3-2.0 SUBGRADE SOILS (ROADBED SOILS)

Subgrade soils consist of the in-situ, prepared, and compacted soils below the pavement structure that affect the structural design.

3-2.01 FIELD AND LABORATORY TESTING

Materials and construction personnel must be able to classify soil encountered in the field with a reasonable degree of accuracy in order to schedule investigations, sample material, and direct earthwork processes. These relatively simple field classifications may then be verified or refined with laboratory testing.

3-2.01.01 FIELD IDENTIFICATION

Identification of soil types in the field, which is typically limited to an estimate of texture, plasticity, and color, is normally done without the benefit of major equipment, supplies, or time. It is necessary for a general assessment of sites during field reconnaissance activities and during the initial phases of more detailed work, such as the investigation of an emergency remediation or a planned geotechnical or pavement survey (Section 4-1.02.03). It may, in some instances, be the only effort ever expended towards classifying the encountered soils, but in most cases it will serve as an aid in assigning more detailed or elaborate laboratory tests.

With increased experience, field personnel should become more competent and skilled in accurately classifying the encountered soils based solely on field techniques. <u>Regardless of experience level, however, laboratory testing should be performed whenever possible to validate and sharpen the field technician's ability.</u>

1. Texture. The following methods may be used in the field to estimate the soil's texture, which is defined as the relative size and proportion of the individual grains in a given soil type.

    a. Visual Examination. By carefully examining the soil, it can be divided into its gravel, sand, and fine (silt and clay combined) components. Silt and clay particles cannot be visually separated without further magnification because the naked eye can only distinguish particle sizes down to about 0.05 millimeters (0.002 inches).

       The examination is performed by drying a sample, spreading it on a flat surface, segregating it into its various components, and estimating the relative percentage of each. The percentage refers to the dry weight of each soil fraction, as compared to the dry weight of the original sample. Table 3-2.1 provides the defined particle sizes for each component and a common reference to aid in identifying the various particle sizes.

    b. Sedimentation/Dispersion. This test is done by shaking a portion of the sample into a jar of water and allowing the material to settle. The material will settle in layers. The gravel and coarse sand will settle almost immediately, the fine sand within about a minute, the silt within an hour, and the clay will remain in suspension indefinitely. The percentage of each component is estimated by comparing the relative thickness of each of the layers in the bottom of the jar, keeping in mind that the larger sized particles will typically settle into a denser mass than the fines.

Table 3-2.1. Visual grain-size identification.

Classification		Approximate Size Limits: Measured mm (in.)	Approximate Size Limits: Sieve mm (in.)	Comparison Example
Boulder		Over 75 mm (3 in.)	> 75 mm (3 in.)	Grapefruit
Gravel				
	Coarse	75mm – 25mm (3 in. - 1 in.)	75mm – 25mm (3 in. - 1 in.)	Lemon
	Medium	25mm – 9.5mm (1 in. - 3/8 in.)	25mm – 9.5mm (1 in. - 3/8 in.)	Diameter of penny
	Fine	9.5mm – 2.0mm (3/8 in. - 1/16 in.)	9.5mm – No. 10 (3/8 in. - No. 10)	Pencil diameter to pea or rock salt
Sand				
	Coarse	2.0 mm - 0.42 mm (0.0066 ft. - 0.0014 ft.)	No. 10 - No. 40	Broom straw diameter to sugar or table salt
	Fine	0.42 mm - 0.075 mm (0.0014 ft. – 0.0002 ft.)	No. 40 - No. 200	Human hair diameter to powdered sugar
	Silt and Clay	< 0.075 mm (<0.0002 ft.)	< No. 200	Cannot be discerned with the naked eye

2. Plasticity. The ability to be molded within a certain range of moisture contents is termed plasticity. It is dependent upon the percentage and type of clay component, and it therefore requires differentiation between silt (non-plastic fines) and clay (plastic fines). Quantitatively, a soil's plasticity is defined by its Atterberg limits, which are discussed in Section 4-2.06.02 and used in the AASHTO and Unified classification systems (Sections 3-2.02.02 and 3-2.02.03, respectively). The following methods can be used in the field to approximate the plasticity of particles that would pass a No. 40 sieve (approximately 0.4mm (1/64 in.)). For field classification purposes, remove by hand the coarser particles that interfere with the tests.

a. Ribbon/Thread. In the ribbon/thread test, a moist roll of soil approximately 12 to 19mm (one-half to three-quarters inch) in diameter and 75 to 125 mm (three to five inches) long is pressed between the thumb and index finger to form a ribbon about 3 mm (one-eighth inch) thick. The longer the ribbon that can be formed before the soil breaks under its own weight, the higher the soil's plasticity. Highly plastic clays can be ribboned to 100 mm (four inches) longer than their original cast. Clays of low plasticity can be ribboned only with some difficulty into short lengths, while non-plastic materials cannot be ribboned at all. Table 3-2.2 relates the results of this test to particular Triangular Textural Classifications, which are discussed in Section 3-2.02.01.

b. Dry Strength/Breaking. The dry strength/breaking test is normally made on a dry pat of soil about 12mm (one-half inch) thick and about 30 mm (1-1/4 inches) in diameter that has been allowed to air dry completely. Attempts are made to break the pat between the thumb and fingers, with very highly plastic clays being resistant to breakage or powdering and highly plastic clays being broken with great effort. Caution must be exercised with highly plastic clays to distinguish between shrinkage cracks, which are common in such soils, and a fresh break. Clays of low plasticity can be broken and powdered with increasing ease.

Silty fine sands and silts both have a very small dry strength; they can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty where as a typical silt has the smooth feel of flour.

c. Shaking/Dilatency. In the shaking/dilatency test, a pat of soil with a volume of about 8 cubic centimeters (0.5 cubic inches) is moistened to a putty-like state and placed in the palm of the hand. The hand is then shaken vigorously or jarred on a table or other firm object. If the sample's surface begins to glisten, it is an indication that moisture within the sample has risen to the surface. When this does not occur, the soil is probably clayey. Where this occurs sluggishly or slowly, the soil is predominantly silty, perhaps with a small amount of clay. For silts or very fine sands, the moisture will rise to the surface rapidly and the test can be repeated over and over by remolding and reshaking the pat.

Table 3-2.2. Probable classification based on plasticity.

Plasticity	Length of Ribbon, mm (in.)	Mn/DOT Triangular Textural Classification
Non-plastic	0	Gravel
	0	Sand
	0	Loamy Sand
Slightly plastic to plastic		Sandy Loam
	0 – 12.5mm (1/2”)	Slightly Plastic
	12.5 – 25mm (1/2” –1”)	Plastic
	6.25 – 25mm (1/4”–1”)	Loam
	0 – 25mm (1”)	Silt Loam
Plastic to highly plastic	25 – 50mm (1”–2”)	Sandy Clay Loam
	25 – 50mm (1”–2”)	Clay Loam
	25 – 50mm (1”–2”)	Silty Clay Loam
Highly plastic	> 50mm (2”)	Sandy Clay
	> 50mm (2”)	Silty Clay
	> 50mm (2”)	Clay

3. Organic Content. Generally, a sample can be adequately classified relative to organic content based on smell and feel. Organic soils have a distinctive musty or foul odor that is enhanced in warm or fresh samples. In addition, they may have the feel of fresh to decomposed vegetable matter. Organic soils are generally undesirable in the highway subgrade and are most often excavated and wasted. See Table 2-1.1 for classification of organic materials.

