: force amplitudes (lb), sample deformation (in), and pore water pressure (psi). The program detailed is attached as Figure B. 1.

![Typical Loading signal vs. Time](image)

Figure 4.3 Cyclic haversine loading signals generated by the program
4.2.3 Signal Conditioner

The signal conditioner amplifies small signals from the load cell and pressure transducer, and smoothes some noise out of signals. Figure 4.4 displays the SA-B cards and signal conditioner.

Figure 4.4 Signal conditioner and SA-B cards
4.2.4 Load Cell, LVDT and Pore water pressure transducer

Figure 4.5 Load cell

This load cell (Figure 4.5) used in this testing had a range of 100 lbs and was specially modified to make it waterproof. The electronic cable is submergible. Before the tests, the load cell is calibrated with dead weights to make sure the reading from the load cell linear and reduce the errors introduced by the load cell. The calibration is usually carried out twice. Increasing dead weights and decreasing dead weights make one time. The calibration factor is taken as the average value of the two results, which is in Figure B.2.
The range of the LVDT (Figure 4.6) used in the resilient modulus testing was 0.2 inches. The LVDT was calibrated before each test. Figure B. 3 is one typical calibration curve for LVDT.

The capacity of pore water pressure transducer (Figure 4.7) is 50 psia. Calibration of transducers was also carried out using a pre-calibrated pressure gage. Figure B. 4 is one typical calibration curve for the transducers.
4.2.5 Modified triaxial chamber

The triaxial chamber (Figure 4.8) was modified to meet the requirement of the tests. One drainage line was enlarged to enable the electrical cable to go inside the triaxial chamber. Some modification was made to the vertical loading rod to attach the load cell with it. The confining media was water in order to keep air from passing through membrane under high confining pressure.
Figure 4.8 Modified triaxial chamber used in the tests

4.2.6 Summary of Instrumentation

The following table is a summary of different instrumentations used in modified resilient modulus testing.
<table>
<thead>
<tr>
<th><strong>Loading Frame</strong></th>
<th>250 kN Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Controller</strong></td>
<td>Servo-hydraulic system</td>
</tr>
<tr>
<td><strong>Personal Computer</strong></td>
<td>IBM compatible PC</td>
</tr>
<tr>
<td><strong>DAQ card</strong></td>
<td>PC+ by National Instrument Co. 4 channels when is differentially connected</td>
</tr>
<tr>
<td><strong>LabView® Program</strong></td>
<td>Version 5.1</td>
</tr>
<tr>
<td><strong>Signal Conditioner</strong></td>
<td>With pressure-load dual-channel amplifier cards</td>
</tr>
<tr>
<td><strong>Load cell</strong></td>
<td>Loading range 100 lbs</td>
</tr>
<tr>
<td><strong>LVDT</strong></td>
<td>0.2-inch range</td>
</tr>
<tr>
<td><strong>Pore water pressure Transducer</strong></td>
<td>50-psia capacity</td>
</tr>
<tr>
<td><strong>Modified Triaxial Chamber</strong></td>
<td>Load cell mounted to its loading rod (inside of chamber)</td>
</tr>
</tbody>
</table>

Table 4.1 Test Instrumentations

4.3 Test Design and procedures

The modified resilient modulus tests were carried on the saturated soil samples at different deviator stresses and different frequencies in order to find the effects of these parameters on the buildup of pore water pressure and degradation of resilient modulus. AASHTO T294-94 Designation for resilient modulus tests is used as a reference in the modified resilient modulus tests.
4.3.1 Compaction of soil samples at optimum content and maximum dry density

According to AASHTO Designation T294-94, the soil was compacted to the maximum dry density at optimum moisture content. Table 4.2 shows the compaction results.

