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 5.1(a). 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 defined by:

$$e = \frac{V_v}{V_s}$$

Equation (5.1)

Where: V_v = volume of voids
V_s = volume of soil solids

- Porosity (η) is the ratio of the volume of voids to the volume of the soil or,

$$\eta = \frac{V_v}{V} \times 100(\%)$$

Equation (5.2)

Where: V = total volume of soil

Moreover,

$$\eta = \frac{V_v}{V} = \frac{V_v}{V_s + V_v} = \frac{\frac{V_v}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}} = \frac{e}{1+e}$$

Equation (5.3)

- Degree of saturation (S), defined by the following, is the ratio of the volume of water in the void spaces to the volume of voids, which indicates the percentage of the total volume of voids filled with water:

$$S = \frac{V_w}{V_v} \times 100(\%)$$

Equation (5.4)

Note that completely dry soil has S = 0%, whereas fully saturated soil has S = 100%.

Weight Volume Relationship:

Total Weight = W
$W = W_a + W_s + W_w = W_w + W_s$ ($W_a \approx 0$)
W_a, W_w , *and* W_s are weights of air, water, and solids in the soil.
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_s):

$$G_s = \frac{\gamma_s}{\gamma_w}$$

Where: γ_s and γ_w are unit weights of solid and water.
γ_w = 62.4 lb/ cu ft.

- Weight of solid (W_s):

$$\begin{aligned} W_s &= \gamma_s \times V_s \\ &= G_s\gamma_s \times W_s \\ &= G_s\gamma_w \text{ (when } V_s = 1) \end{aligned}$$

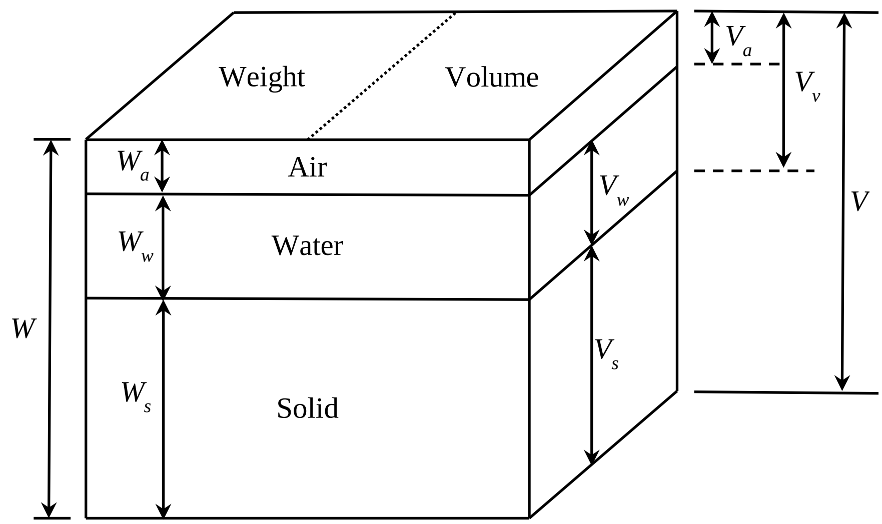
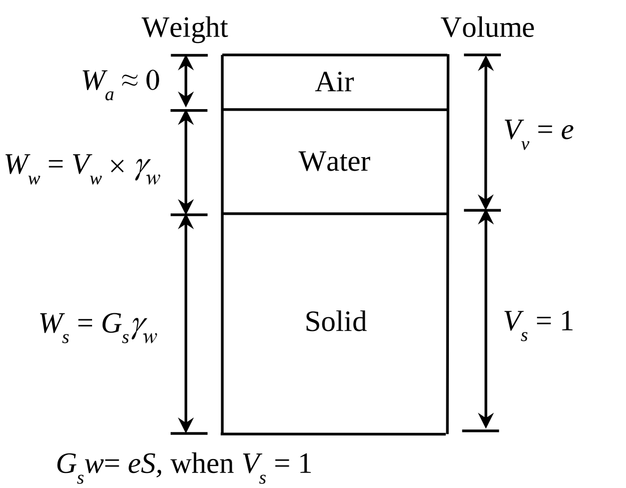
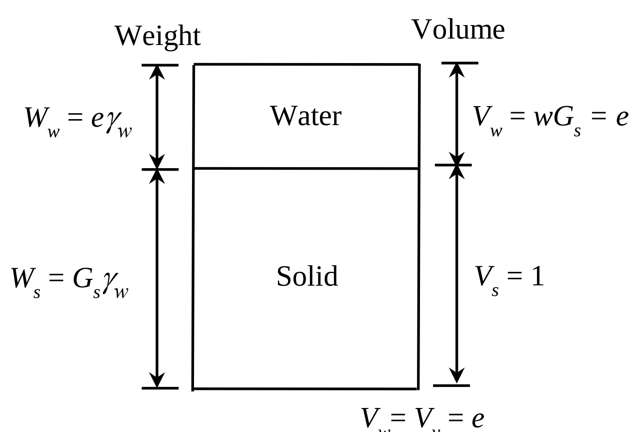