3-2.01.02 LABORATORY TESTING

In order to properly classify a soil, its texture (sieve and hydrometer analyses), plasticity (Atterberg limits), and general organic content are required. These properties can be estimated in the field, as described above, or they can be developed more precisely through laboratory testing as described in Section 4-2.06. The Atterberg limits and indices are described in Section 3-2.03.01.

The amount of testing to be performed depends upon the complexity of the stratigraphy, the experience level of the field personnel describing the obtained samples, and other factors. Obviously, the more complex the stratigraphy, or the less experienced the technician, the more the need for laboratory testing to adequately describe the encountered conditions and verify the field classifications.

3-2.02 CLASSIFICATION SYSTEMS

The purpose of any classification system is to categorize soils by relating their appearance and behavior with previously established engineering properties and performance. Attributes of a good classification system include simplicity, reproducibility under variable conditions, and applicability to all soils likely to be encountered. A good system should make distinctions of practical importance to local designs.

Mn/DOT, as well as many other state highway departments and agencies focused on subgrade performance, uses the triangular textural and AASHTO classification systems to categorize soils. These systems are described in detail in the following two sections. Most private consultants use the Unified Soil Classification System, which is also briefly described below, to categorize soils accordingly to their engineering characteristics. A rough correlation between these three classification systems is provided at the end of this section.

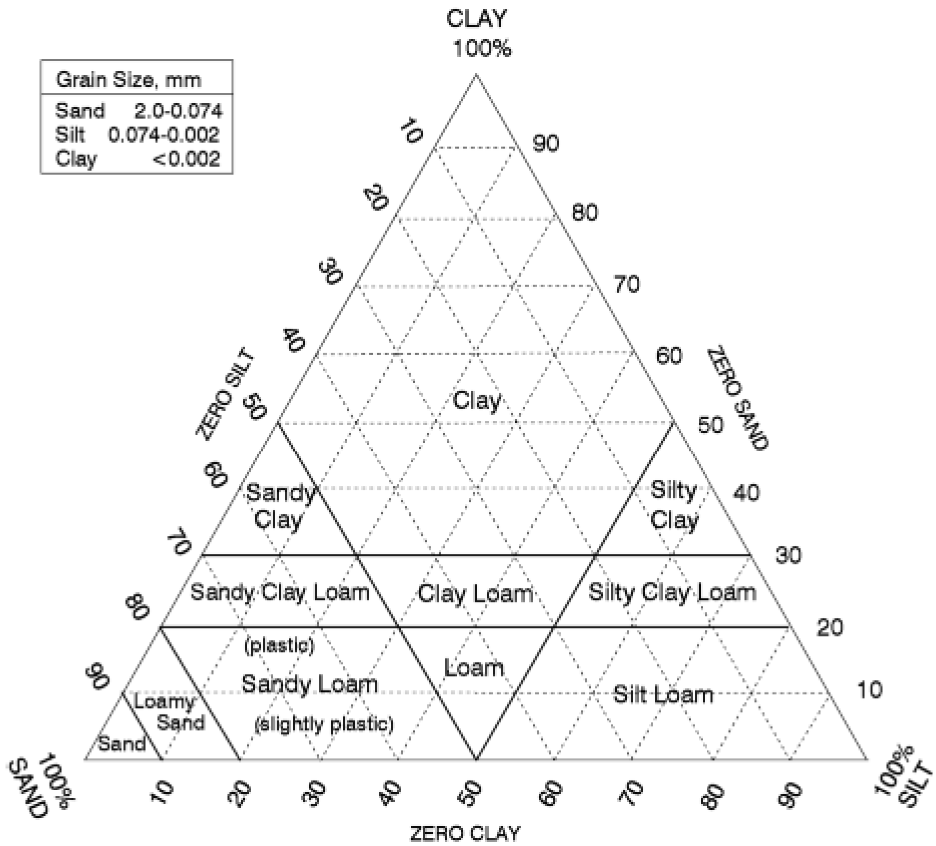
3-2.02.01 TRIANGULAR TEXTURAL

Textural systems of classification, which are solely based on a soil's texture or grain size distribution, have been developed by many different engineers. Some of these systems were developed by the Bureau of Soils (1890 - 95), Atterberg (1911), MIT (1931), and the U.S. Department of Agriculture (1938). The latter system is still in use, although it has been slightly modified by several highway departments, including Mn/DOT, to better differentiate between local soils.

The following procedure is used to categorize a soil within Mn/DOT's triangular textural classification system. First, the sample's gradation must be determined by percentage of each of the following components. Sand is between 2.0 millimeters (No. 10 sieve) and 0.075 millimeters (No. 200 sieve) in size, silt is between 0.075 and 0.002 millimeters in size, and clay is smaller than 0.002 millimeters (two microns) in size. Gravel and other stones larger than 2.0 millimeters are disregarded. Next, the percentage of sand, silt, and clay in the sample is entered into the diagram shown in Figure 3-2.1. The soil classification is determined by locating the point of intersection of the gradation.

In addition to the twelve possible classifications shown on Figure 3-2.1, gravel, defined as smaller than 75 m (three inches) in diameter and larger than 2.0 millimeters (No. 10 sieve) in size, is an acceptable classification. Any sample with more than about 25 percent gravel should include the term "gravelly" as a descriptor. Other modifiers to the textural classification should be used with restraint but are permissible if the result of their use is clear and beneficial.

Figure 3-2.1. Triangular textural classification chart.



Three examples of obtaining the proper classification of a soil using the triangular textural classification chart (Figure 3-2.1) are given below:

Example 1. What is the classification of a soil sample with 18% sand, 58% silt, and 24% clay?

Entering the left axis at 18%, the bottom axis at 58%, and the right axis at 24%, and moving to the intersection point, the soil's classification is Silty Clay Loam.

Example 2. What is the classification of a soil sample with 47% sand, 32% silt, and 21% clay?

Entering the left axis at 47%, the bottom axis at 32%, and the right axis at 21%, and moving to the intersection point, the soil's classification is Clay Loam.

Example 3. What is the classification of a soil sample with 32% gravel, 38% sand, 22% silt, and 8% clay?

Since the sample contains 32% gravel and the classification chart is based on the portions of the sample consisting of sand, silt and clay, the sample proportions need to be adjusted so that the non-gravel portions equal 100 percent. The adjusted proportions should be 56% (or 38% divided by 0.68) sand, 32% (or 22% divided by 0.68) silt, and 12% (or 8% divided by 0.68) clay.

