CHAPTER 5

SOIL CLASSIFICATION AND LABORATORY TESTING

5.0 GENERAL: WEIGHT VOLUME RELATIONSHIP

In nature, soils are three-phase systems consisting of solid soil particles, water, and air (or gas). To develop the weight-volume relationships for a soil, the three phases can be separated as shown in Figure 3.1a. Based on this separation, the volume relationships can be defined in the following manner.

Void ratio, $e$, is the ratio of the volume of voids to the volume of soil solids in a given soil mass, or

$$ e = \frac{V_v}{V_s} $$  \hspace{1cm} \text{Equation (5.1)}

where $V_v =$ volume of voids
$V_s =$ volume of soil solids

Porosity, $\eta$, is the ratio of the volume of voids to the volume of the soil or,

$$ \eta = \frac{V_v}{V} $$  \hspace{1cm} \text{Equation (5.2)}

where $V =$ total volume of soil

Moreover,

$$ \eta = \frac{V_v}{V} = \frac{V_v}{V_s + V_v} = \frac{V_v}{V_s + \frac{V_v}{V_s}} = \frac{e}{1 + e} $$  \hspace{1cm} \text{Equation (5.3)}

Degree of saturation, $S$, is the ratio of the volume of water in the void spaces to the volume of voids, generally expressed as a percentage, or

$$ S = \frac{V_w}{V_v}, \hspace{1cm} S \, (\%) = \frac{V_w}{V_v} \times 100 $$

Weight Volume Relationship

Total Weight = $W$
$W = W_a + W_s + W_w = W_w + W_s \; (W_a = 0)$
$W_a =$ weight of air
$W_w =$ weight of water
$W_s =$ weight of solid
Total Volume = $V$
$V = V_a + V_w + V_s$

$V_a, V_w,$ and $V_s$ are volumes of air, water and solids in the soil.

Specific Gravity of solid ($G = G_s$)

$$ G = \frac{\gamma_s}{\gamma_w} $$
\( \gamma_s \) and \( \gamma_w \) are unit weights of solid and water.

**Weight of solid \( W_s \)**

\[
W_s = \gamma_s \times V_s \\
= (G \gamma_w) \times V_s \\
= G \gamma_w \text{ (when } V_s = 1\text{)}
\]

(a) Components of Soils

Figure 5.1 Weight Volume Relationship

\[
V_s = 1 \\
V_w = V_w \times Y_w \\
W_w = \text{Air} \\
W_a = 0 \\
W_s = G \gamma_w \times V_s = G \gamma_w \text{ (when } V_s = 1\text{)}
\]
\[ V_v = V_a + V_w \]

(Note: \( V_w = w \frac{G_s}{e} \), when \( V_s = 1 \))

(b) Unsaturated soil; \( V_s = 1 \), three phase diagram

\[
\begin{array}{cc}
\text{Volume} & \text{Weight} \\
V_w = w \frac{G_s}{e} & W_w = e \gamma_w = G \gamma_w \\
V_w = V_v = e & W_s = G_s \gamma_w \text{ (} V_s = 1 \text{)} \\
V_s = 1 & \\
\end{array}
\]

(c) Saturated soil; \( V_s = 1 \)

\[ S(\%) = \frac{V_w}{V_v} \times 100 \quad \text{Equation (5.4)} \]

Where \( V_w \) = volume of water

Note that, for saturated soils, the degree of saturation is 100%.

The weight relationships are moisture content, moist unit weight, dry unit weight, and saturated unit weight. They can be defined as follows:

(i). Moisture or water content (\%) = \( w = \)

\[ \frac{\text{Weight of water} (W_w)}{\text{Weight of dry soil} (W_s)} \times 100 \quad \text{Equation (5.5)} \]
(ii). Moist (Natural or Bulk) density (lb/cu ft)

\[ \gamma_{\text{bulk}} = \gamma = \frac{G_s \gamma_w (1 + w)}{1 + e} \]  
Equation (5.6)

(iii). Dry unit weight \( Y_{\text{dry}} \) (lb/cu ft).

\[ \gamma_{\text{dry}} = \frac{\gamma}{1 + w} = \frac{G \gamma_w}{1 + e} \]  
Equation (5.7)

(iv). Saturated unit weight (lb/cu ft)

\[ \gamma_{\text{sat}} = \frac{\gamma_w (G + e)}{(1 + e)} \]  
Equation (5.8)