Figure 5.1 Weight-Volume Relationship
(a) Components of soils



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



(c) Weight-volume relationships, two-phase diagram

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

- Moisture or water content (w):

$$w = \frac{W_w}{W_s} \times 100(\%)$$

Equation (5.5)

- Moist (natural or bulk) density (lb/cu ft):

$$\gamma_{\text{bulk}} = \frac{M_t}{V_t} = \frac{G_s \gamma_w (1+w)}{1+e}$$

Equation (5.6)

- Dry unit weight (lb/cu ft):

$$\gamma_d = \frac{\gamma_{\text{bulk}}}{1+w} = \frac{G_s \gamma_w}{1+e}$$

Equation (5.7)

- Saturated unit weight (lb/cu ft) where the air voids are filled with water so that:

$$V_{\text{w}} = e \text{ and } M_w = e\gamma_w$$

Thus,

$$\gamma_{\text{sat}} = \frac{\gamma_w (G_s + e)}{(1+e)}$$

Equation (5.8)

- Submerged unit weight (lb/cu ft):

$$\gamma' = \gamma_{\text{sat}} - \gamma_w = \frac{\gamma_w (G_s - 1)}{1+e}$$

Equation (5.9)

5.1 MOISTURE CONTENT

This test shall consist of determination of moisture content on all fine-grained soil samples in accordance with AASHTO T-265. It is important to note that the moisture content *w* is expressed as a percentage in Equation 5.5. Moisture content is an important soil property that relates with soil behavior, clay content, organic content, calcium carbonate, shear strength, compressibility, and other engineering properties. This test may not be required on soils with less than 35% passing #200 Sieve (0.075 mm).

5.2 SPECIFIC GRAVITY TEST

Specific gravity tests of soils shall be performed in accordance with AASHTO T-100. Most Indiana soils have specific gravities ranging between 2.60 to 2.75. Soils with organic content or porous particles may have lower specific gravities. Coal combustion residuals may have higher specific gravities.

5.3 CLASSIFICATION TESTS

5.3.1 GRAIN SIZE DISTRIBUTION

The results of sieve analyses, plotted in the form of a gradation curve, are used to estimate soil permeability. The following tests shall be performed on samples obtained to verify 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 analysis is a quantitative determination of the distribution of particle sizes present in the soil sample. The testing will be accompanied by means of a hydrometer analyses. The method of determining the distribution of particle sizes in soils shall be in accordance with AASHTO T-88 and INDOT’s triangular classification chart as given in this section. Soil classification shall be in accordance with AASHTO M-145.

Sieves shall be U.S. sieve sizes: 75 mm (3 in.), 50 mm (2 in.), 25 mm (1 in.), 9.5 mm (⅜ in.), 4.75 mm (No. 4), 2.00 mm (No. 10), 0.425 mm (No. 40), and 0.075 mm (No. 200).

5.3.1.2 HYDROMETER ANALYSIS

This work shall consist of the hydrometer analysis in accordance with AASHTO T-88, which includes a determination of specific gravity in accordance with AASHTO T-100. If more than 20% of a sample 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.

Soil behavior changes when the soil’s clay content is greater than 20 % in soil matrix. Engineering judgment should be used to perform a hydrometer analysis on each predominant soil type.

5.3.2 ATTERBERG LIMITS AND PLASTICITY INDEX (PI)

The liquid limit (LL) is determined according to the AASHTO T-89 method. The plastic limit (PL) and plasticity index (PI) are determined according to AASHTO T-90. At different water contents, a fine-grained soils can exist in several states of consistency. Soil behavior can be predicted with Atterberg limits. The consistency and the behavior (i.e., the relative ease that fine-grained soils can be deformed) depend primarily upon the amount of water present in the soil-water system. Usually, soils with a higher LL contain a greater clay content. In 1911, A. Atterberg, a Swedish soil scientist, proposed the boundaries of four 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 plasticity index (PI) 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 correlate with the soil's engineering behavior, such as compressibility, permeability, shrink-swell, and strength. The shrinkage limit (SL) can be useful in predicting the maximum loss of volume. For example, an embankment material may undergo shrinkage when removed from a wet borrow and subsequently dried after 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.

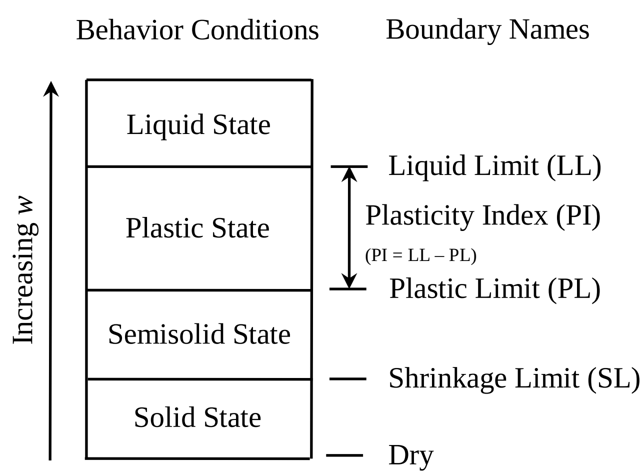


Figure 5.2 Consistency of soils as indicated by Atterberg limits

The LL, PL and SL vary for each soil, but the procedures for obtaining these values have been standardized. The LL is determined in the laboratory 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. 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. diameter. The SL is determined by drying a sample of saturated soil and measuring the limiting moisture content at which no further volume changes occur with loss of water.