<table>
<thead>
<tr>
<th>Dry density (g/cm³)</th>
<th>Moisture content (OMC)</th>
<th>Diameter (cm)</th>
<th>Length (cm)</th>
<th>Dry soil mass (g)</th>
<th>Water mass (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.813</td>
<td>16.6%</td>
<td>72.8</td>
<td>152.4</td>
<td>1149.6</td>
<td>190.8</td>
</tr>
</tbody>
</table>

Table 4.2 Compaction results of soil specimens

4.3.2 Resilient Modulus Tests

Resilient Moduli of unsaturated and saturated soil samples were obtained using standard resilient modulus testing specified in AASHTO T 294-94. The pore water pressure transducer was attached to the triaxial chamber to measure the buildup of pore pressure during the standard resilient modulus testing on the saturated soil samples. During standard resilient modulus testing, the loading period was kept 1-sec/cycle, including 0.1-sec loading time and 0.9-sec relaxing time.
In the following modified resilient modulus testing, number of loading cycles was reduced to 30 to investigate the buildup and dissipation of pore water pressure during the cyclic loading. The correspondent resilient modulus was recorded as well. The loading period was varied to check the effect of length of relaxing time on the pore pressure buildup and degradation of resilient modulus.

The haversine-shaped loading pattern consisted different loading periods. The loading time of each loading cycle at different loading periods was kept constant as 0.1-second, and the relaxing time varied as the loading period changes. For example, a loading period of 1-second consisted 0.1-second of loading time and 0.9-second of relaxing time. A loading period of 2-second consisted 0.1-second of loading time and 1.9-second of relaxing time. Table 4.3 is a summary of the designed modified resilient modulus tests.
<table>
<thead>
<tr>
<th>Sequence</th>
<th>Soil Sample</th>
<th>Confining pressure (psi)</th>
<th>Loading cycles</th>
<th>Loading periods (second)</th>
<th>Deviator stress (psi)</th>
<th>Loads (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Standard Testing, Unsaturated Soil</td>
<td>3</td>
<td>1000</td>
<td>1</td>
<td>2</td>
<td>12.9</td>
</tr>
<tr>
<td>1</td>
<td>Standard Testing, Unsaturated Soil</td>
<td>3</td>
<td>100</td>
<td>1</td>
<td>2</td>
<td>12.9</td>
</tr>
<tr>
<td>2</td>
<td>Standard Testing, Unsaturated Soil</td>
<td>3</td>
<td>100</td>
<td>1</td>
<td>4</td>
<td>25.81</td>
</tr>
<tr>
<td>3</td>
<td>Standard Testing, Unsaturated Soil</td>
<td>3</td>
<td>100</td>
<td>1</td>
<td>6</td>
<td>38.71</td>
</tr>
<tr>
<td>4</td>
<td>Standard Testing, Unsaturated Soil</td>
<td>3</td>
<td>100</td>
<td>1</td>
<td>8</td>
<td>51.61</td>
</tr>
<tr>
<td>5</td>
<td>Standard Testing, Unsaturated Soil</td>
<td>3</td>
<td>100</td>
<td>1</td>
<td>10</td>
<td>64.52</td>
</tr>
<tr>
<td>6</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>1000</td>
<td>1</td>
<td>2</td>
<td>12.9</td>
</tr>
<tr>
<td>7</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>100</td>
<td>1</td>
<td>2</td>
<td>12.9</td>
</tr>
<tr>
<td>8</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>100</td>
<td>1</td>
<td>4</td>
<td>25.81</td>
</tr>
<tr>
<td>9</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>100</td>
<td>1</td>
<td>6</td>
<td>38.71</td>
</tr>
<tr>
<td>10</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>100</td>
<td>1</td>
<td>8</td>
<td>51.61</td>
</tr>
<tr>
<td>11</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>100</td>
<td>1</td>
<td>10</td>
<td>64.52</td>
</tr>
<tr>
<td>12</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>1000</td>
<td>1</td>
<td>3</td>
<td>19.36</td>
</tr>
<tr>
<td>13</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>30</td>
<td>1</td>
<td>2</td>
<td>12.90</td>
</tr>
<tr>
<td>14</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>30</td>
<td>1</td>
<td>6</td>
<td>38.71</td>
</tr>
<tr>
<td>15</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>30</td>
<td>1</td>
<td>8</td>
<td>51.62</td>
</tr>
<tr>
<td>16</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>30</td>
<td>1</td>
<td>10</td>
<td>64.52</td>
</tr>
<tr>
<td>17</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>30</td>
<td>4</td>
<td>6</td>
<td>77.42</td>
</tr>
<tr>
<td>18</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>30</td>
<td>4</td>
<td>8</td>
<td>12.90</td>
</tr>
<tr>
<td>19</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>30</td>
<td>4</td>
<td>10</td>
<td>38.71</td>
</tr>
<tr>
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<td>3</td>
<td>30</td>
<td>6</td>
<td>6</td>
<td>51.62</td>
</tr>
<tr>
<td>21</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>30</td>
<td>6</td>
<td>8</td>
<td>64.52</td>
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<tr>
<td>22</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>30</td>
<td>6</td>
<td>10</td>
<td>38.71</td>
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<tr>
<td>23</td>
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<td>3</td>
<td>30</td>
<td>8</td>
<td>6</td>
<td>51.62</td>
</tr>
<tr>
<td>24</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>30</td>
<td>8</td>
<td>8</td>
<td>64.52</td>
</tr>
<tr>
<td>25</td>
<td>Standard Testing, Saturated Soil</td>
<td>3</td>
<td>30</td>
<td>8</td>
<td>10</td>
<td>38.71</td>
</tr>
</tbody>
</table>