Entering the left axis at 56%, the bottom axis at 32% and the right axis at 12%, and moving to the intersection point, the soil's classification is plastic sandy loam. Since more than 25% of the soil sample is gravel, the term "gravelly" is added to the description, resulting in a classification of plastic, Gravelly Sandy Loam.

A brief description of each of the acceptable classifications, along with additional discussion, has been provided in Section 5-692.600 of Mn/DOT's Grading and Base Manual. To increase the ready usability of this manual, these descriptions are summarized as follows:

Gravel (G): A combination of stones that will pass a 75mm (3") sieve and be retained on a 2mm (3/8") sieve. Fine Gravel (FG) has a predominance of stones between the 9.5mm (No. 10) and 2mm (No. 10) sieves. These materials can be classified by visual inspection. The AASHTO classification is A-1-b.

Sand (S): 100% of this material will pass a 2mm (No. 10) sieve and will have less than 10% silt and clay combined. It will not form a ribbon and will barely hold a cast when moist. It will generally fall in the A-1-b group.

Coarse Sand (CrS): The predominant size is material that will pass a 2mm (No. 10) sieve and be retained on a 425Fm (No. 40) sieve. AASHTO classification, A-1-a.

Fine Sand (FS): The predominant size is material that will pass the 425Fm (No. 40) sieve and be retained on a 75Fm (No. 200) sieve. AASHTO classification, A-1-b or A-3.

Loamy Sand (LS): 100% of this material will pass a 2mm (No. 10) sieve and will contain between 10 and 20% fines (silt and clay). This material is loose and granular when dry and the individual grains can be seen and felt. When moist, it will form a relatively stable cast, but because it is non-plastic it cannot be pressed into a ribbon. Loamy Sand can be further classified as Loamy Coarse Sand (LCrS), Loamy Fine Sand (LFS), or Loamy Very Fine Sand (LVFS). These soils will be generally classified as A-2-4 or A-2-5, but may be classified as A-3 or A-1-b.

Sandy Loam (SL): This soil contains 20% to 50% silt and clay combined, but less than 20% clay. It must always contain 50% or more sand grains to be classified as sandy loam. Sandy loam is divided into two main groups, slightly plastic and plastic sandy loam.

Slightly Plastic Sandy Loam (sl pl SL): This soil usually contains less than 10% clay and will form a thin ribbon 0 19mm (0-3/4") in length before breaking under its own weight. AASHTO classifications A-2-4, A-2-6 or A-2-7 are the most common.

Plastic Sandy Loam (pl SL): This soil usually contains 10% to 20% clay. It feels gritty and can be pressed into a ribbon form 19mm (0-3/4") to 25mm (1") in length. The AASHTO classifications are usually A-2-4 or A-2-5 with a group index of 0, or A-2-6 or A-2-7 with a group index of 1-13.

Loam (L): Loam is a relatively even mixture of sand and silt with less than 20% clay. It has a somewhat gritty feel but is smoother than a sandy loam. It will form a ribbon 5mm (1/4") to 37.5mm (1 1/2") in length and will be thinner and stronger than can be formed with sandy loam. This soil is best classified as A-4 in the AASHTO system.

Silt Loam (SiL): Silt Loam contains more than 50% silt and less than 20% clay. When pressed between the fingers, it will offer little resistance to pressure and feel smooth, slippery, or "velvety". Silt Loam is classified as slightly plastic when the ribbon length is between 0 and 19 mm (0 and 3/4") and plastic when the ribbon length is between 19mm (3/4") and 37.5mm (1 1/2"). Silt Loams often have high moisture contents due to their capillary affinity for water, and are classified under AASHTO Classification A-4.

Silt (Si): A soil that contains more than 80% silt is classified as a Silt. These soils have a slippery feel similar to the Silt Loam, however, they are non-plastic and form only short ribbons 0-10 mm (0-1/2") in length depending on the clay content. Silts can also be classified under AASHTO A-4.

Clay Loam (CL): A Clay Loam is a relatively even mix of sand and silt with 20% to 30% clay. Due to its significant clay content it will, with some difficulty, form a ribbon from 37.5mm (1 1/2") to 62.5mm (2 1/2") in length before breaking. Clay Loams are best classified as AASHTO A-6.

Silty Clay Loam (SiCL): A Silty Clay Loam contains 20% to 30% clay, 50% to 80% silt and 0 to 30% sand. This is a fine textured soil and will form a ribbon 37.5mm (1 1/2") to 62.5mm (2 1/2") in length without breaking. It does not offer as much resistance to pressure as a Clay Loam, but it more slippery. Silty Clay Loams appear in pockets and tend to have a dull appearance. The corresponding AASHTO classification is most often A-6, but more elastic Silty Clay Loams may be A-5.

Sandy Clay Loam (SCL): A Sandy Clay Loam contains 20% to 30% clay, 50% to 80% sand and 0 to 30% silt. It feels gritty compared to other Clay Loams due to its large sand content. It will form a ribbon 37.5mm (1 1/2") to 62.5mm (2 1/2") in length. The corresponding AASHTO classification is A-6.

Clay (C): Clay contains 30% to 100% clay, 0 to 50% silt, and 0 to 50% sand. It is smooth and shiny and will, with considerable difficulty, form a long, thin, flexible ribbon 62.5mm (2 1/2") or more in length. The AASHTO classification is A-7.

Silty Clay (SiC): Silty Clay contains 30% to 50% clay, 50% to 70% silt, and 0 to 20% sand. It is very plastic, but feels smooth and slippery. It will form a ribbon 62.5mm (2 1/2") or more in length. Silty Clay is generally found in small pockets, rather than as an extensive layer. The AASHTO classification is A-7 or A-7-5.

Sandy Clay (SC): Sandy Clay contains 30% to 50% clay, 50% to 70% sand and 0 to 20% silt. It is very plastic, but has a gritty feel. It will form a ribbon 62.5mm (2 1/2") or more in length. A true Sandy Clay is rarely found in Minnesota. The AASHTO classification is A-7 or A-7-6.In general, gravel and coarser sands are excellent for upper embankment materials; finer sands, loamy sand, sandy loam, and loam are excellent to good; clay loam and sandy clay loam are good to fair; silt loam and silty clay loam are fair to poor; and sandy clay, clay, and silty clay are poor.

3-2.02.02 AASHTO

In 1928, the Bureau of Public Roads introduced a classification system with eight soil groups, designated A-1 through A-8, to be used for assessing the suitability of road subgrade materials. Major revisions to the system, most recently in 1987, have resulted in the chart shown in Table 3-2.3. This system is based on the proportion of grain diameters falling between 2.0, 0.425, and 0.075 mm (sieve Nos. 10, 40, and 200) as well as the soil's plasticity. It is a quick, rational method for categorizing both undisturbed natural soil and fill in terms of its performance as a subgrade material. The system has been found to be applicable in areas with vastly different soil types and origins. In addition to the seven classifications shown in Table 3-2.3, an eighth classification, Group A-8, has been added to include highly organic soils (peat or muck).