The PI of A-7-5 subgroup is equal to or less than LL – 30, whereas PI of A-7-6 subgroup is greater than LL – 30. Generally, the optimum moisture content (OMC) of cohesive soils is a couple percent below the moisture content at the PL.

Please note that additional parameters of group index (GI) are determined to classify fine soils.

GI shall be calculated after performing the soil classification and Atterberg limit. Additionally, the GI is reported along with the classification test. The GI indicates the percentage and plastic nature of the portion of the material passing No. 200 sieve. Calculation of the GI is the final part of the AASHTO classification. Generally, the higher the value of the GI for a given classification, the poorer the performance of the soil. Therefore, GI relates with construction properties of a soil, such as strength, compressibility, or volume change.

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

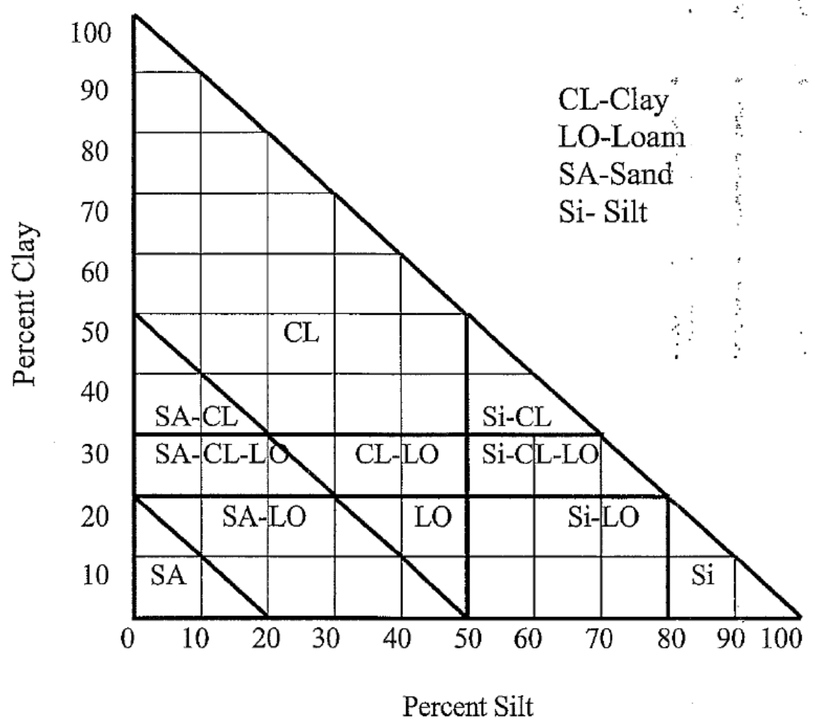
$$\mathrm{GI} = (F - 35)[0.2 + 0.005(\mathrm{LL} - 40)] + 0.01(F - 15)(\mathrm{PI} - 10)$$

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

Organic	**Granular Materials (35% Or Less Passing No. 200)**							**Silt-Clay Materials (More than 35% Passing No. 200)**				**Organic**
Group	A-1		A-3	A-2				A-4	A-5	A-6	A-7	A-8
Subgroup	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6	
Sieve Analysis, % Passing												
No. 10	50 max.	—	—	—	—	—	—	—	—	—	—	
No. 40	30 max.	50 max.	51 min.	—	—	—	—	—	—	—	—	
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.	
LL and PI of Fraction Passing No. 40												
LL	—		N.P	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	
PI	6 max.		N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.	
Types of Significant Materials	Stone Fragments, Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils		Peat
Subgrade Rating	Excellent to Good			Fair to Poor								Unsuitable

Soil Classification



INDOT Textural Soil Classification

Definitions:

Boulder	> 10 in
Cobbles	10 in - 3 in
Gravel	3 in - No. 10 Sieve
Coarse Sand	No. 10 - No. 40 Sieve
Fine Sand	No. 40 - No. 200 Sieve (0.075 mm.)
Silt	0.075 - 0.002 mm.
Clay	< 0.002 mm.
Colloids	< 0.001 mm

Soil with organic content (AASHTO T-267) 8% or less is considered a mineral soil**.**

Soils having 0 to 19% Retained on No. 10 sieve (Chart below may be used)

Classification	% Sand & Gravel	% Silt	% Clay
Sand	80-100	0-20	0-20
Sandy Loam	50-80	up to 50	up to 20
Loam	30-50	30-50	Up to 20
Silty Loam	Up to 50	50-80	0-20
Silt	0-20	80-100	0-20
Sandy Clay Loam	50-80	0-30	20-30
Clay Loam	20-50	20-50	20-30
Silty Clay Loam	Up to 30	50-80	20-30
Sandy Clay	50-70	0-20	30-50
Silty Clay	0-20	50-70	30-50
Clay	0-50	0-50	30-100

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 table above, followed by the 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.