Table 4.3 Testing Sequences
4.3.3 Reliability of Resilient Modulus Testing and Fast Fourier Transform (FFT)

In the pore water pressure data, there are some fluctuations in the peak pressure. This problem could be caused by the loading system, the pore water pressure measurement system, sampling rates, or the structural response of the soil samples.

Two different sampling rates (50 samples/second and 100 samples/second) were used in some tests to check the influence of sampling rates on the response of pore water pressure. The dominant frequency of the pore water response was identified for the two sampling rates.

The natural frequency of the clay column was determined by Equation 4.1 (Kramer, 1993).

\[ f_0 = \frac{1}{2\pi} \sqrt{\frac{M_r \cdot A}{m \cdot L}} \]  

(4.1)

where \( f_0 \) is the natural frequency of the soil specimen, \( A \) is the area of specimen cross-section, \( L \) is the length of the soil specimen, and \( m \) is the mass of the soil specimen.

Some testing sequences were repeated to check the reliability of the test data.

4.4 Conclusions

In order to meet the purpose of modified resilient modulus testing, some modification was made to the testing instrumentation. One calibrated pore water pressure transducer was attached to the triaxial chamber to measure the pore water pressure response inside of the saturated soil specimens.

Testing procedure was modified as well. Conditioning process would be terminated if the pore water pressure was too high or plastic or non-uniform behavior of
soil specimens was not negligible. The loading frequency could be changed to measure the influence of loading frequency on the degradation of resilient modulus and buildup of pore water pressure. The loading cycles were not limited to 100-cycle at each loading sequence in order to check the buildup of the pore water pressure during the cyclic loading.
CHAPTER 5

DATA ANALYSIS AND COMPARISON

5.1 Introduction

The modified resilient modulus tests were carried on the fully saturated soil specimens. The purpose of the tests was to find out the effects on resilient modulus caused by residual pore water pressure buildup under cyclic loading. The loading periods, deviator stress, and number of loading cycles are different parameters influencing the buildup of residual pore water pressure.

The following sections showed effects of varying the loading parameters on the elastic response of the soil as measured by the resilient modulus.
5.2 Test results and discussion

5.2.1 Comparison of the Resilient Moduli from standard testing

As shown in Figure 5.1, the resilient modulus of saturated soil sample is approximately one half of that of soil samples with optimum moisture content at each corresponding deviator stress level. According to previous research (Mohammad et al. 1996), reduction of resilient modulus, caused by an increase in moisture content in clay from optimum to wet, is approximately 30%. In the resilient modulus testing performed in this thesis on saturated soil samples, pore water pressure increases were observed (Figure 5.2). Therefore, it is possible that the pore water pressure buildup in the soil caused the additional reduction of resilient modulus in the cyclic loading process.

5.2.2 Data from 30 cycles of loading with loading period of one second

The first group of tests was carried out after the dynamic pore water pressure dissipated, which was built up in the conditioning process. The loading period was 1-second and number of loading cycles was chosen to be 30. Four sequences of testing were carried out at different deviator stress levels, 2, 6, 8, and 10-psi. Each testing sequence was carried out after the pore water pressure created by the previous testing sequence dissipated. Figure 5.3 shows the dissipation of residual pore water pressure buildup after the 30 cycles of loading at different deviator stress levels.