(v). Submerged unit weight (lb/cu ft)

\[ \gamma_{\text{sat}} - \gamma_w = \frac{\gamma_w (G - 1)}{1 + e} \]  
Equation (5.9)

Where \( G = G_s \) = specific gravity of solids of soils

\( \gamma_w \) = Unit weight of water (62.4 lb/cu ft)

5.1 MOISTURE CONTENT

This test shall consist of determination of moisture content in accordance with AASHTO T-265, on all fine grained soil samples. It is important to note that the moisture content (w) is expressed in percent as expressed in equation 5.5. This test shall not be performed on soils with less than 35% passing #200 Sieve (0.075 mm) without prior approval.
5.2 SPECIFIC GRAVITY TEST

This test shall be performed in accordance with AASHTO T-100. Most Indiana soils shall have specific gravities ranging between 2.60 to 2.75. Soils with organic content or porous particles such as slag or coal combustion by-products may have specific gravities which are much lower.

5.3 CLASSIFICATION TESTS

5.3.1 GRAIN SIZE ANALYSIS

These tests shall be performed on samples that were obtained for verification of the field classification of the major soil types encountered during the investigation. The number of tests shall be limited to reasonably establish the stratification without duplication, unless approved otherwise. A minor soil type, if not critical, may be given a visual classification, instead of performing classification tests for reference.

5.3.1.1 SIEVE ANALYSIS

A sieve is a quantitative determination of the distribution of particle sizes present in the soil sample. The testing will be accompanied by means of sieve and hydrometer analyses. This test consists of determining gradation of a sample in accordance with AASHTO T-88 and Indiana Department of Transportation’s triangular classification chart as given in this section. All the soils shall be classified in accordance with AASHTO M-145.

Sieves shall be U.S. sieve sizes: seventy-five (75) mm, fifty (50) mm, twenty-five (25) mm, 9.5 mm and U.S. No. 4, No. 10, No. 40 and No. 200.

5.3.1.2 HYDROMETER ANALYSIS

This work shall consist of the Hydrometer Analysis in accordance with AASHTO T-88, and includes a Specific Gravity Determination performed in accordance with AASHTO T-100. If twenty percent (20%) or more passes the No. 200 Sieve a Hydrometer Analysis shall be performed. A grain-size distribution curve shall be provided and should include the combined results of the Sieve Analysis.

5.3.2 ATTERBERG LIMITS AND PLASTICITY INDEX (PI)

The Liquid Limit (LL) is determined according to AASHTO T-89 method. The Plastic Limit (PL) and Plasticity Index (PI) are determined according to AASHTO T-90. A fine grained soil can exist in any of several states of consistency. The state of consistency and the behavior of any particular soil depend primarily upon the amount of water present in the soil-water system. In 1911, A. Atterberg defined the boundaries of four (4) states of consistency, in terms of limits. These limits and the zones between the limits are illustrated in Figure 5.2.

Each limit represents a moisture content, beyond which the soil changes from one state to another. The PI (Plasticity Index) represents the range of moisture contents, through which the soil is in the plastic state. The PI is simply the moisture content at the LL, minus the moisture content at the PL.
The limits are useful for soil classification and correlation with the soil behavior; such as, compressibility, permeability, shrink/swell and strength. The SL can be useful in predicting the maximum loss of volume, which an embankment material may undergo when removed from a wet borrow, and subsequently dried and rolled into a fill. As soil dries to the SL, there is a loss of volume and water. Further drying removes water only, without corresponding volume loss.

<table>
<thead>
<tr>
<th>Liquid State</th>
<th>LIQUID LIMIT (LL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plastic State</td>
<td>PLASTIC LIMIT (PL)</td>
</tr>
<tr>
<td>Semisolid State</td>
<td>SHRINKAGE LIMIT (SL)</td>
</tr>
<tr>
<td>Solid State</td>
<td>DRY</td>
</tr>
</tbody>
</table>

Figure 4.2 Soil Water Scale Showing Atterberg Limits And Corresponding Physical States Of Soil In The Remolded Condition

The LL, PL and SL are arbitrary boundaries, but the procedures for obtaining these values have been standardized. The LL is determined in the lab by measuring the moisture content at which a standard groove of soil, placed in a standard brass cup, will close when the cup is dropped 25 times from a 0.394 in. (10 mm) height. The PL is determined by measuring the moisture content at which a thread of soil begins to crumble, when rolled into a 1/8 in. (3 mm) diameter. The SL is determined by drying a saturated soil, and measuring the limiting moisture content at which no further volume changes occur with loss of water.