Table 3-2.3. AASHTO classification of soils and soil-aggregate mixtures (from AASHTO M 145-91).

CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES

General Classification	Granular Materials (35% or less passing 75µm) [No. 200]							Silt-Clay Materials (More than 35% passing 75µm) [No. 200]			
Group Classification	A-1		A-3*	A-2				A-4	A-5	A-6	A-7
	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:											
Percent passing:											
2mm (No. 10)	50 max.	---	---	---	---		---	---	---	---	---
425µm (No. 40)	30 max.	50 max.	51 min.	---	---	---	---	---	---	---	---
75µm (No. 200)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 425µm (No. 40):											
Liquid Limit	---		---	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity Index	6 max.		N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min**
Usual Types of Significant Constituent Materials	Stone Fragments Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General Rating as Subgrade	Excellent to Good							Fair to Poor			

*The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate the superiority of A-3 over A-2.
**The plasticity index of A-7-5 is equal to or less than the liquid limit minus 30. The plasticity index of the A-7-6 subgroup is greater than the liquid limit minus 30.

There are three broad types under which the AASHTO groups and subgroups are divided. These are "granular" (A-1, A-3, and A-2), "silt-clay" (A-4 through A-7), and highly organic (A-8) materials. The transitional group, A-2, includes soils which exhibit the characteristics of both granular and silt-clay soils, making subdivision of the group necessary for adequate identification of material properties. A more detailed discussion of the AASHTO groups is included in Section 5-692.606 of Mn/DOT's Grading and Base Manual.

The engineering considerations for granular and silt-clay soils are significantly different. The following discussion highlights major differences between these two types.

1. Granular. **Granular** materials include mixtures of rock fragments ranging from fine to coarse grained. Granular materials may include a non-plastic to slightly plastic soil binder, but are limited to 35 percent or less of the soil passing the 0.075mm (No. 200) sieve (Note that Mn/DOT's Specification 3149 limits granular backfill to no more than 20

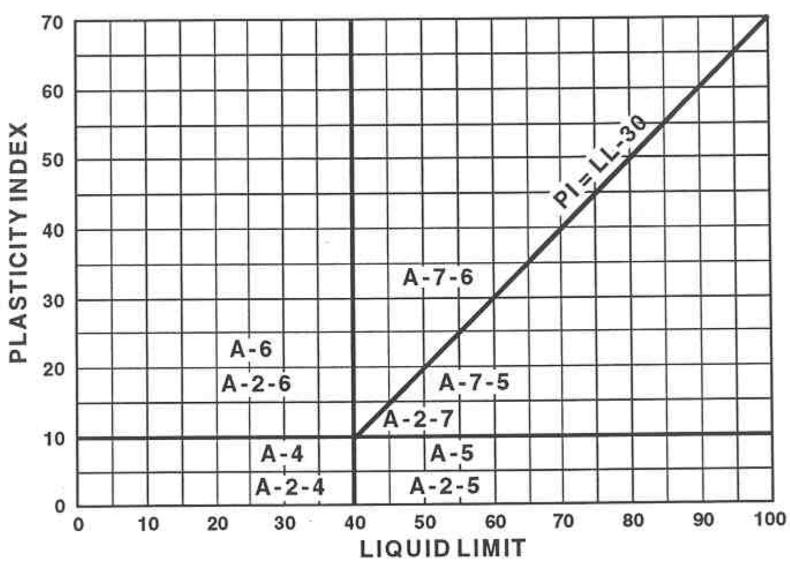
percent passing the 0.075mm (No. 200) sieve). Granular materials generally provide the most desirable subgrade.

It is possible, however, that some granular materials near the silt-clay boundary may have characteristics unsuitable for roadways in the presence of water. This is because capillarity (or a chemical affinity for water) may induce a volume change or softening of the material. In addition, frost heave becomes a concern in materials with high silt contents. Therefore, the elevation of the ground water table should be carefully considered when the subgrade is composed of these transitional soils

2. Silt-clay. **Silt-clay** materials are soils having more than 35 percent passing the 0.075mm (No. 200) sieve. The behavior of these soils is dominated by the fines in the soil mass. Silt-clay materials (A-4 through A-7) can provide suitable road subgrades when their shortcomings are accounted for by proper design or construction practices. Subgrades classified as A-6 or A-7 usually dictate a thickened pavement section and strictly maintained grading tolerances. A-7 materials are generally considered the poorest performers with regard to roadway construction.

Determining the AASHTO classification of a soil is a two-step process. First, the soil is categorized into one of the eight major "A" groups using the gradation limits set in Table 3-2.3. Generally, the lower-numbered soils to the left of the chart are more preferable subgrade materials than those on the right. However, this is not always true: A-3 materials usually out-perform A-2 materials. A subdivision of some of the major groups is necessary to account for varying characteristics, e.g. A-2-6 and A-2-7. These classifications can be checked graphically using Figure 3-2.2.

Figure 3-2.2. Relationship between liquid limit and plasticity index for silt-clay groups (from AASHTO M 145-91).



Two examples of obtaining the proper classification of a soil using the AASHTO system (Table 3-2.3) are given below:

Example 1. What is the classification of a soil sample with 75% passing the 2.0mm (No. 10) sieve, 55% passing the 0.425mm (No. 40) sieve, and 12% passing the 0.075mm (No. 200) sieve, a liquid limit of 20, and a plasticity index of 4?

Start at the left of Table 3-2.3 and move to the right. The soil is granular because 35% or less passes the 0.075mm (No. 200) sieve. The soil is not an A-1-a because 50% or more passes the 2.0mm (No. 10) sieve, not an A-1-b because 50% or more passes the 0.425mm (No. 40) sieve, and not an A-3 because 10% or more passes the 0.075mm (No. 200) sieve. However, it meets all of the requirements of an A-2-4 because 35% or less passes the 0.075mm (No. 200) sieve, its liquid limit is 40 or less, and its plasticity index is 10 or less. The soil should be classified as an A-2-4.

Example 2. What is the classification of a soil sample with 100% passing the 2.0mm and 0.425mm (Nos. 10 and 40) sieves, 72% passing the 0.075mm (No. 200) sieve, a liquid limit of 45, and a plasticity index of 25?

Start at the left of Table 3-2.3 and move to the right. The soil is a silt-clay because 36% or more passes the 0.075mm (No. 200) sieve. The soil is not an A-4 because its liquid limit is 40 or more, not an A-5 because its plasticity index is 10 or more, and not an A-6 because its liquid limit is 40 or more. However, it meets all of the requirements of an A-7 because 36% or more passes the 0.075mm (No. 200) sieve, its liquid limit is 41 or more, and its plasticity index is 11 or more. Furthermore, the soil should be classified as an A-7-6 because its plasticity index (25) is larger than its liquid limit minus 30 (15).