From this figure, some observations can be made. Residual pore water pressure built up after several cycles of loading and it took some time form it to dissipate completely. Residual pore water pressure herein is defined as the pore water pressure measured just after the time when the loading cycle is imposed. Higher deviator stress
created higher dynamic pore water pressure build-up. At a loading period of 1 sec/cycle, there was no obvious pore water pressure buildup of deviator stress levels of 2 and 6-psi. This agreed with the threshold deviator stress theory of Ansal and Erken 1989, Medndoza and Hernandez 1994.

![Resilient Modulus vs. Deviator stress (100 cycles)](image)

(Loading period 1 sec/cycle, confining stress 3-psi)

Figure 5.1 Resilient modulus vs. deviator stress
Figure 5.2 Pore water pressure buildup

Figure 5.4 shows the time required to reach 50% of residual pore water pressure remaining after 30 cycles of loading. The time increased slightly with an increase in the deviator stress. Considering the consolidation after each loading sequence and the experimental variation, the required time didn’t vary significantly at different deviator stress levels. This is consistent to the Terzaghi one-dimensional consolidation equation. However, the time required for pore pressure to dissipate to a fixed small value can be applied easily in practice. Therefore, considering the accuracy of the pore water pressure transducer, the dissipation time is defined as the time which is required for pore pressure to dissipate to 0.05 psi.
Figure 5.3 Pore water pressure vs. time at different deviator stress levels

Figure 5.5 shows the relationship between residual pore water pressure buildup and deviator stress levels. Figure 5.6 shows the relationship between time needed for residual pore water pressure to dissipate and the deviator stress levels. It took longer time for pore water pressure to dissipate under higher deviator stress levels. Larger deviator stress tended to create higher residual pore water pressure, which increased the dissipation time.
Figure 5.4 Time required for pore water pressure to dissipate to 50% of its residual pore water pressure at different deviator stress levels

Residual Pore Water Pressure vs. Deviator Stress (30 cycles)
/loading period 1-sec/cycles, confining stress 3-psi/

Figure 5.5 Residual Pore pressure vs. deviator stress (loading period 1-sec/cycle)
The dissipation time is defined as the time which is required for pore pressure to dissipate to 0.05 psi.

Figure 5.6 Dissipation Time vs. deviator stress (loading period 1-sec/cycle)

5.2.3 Data from 30-cycle loading duration with different loading periods

Similar tests were done with different loading periods of 2, 4, 6, and 8-second to study the relationship between dissipation time and loading periods. Figure 5.6 shows the influence of periods and deviator stresses on the buildup of residual pore water pressure. As shown in Figure 5.6, the residual pore water pressure decreased as the period of loading increased. Larger deviator stresses tended to create higher residual pore water pressure.
Figure 5.7 $U_{\text{residual}}$ immediately after 30-cycle loading vs. loading period (at different deviator stress levels)

Figure 5.7 shows how the dissipation time is related to loading period and deviator stress level. The required dissipation time decreased, as the loading period increased at each deviator stress level, and increased as the deviator stress level increased at each loading period. Again, larger deviator stress level resulted in longer critical loading period.
Dissipation Time vs. Loading period (confining stress 3-psi)

Figure 5.8 Dissipation Time vs. loading period (at different deviator stress levels)

From the above analysis, it is clear that the cyclic loading did result in the buildup of pore water pressure in saturated soil samples. The length of relaxing time between two consecutive loading cycles determined the length of dissipation time of pore water pressure. Shorter loading period created higher pore water pressure buildups and required longer dissipation time. The buildup of pore water pressure reduced effective confining stress, which would possibly reduce the stiffness of the soil. Figure 5.8 shows the degradation of resilient modulus due to the pore water pressure buildup at different loading periods and deviator stress levels. The resilient modulus of saturated soil was
higher at longer loading period at each deviator stress level. It is because that the dynamic pore water pressure had longer relaxing time between two spikes of cyclic loading to dissipate before the next loading cycle. Therefore, the effective confining stress would recover more than that at shorter loading period.