Classification	% Gravel	% Sand	% Silt	% Clay
Gravel	85-100	0-15	0-15	0-15
Sandy Gravel	40-85	15-40	0-20	0-20
Gravelly Sand	20-40	40-80	0-20	0-20
Sand and Gravel	20-50	20-50	0-20	0-20

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. Soils with organic content should have low shear strength, low unit weight, and a high moisture content. Based on research, loss on ignition (AASHTO T 267) overestimates the organic content by a couple of points. The soil's organic content percentage relates well with moisture content, unit weight, shear strength, compressibility, etc. The Engineer should use engineering judgment while reviewing this test.

Classification	Percentage
With Organic Matter	$4 < OC^* \leq 15$
Organic Soil (A-8)	$16 \leq OC^* \leq 30$
Peat (A-8)	$OC^* > 30$

* Organic Content in Percentage

Marly Soils: Marl may be encountered below the organic soils in addition to areas where water is ponded. The following classification system shall be used for marly soils with calcium and magnesium carbonate content. The test shall be performed in accordance with sequential loss on ignition following ITM 507.

With Trace Marl	1% to 9%
With Little Marl	10% to 17%
With Some Marl	18% to 25%
Marly Soil (A-8) No group index	26% to 40%
Marl (A-8) No group index	More than 40%

ITM 507 may either overestimate marl content or involve error on the high side when soils with sand is burned off in muffle furnace. Engineers should perform the test when soils are lighter, spongy, or soils with shells. Engineer should use their engineering judgment while reviewing this test along with moisture, or unit weight. Sequential loss on ignition of soil relates well with moisture content, unit weight, shear strength, and compressibility.

As guide, the sequential loss on ignition marl test is recommended when max dry density is < 105 pcf and moisture is > 25%.

5.4 pH TEST

The pH test is performed in accordance with AASHTO T 289 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 sizes should be 0.04 lbs (20.0 grams) of material passing the No. 4 (4.75 mm) sieve. The samples shall be prepared in accordance with AASHTO T-87.

5.5 ORGANIC CONTENT (LOSS OF IGNITION)

This test shall determine the organic content in accordance with AASHTO T-267. High organic content in soils result in high moisture and low shear strength. Organic soils in foundations or compacted embankments may decompose or decay over time resulting in settlement. Hence, organic soils are undesirable in geotechnical engineering. This method will provide a quantitative estimation of oxidized organic matter in the soil mass. When all the carbon is burned such as coal or minerals, it overestimates loss on ignition. Sometimes, loess from southern Indiana will have a higher loss on ignition due to grass mixed in the soil being burned off.

Based on research, loss on ignition (AASHTO T 267) overestimates the organic content of the soils approximately 2% higher than the organic content determine by other methods. Based on the research (Huang et al., 2009), this overestimation can reach 8 % of loss of ignition. Engineering judgment should be used while reviewing this test. The organic content of soil correlates well with moisture content, unit weight, shear strength, and compressibility. Organic matter present in soil shall be reported as a percentage and rounded to the nearest whole number. For organic soils, a lower drying temperature of approximately 140° F (60° C) is recommended. Tests shall be performed in accordance with ASTM D 2216 (AASHTO T 265).

5.6 LIQUID LIMIT RATIO TEST (LLR)

The LL is defined by the lowest moisture content where the soil begins to flow as a viscus liquid. The liquid limit ratio (LLR) is defined by the LL of the burned off soil (AASHTO T 267) divided by the LL of the natural soil, as show in the equation below. Determination of the LL is performed in accordance with AASHTO T 89.

$$\text{Liquid Limit Ratio (LLR)} = \frac{\text{LL After Loss on Ignition}}{\text{LL}}$$

When the LLR is 1, the soil contains no organic material.

5.7 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.

5.8 DENSITY MOISTURE RELATIONS

5.8.1 STANDARD MOISTURE-DENSITY RELATIONS (STANDARD PROCTOR)

A minimum of 4 points on the curve with at least 2 points on each side of optimum shall be performed in accordance with AASHTO T-99. Additionally, the sample shall be mixed and then cured for 48 hours prior to molding the specimens. This test is called standard Proctor test.

Generally, this test is used to determine the maximum dry density and optimum moisture content (OMC) of soils that can be compared with density achieved during construction. Sometimes the soil sample is molded to a specific density for strength or a resilient modulus test.