Plasticity index of A-7-5 subgroup is equal to or less than Liquid Limit minus 30. Plasticity Index of A-7-6 subgroup is greater than Liquid Limit minus 30.

Note: Additional parameters of group index (GI) are determined to classify fine soils.

Group Index (GI)

Group Index shall be calculated after performing the classification and Atterberg Limit and is reported along with the Classification Test. Group Index indicated the plastic nature of the portion of the material passing No. 200 sieve. Calculation of the group index is the final part of the AASHTO Classification. Generally, the higher the value of the group index for a given classification, the poorer the performance of the soil.

The formula used to calculate the group index is as follows.

\[
GI = (F - 35)[(0.2 + 0.005)(LL - 40)] + 0.01(F - 15)(PI - 10)
\]

\(GI\) = Group Index. Reported as a positive whole number or zero.
\(F\) = Percentage passing the No. 200 sieve.
\(LL\) = Liquid Limit.
\(PI\) = Plasticity Index.
Table 5.4  Classification of Soil and Soil-Aggregate Mixtures from AASHTO M-145

<table>
<thead>
<tr>
<th>General Class.</th>
<th>Granular Materials (35% Or Less Passing No. 200)</th>
<th>Silt-Clay Materials (More than 35% Passing No. 200)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sieve Analysis, % Passing</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 10</td>
<td>50 max.</td>
<td></td>
</tr>
<tr>
<td>No. 40</td>
<td>30 max.</td>
<td>50 max.</td>
</tr>
<tr>
<td>No. 200</td>
<td>15 max.</td>
<td>25 max.</td>
</tr>
<tr>
<td>Charac.’s of Fraction passing No. 40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Liquid Limit Plasticity Index</td>
<td>6 max.</td>
<td>N.P.</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 Rating as Subgrade</td>
<td>Excellent to Good</td>
<td></td>
</tr>
</tbody>
</table>

Fair to Poor
INDOT Soil Classification

INDOT Textural Soil Classification Definitions:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Boulder</td>
<td>&gt; 10 in</td>
</tr>
<tr>
<td>Cobbles</td>
<td>10 in - 3 in</td>
</tr>
<tr>
<td>Gravel</td>
<td>3 in - No. 10 Sieve</td>
</tr>
<tr>
<td>Coarse Sand</td>
<td>No. 10 - No. 40 Sieve</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>No. 40 - No. 200 Sieve (0.075 mm.)</td>
</tr>
<tr>
<td>Silt</td>
<td>0.075 - 0.002 mm.</td>
</tr>
<tr>
<td>Clay</td>
<td>&lt; 0.002 mm.</td>
</tr>
<tr>
<td>Colloids</td>
<td>&lt; 0.001 mm</td>
</tr>
</tbody>
</table>

Any Soil with organic content (as determined in accordance with AASHTO T-267), 3% or less is considered mineral soil.
Soils having 0 to 19% Retained on No. 10 sieve (Chart below may be used)

<table>
<thead>
<tr>
<th>Classification</th>
<th>% Sand &amp; Gravel</th>
<th>% Silt</th>
<th>% Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>80-100</td>
<td>0-20</td>
<td>0-20</td>
</tr>
<tr>
<td>Sandy Loam</td>
<td>50-80</td>
<td>up to 50</td>
<td>up to 20</td>
</tr>
<tr>
<td>Loam</td>
<td>30-50</td>
<td>30-50</td>
<td>Up to 20</td>
</tr>
<tr>
<td>Silty Loam</td>
<td>Up to 50</td>
<td>50-80</td>
<td>0-20</td>
</tr>
<tr>
<td>Silt</td>
<td>0-20</td>
<td>80-100</td>
<td>0-20</td>
</tr>
<tr>
<td>Sandy Clay Loam</td>
<td>50-80</td>
<td>0-30</td>
<td>20-30</td>
</tr>
<tr>
<td>Clay Loam</td>
<td>20-50</td>
<td>20-50</td>
<td>20-30</td>
</tr>
<tr>
<td>Silty Clay Loam</td>
<td>Up to 30</td>
<td>50-80</td>
<td>20-30</td>
</tr>
<tr>
<td>Sandy Clay</td>
<td>50-70</td>
<td>0-20</td>
<td>30-50</td>
</tr>
<tr>
<td>Silty Clay</td>
<td>0-20</td>
<td>50-70</td>
<td>30-50</td>
</tr>
<tr>
<td>Clay</td>
<td>0-50</td>
<td>0-50</td>
<td>30-100</td>
</tr>
</tbody>
</table>