![Resilient Modulus vs. Deviator Stress](image)

Figure 5.9 Degradation of Resilient Modulus
5.2.4 Reliability and Problems in the collected data

5.2.4.1 Fluctuation in Peak Pore Pressure

In the collected pore water pressure data, there is fluctuation in the peak pressure when the soil was subjected to a cyclic loading at constant amplitude of deviator stress (Figure 5.3). Sampling rate is a potential factor which might have some effect on the collected data.

Figure 5.10 Analytical Haversine signals with 0.1-sec loading time and 0.9-sec unloading time at each loading cycle (ideal signals)
Figure 5.11 Loading signals: FFT with the sampling rate at 100 sample/sec (loading period 1-sec/cycle)
Figure 5.12 Loading signals: FFT with the sampling rate at 50 sample/sec (loading period 1-sec/cycle)

Figure 5.9 shows the ideal frequency components of input loading signal with a 0.1-second loading time and 0.9-sec unloading time at each cycle. Figure 5.10 and Figure 5.11 show actual frequency components of the input loading wave in frequency domain after Fast Fourier Transform (FFT) at the sampling rate of 100-sample/sec and 50-sample/sec, respectively. As shown in those three plots, they are similar to each other.
The distribution of actual frequency components is close to that of the ideal frequency components, which indicates the data collected at 100-sample/sec were accurate to represent the ideal input signals. In addition, the difference of amplitudes and positions between the plot of sampling rate at 100-sample/sec and that at 50-sample/sec is negligible at low frequency range (0 – 25 Hz). There are some small spikes at high frequency range (25 – 50 Hz) in the plot of sampling rate at 100-sampl/sec, which those high frequency spikes have far less energy than those low frequency spikes and it will not introduce too many errors if those high frequency components spikes are neglected. Therefore, given the high frequency (> 50 Hz) spikes are negligible, sampling rate at 100-sample/sec is fast enough to collect the necessary information.
Figure 5.13 Pore Pressure: FFT with the sampling rate at 100 sample/sec (loading period 1-sec/cycle)
Figure 5.14 Pore pressure: FFT with the sampling rate at 50 sample/sec (loading period 1-sec/cycle)

Figure 5.12 and Figure 5.13 show the frequency components of the pore water pressure response in frequency domain after FFT with the sampling rate of 100-sample/sec and 50-sample/rate. The difference in amplitudes and positions of low frequency spikes is negligible except the amplitude of the spikes at 0-Hz. This is because the starting residual pore water pressures were different at these two processes. The 0-Hz spikes represent the DC signal which is the residual pore water pressures in these two
plots. The dynamic part of pore water pressure is identical to each other at low frequency range and the high frequency spikes are small enough to neglect. Therefore, the 100-sample/sec sampling rate is fast enough to correctly record the pore water pressure response. Therefore, the fluctuation in the peak pressure response is not a sampling rate-related issue. There are some other possibilities causing it.

The natural frequency of saturated soil specimen is calculated with Equation (4.1). Table 5.1 lists the natural frequencies of specimens at different deviator stress levels.

**Resilient Modulus of 100-cycle loading**  
*(loading period 1 sec/cycle, confining stress 3-psi)*

<table>
<thead>
<tr>
<th>Deviator Stress</th>
<th>Resilient Modulus</th>
<th>Natural Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 (psi)</td>
<td>5968</td>
<td>144.5 (Hz)</td>
</tr>
<tr>
<td>4 (psi)</td>
<td>4431</td>
<td>124.5 (Hz)</td>
</tr>
<tr>
<td>6 (psi)</td>
<td>2907</td>
<td>100.8 (Hz)</td>
</tr>
<tr>
<td>8 (psi)</td>
<td>1678</td>
<td>76.6 (Hz)</td>
</tr>
</tbody>
</table>

Table 5.1 Natural Frequency

As shown in Table 5.1, natural frequencies of soil specimens at different deviator stress levels were higher than the loading frequency. However, if there is high-frequency
fluctuation in the loading or pressure signals, the soil specimens could have resonant response because the natural frequency might be close to these high-frequency signals. Therefore, fluctuation-free signals in loading waves are important to find what caused the fluctuation in peak pore water pressure response.