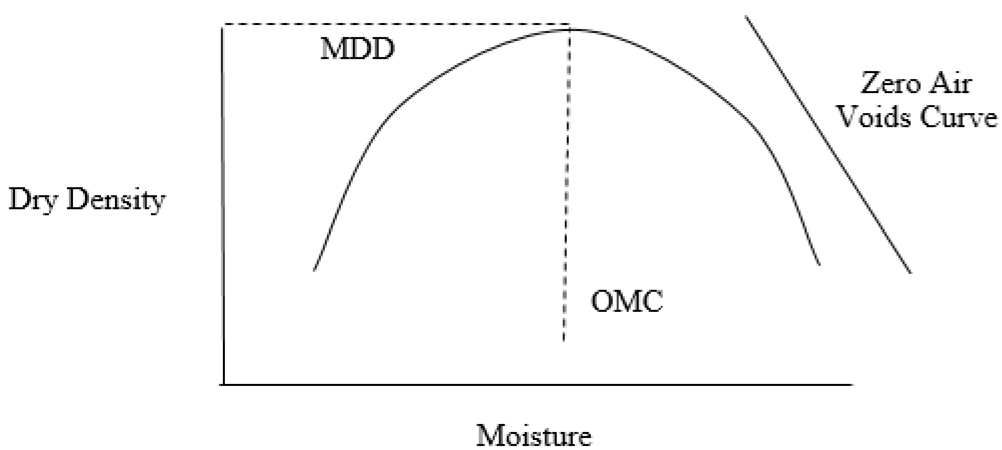


Figure 5.3 Standard Proctor compaction curve. Zero air voids curve represents 100% saturation.

Well-graded soils have higher density whereas poorly or gap graded soils has a lower density. Cohesive soils have a higher OMC, whereas granular soils have a lower OMC. It is due to a soil's specific surface, i.e. the total surface area of soil grains per volume. Cohesive soils have larger specific surface as compared to granular soil, which explains why clays exhibit higher OMC than sands.

5.8.2 MODIFIED MOISTURE AND DENSITY RELATION (MODIFIED PROCTOR)

The modified Proctor test is similar to the standard Proctor test. This test is performed in accordance with AASHTO T 180. Soils are compacted in 5 layers with hammer weighing 10 lbs and a drop height of 18 inches. The heavier hammer and increased drop height along with increased with layers significantly increases the compactive effort. The modified Proctor test results in greater compaction energy, creating higher unit weight at a lower moisture content. Higher unit weight corresponds to less void space so maximum unit weight can be achieved using less water.

Clayey and silty soils have a major improvement in densities when compacted at modified Proctor, whereas density increases are small in sands and gravels.

5.8.3 ONE POINT PROCTOR MOISTURE AND DENSITY RELATION

Soils are required to be compacted to not less than the specified construction value. If soils are mixed during construction, the soil material differs from the unique laboratory samples, which alters maximum dry density and optimum moisture. This test is performed at the construction site to obtain the corrected maximum dry density and optimum moisture for compaction control. This test is for cohesive soils only and should be performed in accordance with ITM 512.

Here are also relationship for density – moisture and Atterberg limits. Geotechnical engineering judgement should be used while using these relationships.

$$\gamma d\ max = 130.3 - 0.82\ LL + 0.3\ PI$$

$$Omax = 6.77 + 0.43\ LL - 0.21\ PI$$

These relations are helpful in estimating maximum dry density and optimum moisture content.

5.9 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. The test consists of applying a series of pressure increments on soils sample. Consolidation consists of primary and secondary consolidation. Primary consolidation is completed when water is forced out. Secondary consolidation occurs when primary consolidation is completed. The load increment should be the same as design, and the construction schedule should be reviewed. At each load settlement increment, time readings are taken for 24 hours or until the linear portion of the secondary compression appears. Consolidation means reduction of water voids that is due to the gradual transfer of load from pore water to soil grains, permeability of soils, and soil type. Foundation consolidation is important when embankments are constructed over thick, compressible, and relatively less permeable strata. Where underlying embankment soils are compressible and clayey, Shelby tube sample should be taken for consolidation testing. Engineering judgment should be used to determine sample depths and loading increments. When foundation soils are primarily granular, consolidation settlement will be small and all the settlement would likely to complete during construction.

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 3 to 5. Foundation soils with less than 18% moisture seldom experience settlement. When the moisture content reaches 22% settlement is a concern. When a consolidation test is not available, natural moisture content can be used to estimate compression index.

5.10 CONSTANT RATE OF STRAIN TEST

Other loading methods include the constant rate of strain test (ASTM D 4186) in which the sample is subjected to a constantly changing load while maintaining a constant rate of strain. Additionally, in the single-increment test (ASTM D 2435) sometimes used for organic soils, the sample is subjected only to the load expected in the field. A direct analogy is drawn between laboratory consolidation and field settlement amounts and rates.

5.11 EXPANSIVE INDEX OF SOILS

This work shall consist of determining the expansion potential of soils in accordance with ASTM D 4829.