Soils having 20% or more retained on No. 10 sieve and more than 20% passing No. 200 sieve (Silt and Clay).

Classify in accordance with Paragraph A, followed by term describing relative amount of gravel according to the following:

- 20% to 35% gravel – “With some gravel”
- 36% to 50% gravel – “and gravel”

Examples: Clay Loam with some gravel
          Sandy Loam and gravel

Soils having 20% or more retained on No. 10 sieve less than 20% passing No. 200 sieve.

<table>
<thead>
<tr>
<th>Classification</th>
<th>% Gravel</th>
<th>% Sand</th>
<th>% Silt</th>
<th>% Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>85-100</td>
<td>0-15</td>
<td>0-15</td>
<td>0-15</td>
</tr>
<tr>
<td>Sandy Gravel</td>
<td>40-85</td>
<td>15-40</td>
<td>0-20</td>
<td>0-20</td>
</tr>
<tr>
<td>Gravelly Sand</td>
<td>20-40</td>
<td>40-80</td>
<td>0-20</td>
<td>0-20</td>
</tr>
<tr>
<td>Sand and Gravel</td>
<td>20-50</td>
<td>20-50</td>
<td>0-20</td>
<td>0-20</td>
</tr>
</tbody>
</table>

Note: When the gradation of a given sample does not meet the requirements for any classification exactly, it shall be placed in the classification to which it comes the closest.

Organic Soils. The following classification system shall be used for organic soils in accordance with AASHTO T-267.

<table>
<thead>
<tr>
<th>Classification</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>With Some Organic Matter</td>
<td>3&lt;oc*≤15</td>
</tr>
<tr>
<td>Organic Soil (A-8)</td>
<td>15&lt;oc≤30</td>
</tr>
<tr>
<td>Peat (A-8)</td>
<td>&gt;30</td>
</tr>
</tbody>
</table>

- Organic Content in Percentage
Marly Soils: The following classification system shall be used for marly soils with calcium and magnesium carbonate content.

<table>
<thead>
<tr>
<th>Classification</th>
<th>Content Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>With Trace Marl</td>
<td>1% TO 9%</td>
</tr>
<tr>
<td>With Little Marl</td>
<td>10% to 17%</td>
</tr>
<tr>
<td>With Some Marl</td>
<td>18% to 25%</td>
</tr>
<tr>
<td>Marly Soil (A-8) No group index</td>
<td>26% to 40%</td>
</tr>
<tr>
<td>Marl (A-8) No group index</td>
<td>More than 40%</td>
</tr>
</tbody>
</table>

5.4 pH TEST

This test shall consist of performing the pH test in accordance with ASTM D-2976 using only distilled water. The test should be performed on all classification test samples, and others as necessary. When the test is performed on moderate to non-organic material, sample size should be 0.04 lbs (20.0 grams) of material passing the sieve size 4.75 millimeter. The samples shall be prepared in accordance with AASHTO T-87.

5.5 LOSS OF IGNITION

This test shall consist of determination of the Loss on Ignition (Organic Content) in accordance with AASHTO T-267. This method will provide a quantitative estimation of oxidized organic matter in the soil mass. Organic matter present in soil shall be reported as a percentage in whole number (not in fraction).

5.6 UNIT WEIGHT DETERMINATION

This test shall consist of the determination of the Unit Weight by measurement of the length and diameter of sample. The procedure to determine unit weight is also described in EM-1110-2-1906 App. II of the Corps of Engineers.