5.2.4.2 Repeatability of Modified Resilient Modulus

As shown in Figure 5.8, the variation of resilient modulus is not significant in different soil samples, as long as the moisture content is the same. Therefore, the testing method listed in Chapter 4 is repeatable.
CHAPTER 6

SUMMARY, CONCLUSION AND RECOMMENDATIONS

According to AASHTO Guide for Design of Pavement Structures 1993, the resilient modulus of subgrade soil is obtained at optimum moisture content with the consideration of effects of the variation of moisture content. However, some pavement sections show larger deflection than they were designed for. In the current SHRP field monitoring, high pore water pressures were observed in some pavement sections under traffic loading. The possible pore water pressure buildup in saturated soils, which were subjected to cyclic loading, would reduce the effective confining stress. Consequently, the resilient modulus would decrease in saturated soils. Therefore, the current method of calculating resilient modulus of subgrade soil may not be conservative without consideration of the effects of pore water pressure buildup in saturated subgrade soils.

In order to investigate this hypothesis, a set of resilient modulus tests were carried out on soil specimens at optimum moisture content and under saturated conditions. The test results showed the resilient modulus of saturated soils reduced by around 50% of that of soil specimens at optimum moisture content. According to the previous research
(Mohammad, 1996), this mount of increase in moisture content would lead to around 30% of reduction of resilient modulus at different deviator stress levels. Some reduction would possibly be caused by the buildup of pore water pressure, which was shown by the modified resilient modulus testing under laboratory cyclic loading to simulate traffic loading in the field.

The modified resilient modulus testing also investigated the parameters influencing resilient modulus degradation. It is known that resilient modulus will decrease with the increase in deviator stress (Mohammad 1994, 1996, Dram et al, 1997). The resilient modulus of partially saturated soil is independent of loading period (Brown et al., 1975, Seim, 1989, and Ansal et al, 1989). However, saturated soil has a different behavior. The modified resilient modulus testing showed the pore water pressure buildup depended on the relaxing time between two spikes of loading cycles, that is, loading period. As shown in the testing, shorter loading period tended to create higher residual pore water pressure because the residual pore water pressure didn’t have enough time to fully dissipate before the next cycle of loading. Thus, it took longer time for residual pore water pressure to dissipate at shorter loading period. The resilient modulus of saturated soil was smaller under faster cyclic loading.

Therefore, the resilient modulus of saturated soils is not a basic soil property, instead it depends on the boundary conditions, including loading periods. In the design of pavement sections, it is important to select proper loading period to simulate traffic loading on highway pavements with different speed limits.

Some difficulties need to be solved and further testing needs to be done for comprehensive research on the behavior of saturated soil under cyclic loading. It is time-
consuming to saturate compacted clay specimens even with high backpressure because of the low permeability of compacted clay. It will be more accurate to draw comprehensive conclusions statistically with different types of clay samples involved. Different types of soil samples will make analogy or comparison available for the purpose of research on the behavior of saturated soils. There are some other parameters which might also influence the resilient modulus and buildup of pore water pressure, for example, cyclic loading number. It will be interesting to check the effects of different loading cycles on the behavior of saturated soils, if the modified resilient modulus testing can be run continuously without failing the specimens.

Since subgrade soil is usually compacted at optimum moisture content, it is not clear to us how the subgrade soil get saturated under pavement layers as time passes by. It will be important to show how the soil gets saturated using the laboratory testing due to the negative pore water pressure and to compare the response of pore water pressure in unsaturated soil under traffic loading to that of saturated soil. It is important to realize the non-uniform distribution of pore water pressure in partially saturated soil under cyclic loading since water flow rate is low in compacted clay during cyclic loading. It is hard for pore water pressure to reach a balance throughout the height of the specimens during the interval between two cycles of loading. Therefore, some special measuring equipment needs to be plugged into the soil sample to measure the negative pore water pressure and its distribution, instead of using porous stone to measure the pore water pressure at boundary. This equipment should be able to separate pore water pressure from pore air pressure since only the pore water pressure causes the water suction in partially saturated soil.
The reason causing the fluctuation in the pore water pressure response in saturated soil is still unknown in spite of the current laboratory testing. Further study is recommended for a better understanding the behavior of saturated soil under cyclic loading.
LIST OF REFERENCES