5.12 UNCONFINED COMPRESSIVE STRENGTH TEST, q_u

This test, commonly referred to as the q_u test, 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 (σ_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. Test results may be expressed in terms of force per unit area, commonly tsf or kPa. It is important to note that the angle of internal friction, φ is assumed to be zero in case of clay. Cohesion or shearing strength, denoted as *c* or *s*, is equal to $\frac{q_u}{2}$ 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 that represents stress and strain during these tests is presented in the Appendix.

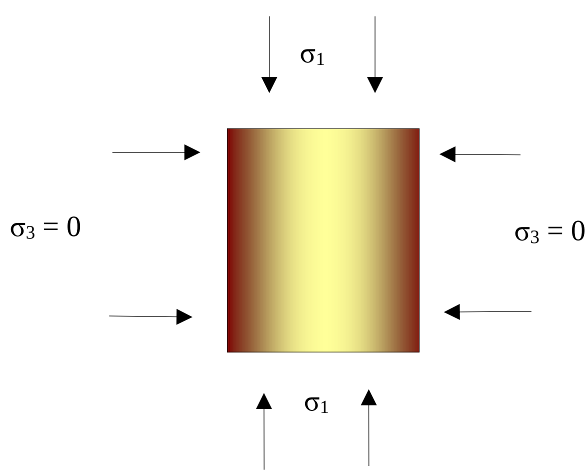


Figure 5.4 UCS Loading

5.13 UNIAXIAL COMPRESSIVE STRENGTH TEST OF ROCK, q_u

The uniaxial compressive strength of a rock core sample is an unconfined test and shall be determined in accordance with ASTM D7012-14 Method C. The unconfined compressive strength of rock is used in many design formulas and is sometimes used as an index property to select the appropriate excavation technique. Deformation and strength of rock are known to be functions of confining pressure. Tests are performed on selected pieces of rock core. Sometimes cores are cut to form a specimen of appropriate dimensions. It is important to recognize that q_u represents the compressive strength of the intact rock under condition with no confinement and may not reflect the actual strength of the rock under confined conditions.

5.14 HYDRAULIC CONDUCTIVITY

This test is conducted to determine the rate of flow of water through the soil mass to determine the drainage property of subgrade and base materials. The hydraulic conductivity is considered to be steady when four or more consecutive hydraulic conductivity (k) measurements fall within $\pm$25% of the average k value for k values greater than 1 x 10^{-8} cm/s, or $\pm$50% of the average for k values less than 1 x 10^{-8} cm/s. Since the hydraulic conductivity is defined at a temperature (*C) (*in degrees Celsius), of 20 °C, a temperature correction factor, RC, may need to be incorporated into the calculations.

5.14.1 CONSTANT HEAD

A constant head test, as detailed in AASHTO T 215, is generally used to determine the hydraulic conductivity of granular materials. The sample for testing is selected and compacted into a mold. Subsequent compactive efforts affect the hydraulic conductivity. The sample is then saturated under vacuum to assure that there is no air in the sample. Water is 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. This test is recommended for soils where k values are greater than 1 x 10^{-3} cm/sec.

5.14.2 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 tank from the constant head 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.

5.14.3 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 is recommended for relatively low permeability soils where k < 1 x 10^{-3} cm/sec and the rigid wall method (AASHTO T 215) is recommended for higher permeability soils where k > 1 x 10^{-3} cm/sec.

Typical Values of Permeability Coefficients

Soil Type	***k* (cm/sec)**
Clean gravel	1.0-100
Coarse sand	1.0-0.01
Fine sand	0.01-0.001
Silty	0.001-0.00001
Clay	Less than 0.000001

5.15 TRIAXIAL COMPRESSION TEST

The triaxial compression test is used to determine the shear strength parameters of a given soil sample. Each test shall consist of at least 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