The subgrade quality of silt-clay soils can vary from poor to good within each major group. Therefore, a group number (G.I.) is added to the group symbol found in Table 3-2.3to indicate the plastic properties of the fines passing the 0.075mm (No. 200) sieve. Calculation of this group index is the second and final part of the AASHTO classification. Generally, the higher the value of the group index for a given group classification the poorer the performance as a subgrade material. Therefore, a group index of zero (0) indicates a "good" subgrade material and a group index of 20 or more indicates a "poor" subgrade material.

The formula used to compute the group index is

$$G.I. = (F - 35)\,[0.2 + 0.005\,(LL - 40)] + 0.01\,(F - 15)\,(PI - 10) \qquad \text{Eq. 3-2.1}$$

where

G.I.	=	group index, reported as a positive whole number or zero
F	=	percentage passing the 0.075mm (No. 200) sieve, expressed as a whole number (This percentage is based only on the material passing the 75mm (three-inch) sieve)
LL	=	liquid limit
PI	=	plasticity index

Note that only the second term, which accounts for the effect of the plasticity index, is used for the group classifications of A-2-6 and A-2-7.

The group index is added in parenthesis after the group symbol, i.e., A-4(5) or A-7-5(17), etc. Two examples are given below:

Example 1. What is the complete classification of an A-7-5 with 80% passing the 0.075mm (No. 200) sieve, a liquid limit of 90, and a plasticity index of 50?

$$\text{The G.I.} = (80 - 35)\,[0.2 + 0.005\,(90 - 40)] + 0.01\,(80 - 15)\,(50 - 10) = 46.$$

Therefore, the complete classification is A-7-5(46).

Example 2. What is the complete classification of an A-2-7 with 30% passing the 0.075mm (No. 200) sieve, a liquid limit of 50, and a plasticity index of 30.?

Using only the second term in Equation 3-2.1, the G.I. = 0.01 (30 - 15) (30 - 10) = 3.

Therefore, the complete classification is A-2-7(3).

The influence of fine content, plasticity, and liquid limit on group index is shown graphically in AASHTO M145-91.

The following descriptions provide profiles of each of the groups within the AASHTO classification system shown in Table 3-2.3:

Group A-1 includes well-graded gravel through fine sand with little or no non-plastic binder. Subgroup A-1-a includes stone fragments and gravel, with or without fines. Subgroup A-1-b includes predominantly coarse sand with or without fines. When properly placed and compacted, these materials perform well as road subgrades, as they are free draining and possess ample strength when properly placed.

Group A-3 is mostly poorly graded fine sand with few fines. Typical examples include blow sand, some beach sands, or poorly graded stream or river sand with minimal gravel content. A-3 soils are relatively free draining and possess desirable strength characteristics, but they may be somewhat difficult to compact due to their uniformity.

Group A-2 consists of transitional granular materials, all of which have less than 35 percent fines. Subgroups A-2-4 and A-2-5 have fines that are silty (non-plastic). Subgroups A-2-6 and A-2-7 have fines that are similar to A-6 or A-7 soils; that is, the fines are more plastic. A-2 soils, usually having group indices up to four, may range from good to fair as road subgrade. Frost susceptibility begins to be a problem in the A-2 soils, especially where the water table is in proximity to the zone of yearly frost depth.

Group A-4 soils are non-plastic to moderately plastic silts. Sand and gravel contents can range up to 64 percent. Group indices usually range up to eight, with lower values indicative of higher gravel and/or sand contents. Again, where drainage is poor and free water is available to the silty subgrade, frost heave should be considered as a significant factor affecting the desirability of this material.

Group A-5 soils are similar in grain-size distribution to A-4 soils, but have higher liquid limits, indicative of diatomaceous or micaceous soils. The elastic nature of these soils, especially in the absence of sand, causes group indices to be higher than the A-4 soils, perhaps as high as 12. Frost considerations are, again, a significant factor affecting usage of these soils as road subgrade.

Group A-6 soils are clays, usually plastic with 75 percent or more passing the 0.075mm (No. 200) sieve. With increasing sand content, up to 64 percent, the group index may be held low; but the group index can range up to 16 if the soil is devoid of sand. Usually, significant changes of volume will occur between dry and wet states. These materials may compact sufficiently at proper moisture content, but they will generally require a thicker pavement section to provide a non-yielding road surface. Frost considerations are usually outweighed by their affinity for water and the resulting volume changes and strength reductions that can result.

Group A-7 soils may be very elastic and plastic, subject to very high volume change with variations in moisture content. Strength can be low to high, but all A-7 soils are quite impermeable. A-7 soils are only utilized as road subgrade where nothing else is available.

Group A-8 soils are highly organic peats or mucks. These soils are highly undesirable for road subgrades and generally require removal.

3-2.02.03 UNIFIED

Another classification system used widely throughout the engineering community is the Unified Soil Classification System (USCS). The present system, modified by the U.S. Army Corps of Engineers and the Bureau of Reclamation, was introduced during World War II by Casagrande of Harvard University to assist engineers in the design and construction of airfields. As with the AASHTO system, the USCS utilizes grain-size distribution and plasticity characteristics to classify soils. The USCS, however, categorizes soils into one of 15 major soil groups that additionally account for the shape of the grain-size distribution curve.

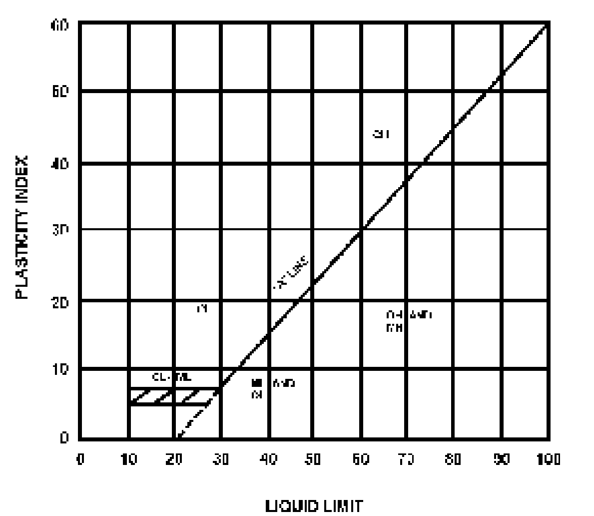
Table 3-2.4 shows the USCS classification system along with the criteria utilized for associating the group symbol, such as "CL," with the soil. In this chart, D_{60} refers to the diameter of the soil particles that 60 percent of the sample would pass on a sieve, as indicated on the gradation curve. Similarly, D_{10} relates to the maximum diameter of the smallest 10 percent, by weight.

Table 3-2.4. Unified Soil Classification System chart (after U.S. Army Corps of Engineers, Waterways Experiment Station, TM 3-357, 1953).