5.7 STANDARD MOISTURE-DENSITY RELATIONS

This work shall consist of performing the Standard Moisture-Density Relations in accordance with AASHTO T-99 Method D. A minimum of four (4) points on the curve with at least two (2) points on each side of optimum shall be performed in conjunction with all CBR test samples, and shall be prepared and tested in accordance with AASHTO T-193 except the sample shall be mixed and then cured for forty-eight (48) hours prior to molding the specimens. AASHTO T-99 Method D shall be used in conjunction with all CBR tests.
Figure 5.3 Typical Curve Showing the Relationship Between Moisture Content and Dry Density. Zero Air Voids Curve Represents 100% Saturation.

5.8 ONE DIMENSIONAL CONSOLIDATION TEST

This work shall consist of performing the consolidation test in accordance with AASHTO T-216 except the load increments shall be 0.06, 0.12, 0.25, 0.50, 1, 2, 4, 8, and 16 T/Sq.Ft. This test also includes Specific Gravity, Initial and Final moisture Content tests, Initial and Final Degrees of Saturation and Unit Weights (density). Time curves for all load increments and e-log-p curve shall also be furnished. Laboratory data, sheets and e-log-p graph sheets are included in Appendices 5 (4.3) to 7 (4.5).

5.9 UNCONFINED COMPRESSIVE STRENGTH TEST

This test is commonly referred to as the Qu Test, and shall consist of performing the Unconfined Compressive Strength Test in accordance with AASHTO T-208. This test includes determination of initial and final moisture contents, unit weight determination, visual descriptions of the soil, average rate of strain to failure and strain at failure. The sample shall be undisturbed and have a minimum diameter of 1.3 inches (33 millimeters) unless other types are approved in advance for the specific project. The test is a special case of Triaxial Compression Test in which the confining pressure, \( \sigma_3 \) is zero as shown in Figure 5.4. It is performed by loading a soil specimen at a constant rate, to the failure load. It may be expressed in tsf, or kPa, or in terms of any force per unit area. It is important to note that the angle of internal friction, \( \phi \) is assumed to be zero in case of clay. Cohesion or shearing strength as denoted as \( c \) or \( s \) is equal to one-half qu for pure clay. This test is not suitable for granular soils. Failure load is the load at which sample fails or the load corresponding to 15% strain whichever occurs first. The data sheet to represent stress and strain during these tests is presented in Appendix 8 (4.6).

5.10 UNI-AXIAL COMPRESSIVE STRENGTH TEST

The uni-axial compressive strength of a rock core sample shall be determined in accordance with ASTM D7012-10 Method C.
5.11 HYDRAULIC CONDUCTIVITY

This test is conducted to determine the rate of flow of water through the soil mass. It is determined as the following:

\[ \sigma_1 \]

\[ \sigma_3 = 0 \]

\[ \sigma_1 \]

Figure 5.4  UCS Loading

5.11.1 RIGID WALL METHOD

5.11.1.1 CONSTANT HEAD

A constant head test, as described in detail in AASHTO T-215 (ASTM 2434), is generally used to determine the hydraulic conductivity of granular materials. The sample for testing is selected and compacted into the mold. (The compactive efforts affect the hydraulic conductivity.) It is then saturated under vacuum to assure that there is no air in the sample. Water is then allowed to flow through the sample from the constant head tank to a collector tank. Water is continually added to the constant head tank to maintain the water level.

5.11.1.2 FALLING HEAD

A falling head test can be used to determine the hydraulic conductivity of fine-grained soils. The sample should be compacted and saturated as above for the constant head test. The constant head tank from the previous test is replaced with a burette. The difference in water level from the burette to the collector tank is measured and recorded as \( h_1 \). Water is then allowed to flow out of the burette and into the collector tank. Once a predetermined change has occurred, the head is measured again and recorded as \( h_2 \). The time required for the change in head and the temperature of the test water should also be recorded.
5.11.2 FLEXIBLE WALL METHOD

Hydraulic conductivity tests simulating various confining pressures and pressure differentials may be conducted using flexible wall parameters as per ASTM D-5084. The use of the flexible wall method (ASTM D-5084) is recommended for relatively low permeability soils \((K < 1 \times 10^{-3} \text{ cm/sec})\) and the rigid wall method (AASHTO T-215) is recommended for soils with higher permeability \((k > 1 \times 10^{-3} \text{ cm/sec})\).