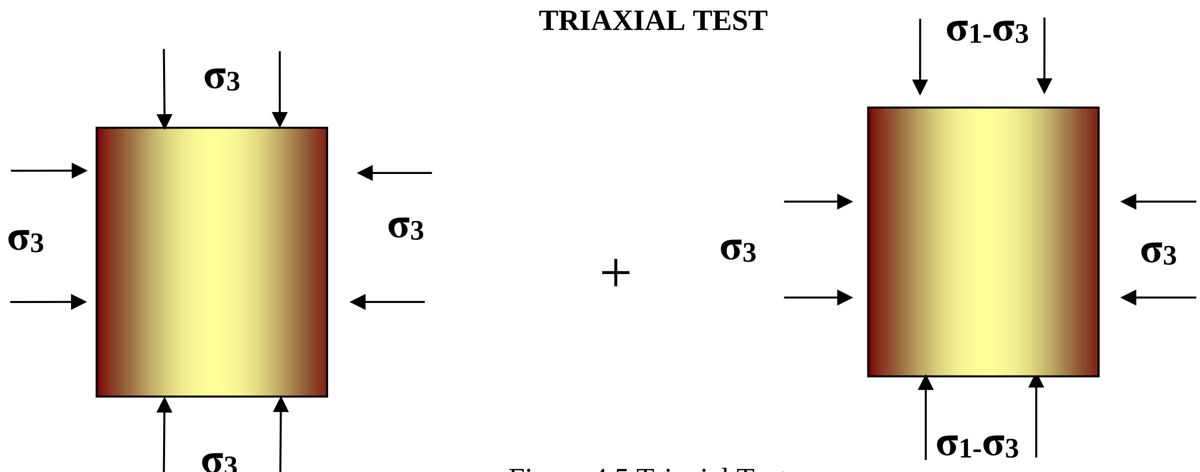
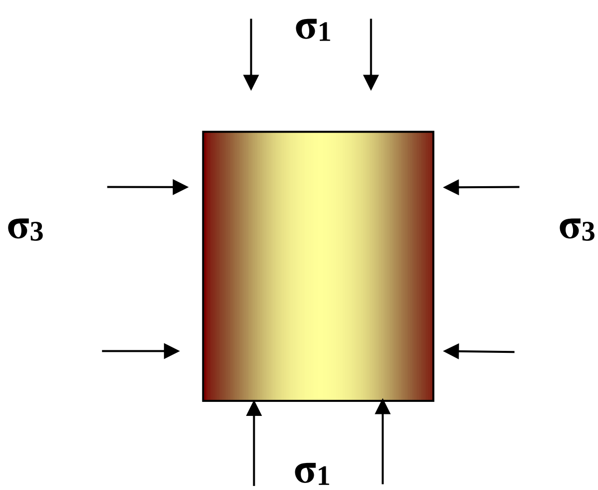


Figure 4.5 Triaxial Tests

(a) Stress Conditions in a Typical Triaxial Test

σ_1 – Vertical Stress
σ_3 – Confining Pressure
σ_1 - σ_3 – Deviator Stress



(b) Effective Stresses

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 AASTHO T 296 and AASHTO T 297 test methods, respectively. CD test, or slow test, is generally conducted for earthen embankment or landfill construction. These tests may be performed on all soils ranging from cohesive to cohesionless.

5.15.1 UNCONSOLIDATED UNDRAINED (UU)

The unconsolidated-undrained (UU) test is the most common test method, which is performed in accordance with AASHTO T 296. 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. This test is to evaluate the failure which occurs quickly such as slope stability failures during or at the end of construction. The test provides information when the embankment and foundation soils are relatively impermeable.

5.15.2 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. This test is performed in accordance with ASTM D4767. When pore water pressure measurement are made Mohr circles are adjusted for pore water pressure, which results in an effective stress analyses.

The purpose of Mohr circle preparation diagram is to determine the values of total, effective cohesion intercept, total and effective angle of internal friction.

5.15.3 CONSOLIDATED -DRAINED (CD)

The 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 the Appendices.

5.16 DIRECT SHEAR TEST

This work shall consist of determining the consolidating drained shear test of sandy soils in accordance with AASHTO T 236

5.17 RESILIENT MODULUS (MR)

Resilient modulus (MR) of subgrade soil should be determined in accordance with AASHTO T 307. MR shall be performed at 95% of the standard Proctor for reconstruction, widening, or new alignment. The specimen shall be prepared at optimum moisture content. For subgrade foundation, MR shall be performed either on remolded soils from the embankment fill or on Shelby tube samples from the subgrade foundation.

MR should be performed on Shelby tube samples for pavement improvement, pavement rubblization, and FDR projects. In situ modulus shall be performed on Shelby tube samples. MR shall also be performed on cement and hydrated lime stabilized soils when project requires stabilization. Results should be presented as described in the Appendix.

Based on the resilient modulus test, 3 regression equations to predict resilient modulus shall be provided.

$$1)\ MR = K_1 \times Pa \times (\theta/Pa)^{k2} \times (S_d/Pa)^{k3}$$
$$2)\ MR = k_1\ \theta^{k2}$$
$$3)\ MR = k_1 \times S_d \times k_2$$

Where k_1, k_2, k_3 = regression coefficients
θ= Sum of principal stresses(psi)
P_a= reference pressure = 14.5 psi
S_d=deviator stress (psi)

Note: Equation #2 should be used for granular soils.

We suggest a deviator stress of 6 psi and confining stress of 2 psi should be chosen for MR. A deviator stress such as 4 psi and confining stress of 3 psi may be selected for foundation resilient modulus. A reasonable resilient modulus for the proposed subgrade and foundation should be recommended.

5.18 SOLUBLE SULFATE TEST

Soluble Sulfate test shall be performed in accordance with ITM 510. Sulfate test shall be performed when soils are used in subgrade construction. When high sulfate soils are mixed with chemical for the purpose of modification or stabilization, there is a potential for a subgrade heaving. INDOT Standard Specifications specify the soluble sulfate limit in Section 215. Special attention shall be given in southeast and southwest areas of Indiana due to the presence of shale.