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES	LABORATORY CLASSIFICATION CRITERIA		
COARSE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE)	GRAVELS (MORE THAN HALF OF COARSE FRACTION IS GREATER THAN NO. 4 SIEVE SIZE)	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	DETERMINE PERCENTAGES OF SAND AND GRAVEL FROM GRAIN-SIZE CURVE. DEPENDING ON PERCENTAGE OF FINES (FRACTION SMALLER THAN NO. 200 SIEVE), COARSE-GRAINED SOILS ARE CLASSIFIED AS FOLLOWS: LESS THAN 5 PERCENT - GW, GP, SW, SP; MORE THAN 12 PERCENT - GM, GC, SM, SC; 5 TO 12 PERCENT - BORDERLINE CASES REQUIRING DUAL SYMBOLS (b)	$Cu = \frac{D60}{D10}$ GREATER THAN 4; $Cc = \frac{(D30)^2}{D10 * D60}$ BETWEEN 1 AND 3	
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES		NOT MEETING ALL GRADATION REQUIREMENTS FOR GW	
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM(a) d / u	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES		ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4	ABOVE "A" LINE WITH P.I. BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS BELOW "A" LINE WITH P.I. GREATER THAN 7	
	SANDS (MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE)	CLEAN SANDS (LITTLE OR NO FINES)	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		$Cu = \frac{D60}{D10}$ GREATER THAN 6; $Cc = \frac{(D30)^2}{D10 * D60}$ BETWEEN 1 AND 3	
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		NOT MEETING ALL GRADATION REQUIREMENTS FOR SW	
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM(a) d / u	SILTY SANDS, SAND-SILT MIXTURES		ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4	LIMITS PLOTTING IN HATCHED ZONE WITH P.I. BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS
			SC	CLAYEY SAND, SAND-CLAY MIXTURES		ATTERBERG LIMITS BELOW "A" LINE P.I. GREATER THAN 7	
FINE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE)	SILTS AND CLAYS (LIQUID LIMIT LESS THAN 50)		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY	PLASTICITY CHART		
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS			
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY			
	SILTS AND CLAYS (LIQUID LIMIT GREATER THAN 50)		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS			
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS			
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS			
	HIGHLY ORGANIC SOILS		Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS			

PLASTICITY CHART



a) Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u used when L.L. is greater than 28.
b) Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

The plasticity chart shown in the lower right-hand portion of Table 3-2.4 is a graphical representation of the USCS based solely on the plastic and liquid limits (Section 4-2.06.02) of the material passing the 0.425mm (No. 40) sieve. Clays will plot above the "A-line" and silts below. The chart further divides the clays and silts into low (less than 50) and high liquid limits.

Two examples of using Table 3-2.4 to obtain the soil's proper Unified Classification are:

Example 1. What is the classification of a soil sample with 88% passing the 4.76mm (No. 4) sieve, 38% passing the 0.075mm (No. 200) sieve, a liquid limit of 15, and a plastic limit of 4?

Initially, it is determined that the soil is coarse grained because more than half (62%) is retained on the 0.075mm (No. 200) sieve. It is then determined to be a sand because more than half of the 62% that is retained on the 0.075mm (No. 200) sieve passes the 4.76mm (No. 4) sieve. Since there is more than 12% passing the 0.075mm (No. 200) sieve, the soil is a sand with fines. The intersection of the liquid limit (15) and plasticity index (15 - 4 = 11) is above the "A line" on the plasticity chart. Therefore, the soil is an SC.

Example 2. What is the classification of a soil sample with 77% passing the 0.075mm (No. 200) sieve, a liquid limit of 44, and a plastic limit of 18?

Initially, it is determined that the soil is fine grained because more than half (77%) passes the 0.075mm (No. 200) sieve. The intersection of the liquid limit (44) and plasticity index (44 - 18 = 26) indicates a classification of CL.

3-2.02.04 CORRELATION OF CLASSIFICATION SYSTEMS

The triangular textural, AASHTO, and USCS classification systems are all attempts to associate pertinent engineering properties with identifiable soil groupings. However, each system defines soil groups in a slightly different manner. For example, the triangular textural and AASHTO classification systems distinguish gravel from sand at the 2.0 millimeters (No. 10) sieve, whereas the USCS uses a break at the 4.76 millimeters (No. 4) sieve. The same coarse-grained soil could, therefore, have different percentages of gravel and sand in the triangular textural and USCS classification systems.

Because of such differences, a direct correlation of these soil classifications cannot be made. However, it is possible to make a general comparison as shown in Table 3-2.5.

3-2.03 ENGINEERING PROPERTIES

This section discusses the soil properties of principal interest for analysis and design of highway subgrades/embankments and pavement structures. Reference is made to Section 4-2.06 for more detailed test information.

3-2.03.01 ATTERBURG LIMITS

The engineering properties of fine-grained soils vary with the amount of water present. In 1911, A. Atterberg established limiting water contents for key physical states of interest to engineers. These limits are known as the Atterberg limits and consist of the liquid limit, the plastic limit and the shrinkage limit. These values are water contents (expressed as percentages) where the soil behavior changes. A description of these states, limits, and indices is shown in Figure 3-2.3.

Table 3-2.5. Approximate equivalent classifications.

Mn/DOT Triangular Textural	AASHTO (Group Index)	Unified (USCS)
Gravel	A-1-a(0)	GW, GP
Sand	A-1-b(0)	SW, SP
Coarse	A-1-a, A-1-b(0)	SW, SP
Fine	A-1-b, A-3(0)	SW, SP
Loamy Sand	A-2-4, A-2-5(0)	SM, SC
Sandy Loam		
a) Slightly plastic	A-2-4, A-2-6, A-2-7(0)	SM, SC
b) Plastic	A-4(0-4)	SM, SC
Loam	A-4(0-4)	ML, OL, MH, OH
Silt Loam	A-4(0-4)	ML, OL, MH, OH
Silt	A-4	ML, OL, MH, OH
Sandy Clay Loam	A-6, A-5(0-16)	SC, SM
Clay Loam	A-6(0-16)	ML, OL, CL, MH, OH, CH
Silty Clay Loam	A-6, A-5(0-16)	ML, OL, CL, MH, OH, CH
Sandy Clay	A-7, A-7-6(0-20$^+$)	SC, SM
Silty Clay	A-7, A-7-5(0-20$^+$)	OL, CL, OH, CH
Clay	A-7(0-20$^+$)	CL, CH, OH, OL

Above the liquid limit, LL, the soil-water system is a suspension. Below the liquid-limit and above the plastic limit, PL, the soil-water system is said to be in a plastic state. In this state the soil maybe deformed or remolded without the formation of cracks and without change in volume. The range of water content over which the soil-water system acts as a plastic material is frequently referred to as the plastic range. The extent of this range is represented by the plasticity index, which is the liquid limit minus the plastic limit:

Plasticity Index PI = LL – PL

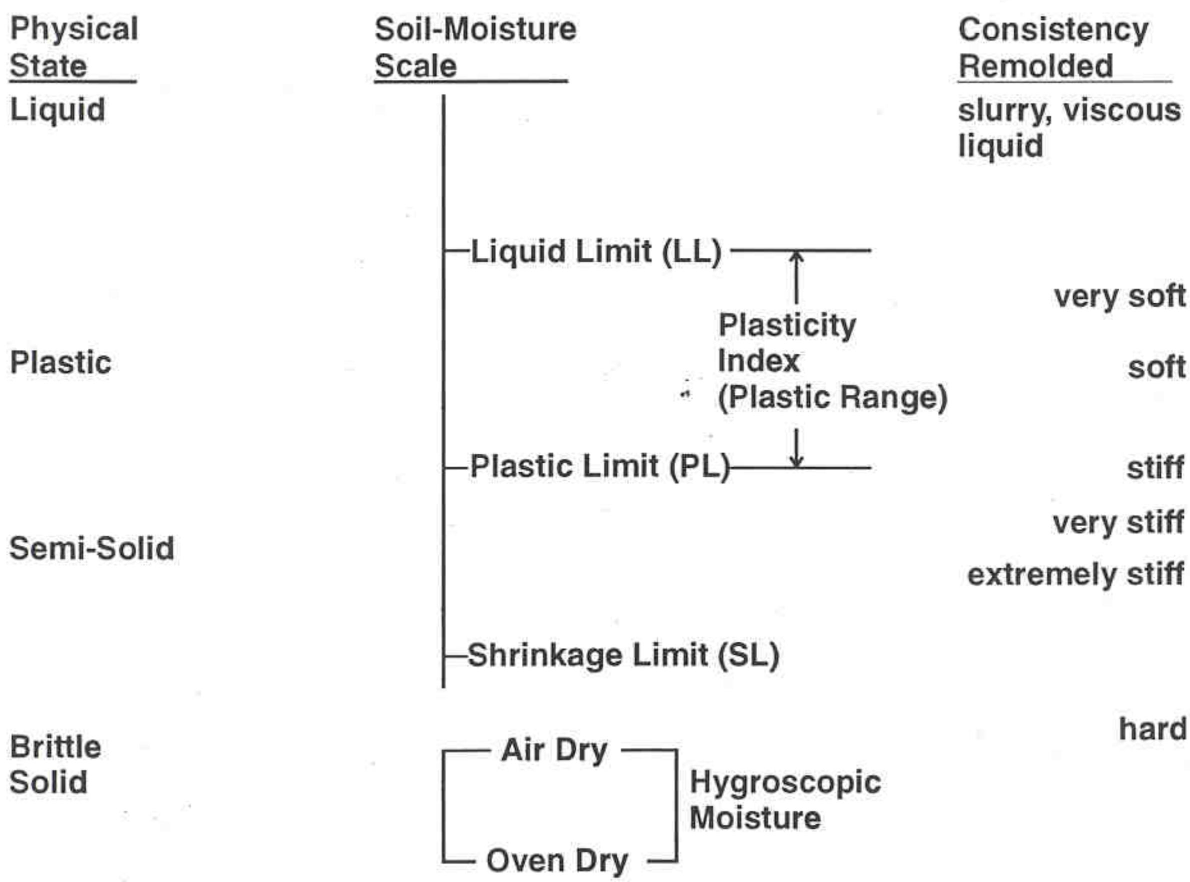
- A high PI indicates a compressible material with a high degree of cohesion
- A low PI indicates a cohesionless or non-plastic material
- The higher the PI, the lower the permeability
- A low PI indicates sensitivity to change in moisture content

Somewhat below the plastic limit, the soil-water system reaches the shrinkage limit, SL. At this point, all soil particles are in contact and the material can shrink no further. Therefore, the material's volume will not be reduced if the moisture content falls below the shrinkage limit. Instead, air will enter the voids formerly occupied by the water.

The most common use of these Atterberg limits and indices is soil classification. Soils with comparable limits and indices are classed together. The number is used to classify fine-grained soils and the indices to characterize soil behavior. Generally, soils with high liquid limits are clays with poor engineering properties. Both the liquid limit and plasticity index are used to some degree as a quality-measuring device for subgrade and aggregate base materials.

Test methods for establishing the limits and the associated indices are described in Section 4-2.06.

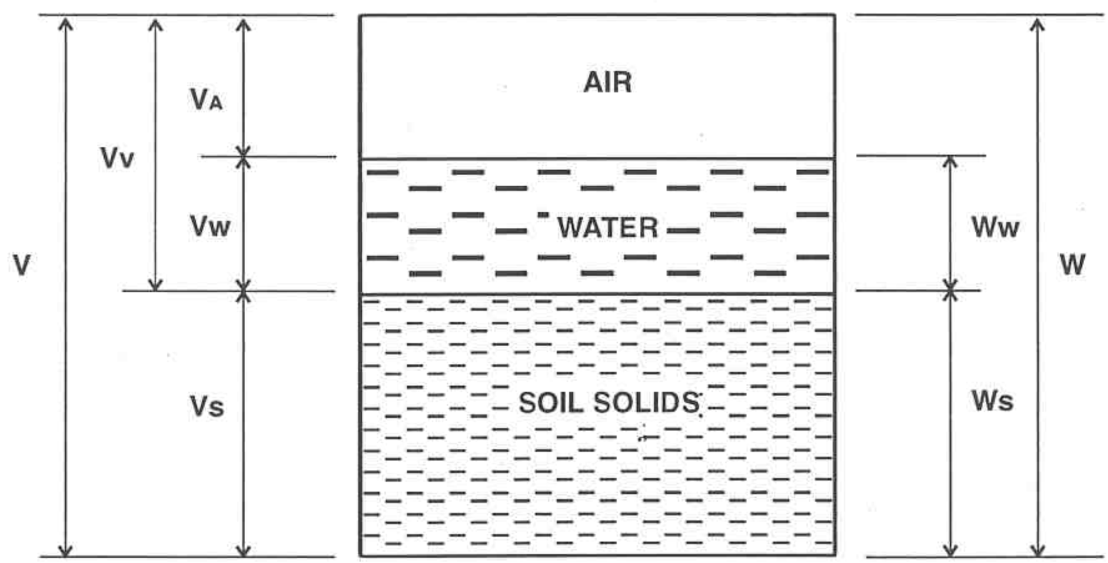
Figure 3-2.3 Atterberg Limits Relationships



3-2.03.02 VOLUME AND WEIGHT RELATIONSHIPS

Soil is comprised of a mixture of soil solids, water, and air. The relative proportion of each of these constituents determines many of the properties of the soil. A soil block diagram, with symbols for each of its volume and mass components, is shown in Figure 3-2.4.

Figure 3-2.4. Volume and weight relationships for a soil.