5.12 TRIAXIAL COMPRESSION TEST

This test shall consist of performing the Triaxial Compression Test to determine the shear strength parameters of a given soil sample. Each test shall consist of at least three (3) points for plotting a Mohr Failure Envelope and determining the strength parameters. Test results shall include: initial and final moisture content tests, specific gravity, Atterberg Limits, initial and final void ratio, initial and final degrees of saturation, unit weight (density), visual textural description, plot of Mohr circles failure envelope and sketch of failure.

![Triaxial Test Diagram](image)

**Figure 4.5** Triaxial Tests, Step I and Step II
The test may be one of the following types.

- Unconsolidated – Undrained (UU) test, quick test.
- Consolidated – Undrained (CU) test.
- Consolidated – Drained (CD) test, slow test.

The UU and CU Triaxial tests are to be conducted in accordance with AASHTO T-296 and AASHTO T-297 test methods, respectively. These tests may be performed on all the soils ranging from cohesive to cohesionless. Consolidated drain test or Slow Test (CD) is generally conducted for earthen embankment or landfill construction.

### 5.12.1 UNCONSOLIDATED-UNDRAINED (UU)

The Unconsolidated-Undrained (UU) test is the most common test method. During this test no drainage from the sample is permitted, either during the application of the confining pressure or during the axial loading to failure. This is referred to as Total Stress Analysis.

### 5.12.2 CONSOLIDATED-UNDRAINED (CU)

The Consolidated-Undrained (CU) test is a test method in which drainage of a sample is allowed during the application of the confining pressure but no drainage is allowed during axial loading. This is referred to as Effective Stress Analysis.

### 5.12.3 CONSOLIDATED-DRAINED (CD)

Consolidated-Drained (CD) test or slow test is the method in which drainage from the sample is permitted in both conditions, during the application of the confining pressure as well as during axial loading to failure. The calculation for the triaxial tests shall be completed according to the data sheet presented in Appendices 10 (4.8) and 11 (4.9).

### 5.13 TOPSOIL TESTING

Each sample shall be prepared in accordance with the appropriate AASHTO, ASTM, ITM, and NCRR methods as detailed below to determine the following constituents: grain size distribution, organic content, pH, total phosphorus and total potassium. Soil classification testing shall be performed in accordance with AASHTO T 88 and T 89. The reported soil description shall include percents: gravel, sand, silt and clay and the AASHTO soils classification. Organic content testing (loss on ignition) shall be performed in accordance with AASHTO T 267. In Daviess, Gibson, Knox, Pike, Posey and Vanderburgh counties AASHTO T 21 testing shall be completed in addition to AASHTO T 267. The pH test shall be performed in accordance with ASTM T 289. Sample size for the pH test shall be .04 lbs (20g) of material passing No. 4 sieve. Total phosphorus and potassium shall be performed in accordance with the North Central Regional Research testing methodology. A summary of all the above testing results shall be placed in Geotechnical report.
5.14 CALIFORNIA BEARING RATIO (CBR)

This work shall consist of the determination of California Bearing Ratio values in accordance with AASHTO T-193 with the following exceptions.

1. Three (3) specimens shall be molded at optimum moisture content, one at approximately ninety (90) percent, one at ninety-five (95) percent, and one at one hundred (100) percent, of the maximum dry unit weight, respectively.

2. The samples shall be molded between –0.8 percentage to +0.8 percentage of optimum moisture content.

3. A minimum surcharge weight of 22, 26, and 30 lbs. shall be applied based on the volume of estimated traffic. However, 25 to 30 lbs. has been found acceptable.

4. California Bearing Ratio tests shall be performed on the soil which has a maximum unit weight of at least one hundred (100) pcf.

5. A dry unit weight (abscissa) versus CBR (ordinate) curve shall be plotted and furnished for each sample tested, or the graph as shown in Appendix 12 (4.10). Other data for CBR tests should be presented on the data sheet as shown in Appendix 13 (4.11).

Note: CBR test shall not be performed on the granular soil without prior approval.
5.15 RESILIENT MODULUS (Mr)

Resilient modulus of subgrade soil should be determined as per AASHTO T-307 or other procedures as specified by INDOT. For rubbilization resilient modulus of subgrade soil should be conducted on a sample molded at in-situ density. Results of Resilient Modulus Test should be presented as described in item 49(b) in Appendix 16, Services to Be Furnished by Consultant, (Appendix A). A sample of laboratory test data is presented in Appendix 13a, (4.21).