5.19 CORROSION TEST

Corrosion or electro chemical classification tests provide the engineer with quantitative information related to the aggressiveness of the soil conditions and the potential for deterioration of a foundation material. Corrosion tests include, pH, organic, resistivity, sulfate, and chlorides. Corrosion tests required when either deep foundation is proposed or any structure may come in contact with soils. INDOT standard Specifications require structural backfill to be tested for the corrosion related tests. Tests that characterize the aggressiveness of a soil environment are important for design applications that include metallic elements, especially for ground anchors comprised of high strength steel and for metallic reinforcements in mechanically stabilized earth walls.

5.19.1 SULFATE TEST

A sulfate test shall be performed in accordance with AASHTO T 290. Sulfate testing is performed as a part of corrosion testing on structural backfill for mechanically stabilized earth walls.

5.19.2 CHLORIDE TEST

This work shall consist of determining the chloride ion content in the soil in accordance with AASHTO T 291.

5.19.3 SOILS RESISTIVITY TEST

This work shall consist of determining the electric conduction potential of the surface environment. The resistivity shall be performed in accordance with AASHTO T 288.

5.20 ROCK AND SHALE TESTING

Shale undergoes degradation when exposed to air and water. These tests are used to define behavior when it is subjected to drying and wetting conditions.

5.20.1 POINT LOAD STRENGTH INDEX OF ROCKS

This work shall consist of determining point load strength index of rocks in accordance ASTM D 5731 and it is used to classify the rock strength. The point load strength test is an appropriate method used to estimate the unconfined compressive strength of rock in which both core samples and fractured rock samples can be tested. The point load test provides an index value for the strength. To calibrate the results do a limited number of uniaxial compressive tests on prepared core samples. The point load test is also used with other index values to assess degradation potential of shales.

5.20.2 COMPRESSIVE STRENGTH AND ELASTIC MODULUS OF INTACT ROCK

The unconfined compressive strength of an intact rock core can be evaluated accurately using ASTM D 7012. The rock core specimen is cut to length so that the length to diameter ratio is 2.5 to 3.0 and the ends of the specimen are machined flat. The unconfined compressive strength of the specimen is calculated by dividing the maximum load.

The elastic modulus of intact rock test shall consist of determining the strength of intact rock core specimens in uniaxial (compressive strength) and triaxial (elastic modulus) in accordance with ASTM D 7012. This test is performed similarly to the unconfined compressive test, except that deformation is monitored as a function of load. This test is performed when it is necessary to estimate both elastic modulus and Poisson’s ratio of intact rock core.

5.20.3 SLAKE DURABILITY INDEX OF SHALE

This test determines the slake durability index of shale or similar rock in accordance with ASTM D 4644. It is used to estimate qualitatively the durability of weak rocks through weakening and disintegration resulting from two standard cycles of wetting and drying in the service environment and is used to assign quantitative durability index values to weak rocks.

5.20.4 JAR SLAKE TEST

This test shall be performed in accordance with ITM 511. It is used to determine the reaction of weak rock material to water during a certain period of time. It indicates the porosity, grains, interactions, and density of the material. The total index is observed for 10 minutes followed by an additional 20 minutes. Any reaction of the rock sample takes place primarily during this time period. The jar slaking test was found to be more suitable to measure the durability of weak rocks.

5.21 TOPSOIL AND PLANT GROWTH LAYER TESTING

Each sample shall be prepared in according with the appropriate AASHTO, ASTM, ITM, and North Central Regional Research (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 the AASHTO soils classification and percent: gravel, sand, silt and clay. Organic content testing (loss on ignition) shall be performed in accordance with ASTHO 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 0.04 lbs (20g) of material passing the 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 the geotechnical report.

5.21.1 PHOSPHORUS

This test shall consist of determining phosphorus in the topsoil and shall be performed in accordance with North Central Regional Publication 221, Chapter 6, Mehlich 3 Data Bray P equivalent. Test results shall be reported in ppm.

5.21.2 POTASSIUM

This test shall consist of determining potassium in the topsoil and shall be performed in accordance with the North Central Regional Research (NCRR) Publication 221, Chapter 7. Test results shall be reported in ppm

5.21.3 LOSS ON IGNITION

This test shall be performed in accordance with AASHTO T 267 and AASHTO T 21.

5.21.4 pH

This test shall be performed in accordance with AASHTO T 289.

Topsoil Summary			
Requirement	**Measurement**	**Test Results**	**Test Method**
pH	---		AASHTO T 289
Clay	Weight (%)		AASHTO T 88 and T 89
Silt	Weight (%)		AASHTO T 88 and T 89
Sand	Weight (%)		AASHTO T 88 and T 89
Organic Material	Weight (%)		AASHTO T 267 and AASHTO T 21
Phosphorus	ppm		North Central Regional Research Publication 221, Chapter 6, Mehlich 3 data
Potassium	ppm		North Central Regional Research Publication 221, Chapter 7