**Internet Sources Citation:**


APPENDIX A

ENGINEERING INDICES
<table>
<thead>
<tr>
<th>Sample Name</th>
<th>ODOT Del. 23 390901 7-25-95</th>
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<tbody>
<tr>
<td>Test date</td>
<td>02/15/2000</td>
</tr>
<tr>
<td>Mass of can (g)</td>
<td>15.50</td>
</tr>
<tr>
<td>Mass wet soil + can (g)</td>
<td>124.81</td>
</tr>
<tr>
<td>Mass wet soil (g)</td>
<td>109.31</td>
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<tr>
<td>Mass dry soil + can (g)</td>
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<tr>
<td>Mass dry soil (g)</td>
<td>98.75</td>
</tr>
<tr>
<td>Mass water (g)</td>
<td>10.56</td>
</tr>
<tr>
<td>Moisture content (%)</td>
<td>10.7%</td>
</tr>
</tbody>
</table>

Table A.1 Initial Moisture Content
<table>
<thead>
<tr>
<th>Sample No.: 390101</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Compaction method:</strong></td>
</tr>
<tr>
<td><strong>Rammer Weight:</strong> 5.50 lb</td>
</tr>
<tr>
<td><strong>Rammer Height:</strong> 305 mm</td>
</tr>
<tr>
<td><strong>Moisture Content:</strong></td>
</tr>
<tr>
<td><strong>Test Number</strong></td>
</tr>
<tr>
<td><strong>Date</strong></td>
</tr>
<tr>
<td><strong>Mass of can (g)</strong></td>
</tr>
<tr>
<td><strong>Mass can + wet soil (g)</strong></td>
</tr>
<tr>
<td><strong>Mass wet soil (g)</strong></td>
</tr>
<tr>
<td><strong>Mass dry soil + can (g)</strong></td>
</tr>
<tr>
<td><strong>Mass dry soil (g)</strong></td>
</tr>
<tr>
<td><strong>Mass water (g)</strong></td>
</tr>
<tr>
<td><strong>Water Content (%)</strong></td>
</tr>
<tr>
<td><strong>Density and Unit Weight:</strong></td>
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</tr>
<tr>
<td><strong>Water Content (%)</strong></td>
</tr>
<tr>
<td><strong>Mass of mold (g)</strong></td>
</tr>
<tr>
<td><strong>Mass of mold + soil (g)</strong></td>
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<tr>
<td><strong>Mass of soil (g)</strong></td>
</tr>
<tr>
<td><strong>Volume of mold (cm³)</strong></td>
</tr>
<tr>
<td><strong>Wet unit weight (kg/m³)</strong></td>
</tr>
<tr>
<td><strong>Dry unit weight (kg/m³)</strong></td>
</tr>
</tbody>
</table>

Table A. 2 Proctor compaction
Figure A. 1  Standard Proctor Compaction Test
Figure A. 2  Unconfined Compression Test on Unsaturated Soil Specimens
Figure A.3  Unconfined Compression Tests on Saturated Soil Specimens
APPENDIX B

DATA ACQUISITION PROGRAM
B.1 Data Acquisition Program

This Data Acquisition Program was modified from the one used in Kim, D. G. (1999). Figures B.1 and B.2 show a main front panel and a block diagram, respectively. This program can generate a continuously haversine wave for a cyclic loading. For channels of data are displayed and collected at a certain sampling rate, including loading signal, LVDT, pore water pressure, and the generated haversine wave signal.

The modified program can generate a continuously haversine wave at differently frequency by keeping the loading time constant and varying relaxing time between two spikes of loading cycles.

Figure B.2 shows that the program is implemented by two loops and one sequence. The sequence is to generate different haversine waves as the input parameter varies. One loop is responsible for data collecting. The upper loop is to count applied load and to quit each loading stage.
Figure B. 1 Front Panel of Data Acquisition Program
Figure B. 2 Block Diagram of Data Acquisition Program
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(confining stress 3-psi, loading period 6-sec/cycle, deviator stress 8-psi, sampling rate 100-samples/sec)
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sample/sec)