The moisture content is the ratio of the weight of water to that of the dry soil solids, expressed as a percent. It is determined as follows:

$$w = \frac{W_W}{W_S} * 100 \qquad \text{Eq. 3.2.1}$$

where:

w = moisture content (%)
W_s = dry weight of solids (gm)
W_w = weight of water (gm)

Table 3-2.6 Typical Moisture Contents[a]

Material	Moisture Content, w (%)
Gravel	2-10
Sand	5-15
Silts	5-40
Clays	10-50 (or more)
Organic (Peat)	> 50

a) Terzaghi, K. and Peck, R. B., “Soil Mechanics in Engineering Practice”

The porosity is the ratio of the volume of voids to the total volume and may be expressed as either a percent or decimal. It is determined as follows:

$$n = \frac{V_v}{V} \qquad \text{Eq. 3.2.2}$$

where:

n = porosity

V_v = volume of voids (cm^3)
V = total volume, (cm^3)

The degree of saturation is the ratio of the volume of water to the total volume of voids, expressed as a percent. It is determined as follows:

$$S = \frac{V_w}{V} \text{ x } 100 \qquad \text{Eq. 3-2.4}$$

where:

S = saturation,(%)
V_w = volume of water (cm^3)
V_v = volume of voids (cm^3)

The void ratio is the ratio of volume of voids to volume of solids and may be expressed as a percent or decimal. It is determined as follows:

$$e = \frac{V_v}{V_s} \text{ x } 100 \qquad \text{Eq. 3-2.5}$$

where:

e = void ratio
V_v = volume of voids (cm^3)
V_s = volume of solids (cm^3)

The density, or unit weight, of the soil mass is further divided into moist density and dry density. Moist density is the weight of water and soil solids divided by the volume of the soil mass. Dry density is the weight of only the soil solids divided by the volume of the soil mass. These values are determined using the following formulas:

$$V_m = \frac{W_w + W_s}{V} \qquad \text{Eq. 3-2.6}$$

where:

Y_m = moist density (kg/m^3(pcf))
W_s = weight of solids (kg (lb))
W_w = weight of water (kg (lb.))
V = total volume (m^3 (ft^3))

$$Y_d = \frac{Y_m}{1 + \frac{w}{100}} \qquad \text{Eq. 3-2.8}$$

or

$$Y_d = \frac{W_s}{V} \qquad \text{Eq. 3-2.7}$$

where:

Y_d = dry density (kg/m^3 (pcf))
Y_m = moist density (kg/m^3 (pcf))
W_s = weight of solids (kg (lb.))
w = moisture content(%)
V = total volume (m^3 (ft^3)).

Table 3-2.7 Typical Dry Densities[a]

Soil	γ_d kg/m^3 (lb/ft^3)
Gravel and Sand	1,900 – 2,250 (120 – 140)
Silts and Clay	1,450 – 1,750 (90 – 110)
Peat	~ 300 (20)

a) Terzaghi, K. and Peck, R. B., "Soil Mechanics in Engineering Practice"

The density of the soil mass affects the strength of the soil. Generally, the strength of a soil increases as its dry density increases. Also the potential for the soil to take on water at later times is decreased by higher densities. This is due to the decreased presence of air space in the soil mass.

The in-place moisture content of a soil is often used, along with the soil classification, to determine the suitability of the material as a subgrade. Generally, as the moisture content of a soil increases its strength decreases and the potential for deformation and instability increases. For example, if the natural moisture content is near the liquid limit then the soil will quickly be disturbed by earth moving equipment and is unlikely to be suitable subgrade material. On the other hand, a natural moisture content below the plastic limit indicates a relatively firm material that could provide a suitable subgrade, provided that additional moisture is not added. The moisture content of a soil should be expected to vary seasonally.

The standard Proctor Test is used by Mn/DOT to establish a relationship between moisture and density. The test provides the maximum dry density that can be achieved at a variety of moisture contents under a given compactive effort. The moisture content corresponding to the maximum density is referred to the optimum moisture content.

The test density and moisture content can be compared to the in-situ density and moisture properties. These properties can be determined using thin-wall tubes or sand cone procedures. A relationship between the two densities and moisture contents will provide an indication how a given soil / material will perform in a structure or what modifications may be necessary to improve its engineering properties so as to increase suitability.

3-2.03.03 SWELL/SHRINKAGE

Soils may undergo volume changes as their moisture content varies. This phenomenon is known as shrinkage or swell. Volume changes occur for moisture contents varying between the shrinkage limit and saturation.

Shrinkage is primarily related to the particle size and structure of the soil and is caused by the capillary action of the water in the soil mass. As water evaporates, tension is exerted on the soil solids causing them to move closer together. The amount of shrinkage is dependent on many factors, including the clay minerals present, the soil structure, and chemical aspects of the soil. Shrinkage pressures have been recorded in excess of 20 ksf, which is equivalent to the weight of an embankment over 45 m (150 feet) in height. Significant volume changes can result from these forces.

Shrinkage and swelling of the soil subgrade can be detrimental to the pavement structure. Table 3-2.8 can be used as a guide to determine which soils have swell potential.

3-2.03.04 FROST SUSCEPTIBILITY

Frozen soils can exhibit frost heave and, sometimes, detrimental strength loss during the subsequent thawing process. When the ground water is shallow, or when water is drawn above the ground water table through connected voids (soil tubes) by capillary action, it can cause up to a nine percent expansion of the soil when frozen. The depth to which freezing will occur is dependent upon the available moisture in the subgrade, the subgrade soil type, and the freezing index.

Table 3-2.8. Probable Expansion as Estimated from Classification Test Data* (Swell potential of soils.)

Degree of Expansion	Volume Change**	Plasticity Index (PI)	Shrinkage Limit (SL)
Very High	> 30	> 35	< 11
High	20 - 30	25 - 41	7 - 12
Medium	10 - 20	15 - 28	10 - 16
Low	< 10	< 18	> 15

* After Holtz (1959) and U.S.B.R. (1974)

** Volume change as a percent of total change going from a dry to saturated condition (under a surcharge of 6.9 kPa).

Frost heave can be reduced by using drainage to keep the ground water below the pavement structure, by removing the frost-susceptible materials and replacing them with non-frost-susceptible materials, or by mixing low-susceptible materials with existing highly susceptible materials.

Inorganic mineral soils that contain particles finer than 0.02 millimeters are susceptible to frost action. Generally, the greater the percent finer than 0.02 millimeters, the greater the susceptibility to frost action. Figure 3-2.5 shows the relative degree of frost susceptibility of various soil types. As can be seen, silts exhibit the highest rate of heave while clean sands and gravels exhibit the lowest rate of heave.

The frost depth for a pavement section can be calculated using the following procedure. First, the number of degree-days required to freeze the pavement must be determined using Equation 3-2.10. Next, the number of degree-days required to freeze the base course is similarly determined using Equation 3-2.10. The freezing index (as taken from Figure 3-2.6) is then reduced by the number of degree days required to freeze the pavement and the base course. Lastly, the depth of frost penetration into the subgrade is determined using the reduced freezing index and Equation 3-2.11. If additional layers of base course or different soil layers are present, then the first two steps are repeated until the last layer is reached or until the reduced freezing index is equal to or lower than zero, which indicates that the maximum depth of frost penetration has been reached.

$$L = 1.43\, wY_d \qquad \text{Eq. 3-2.9}$$

where:

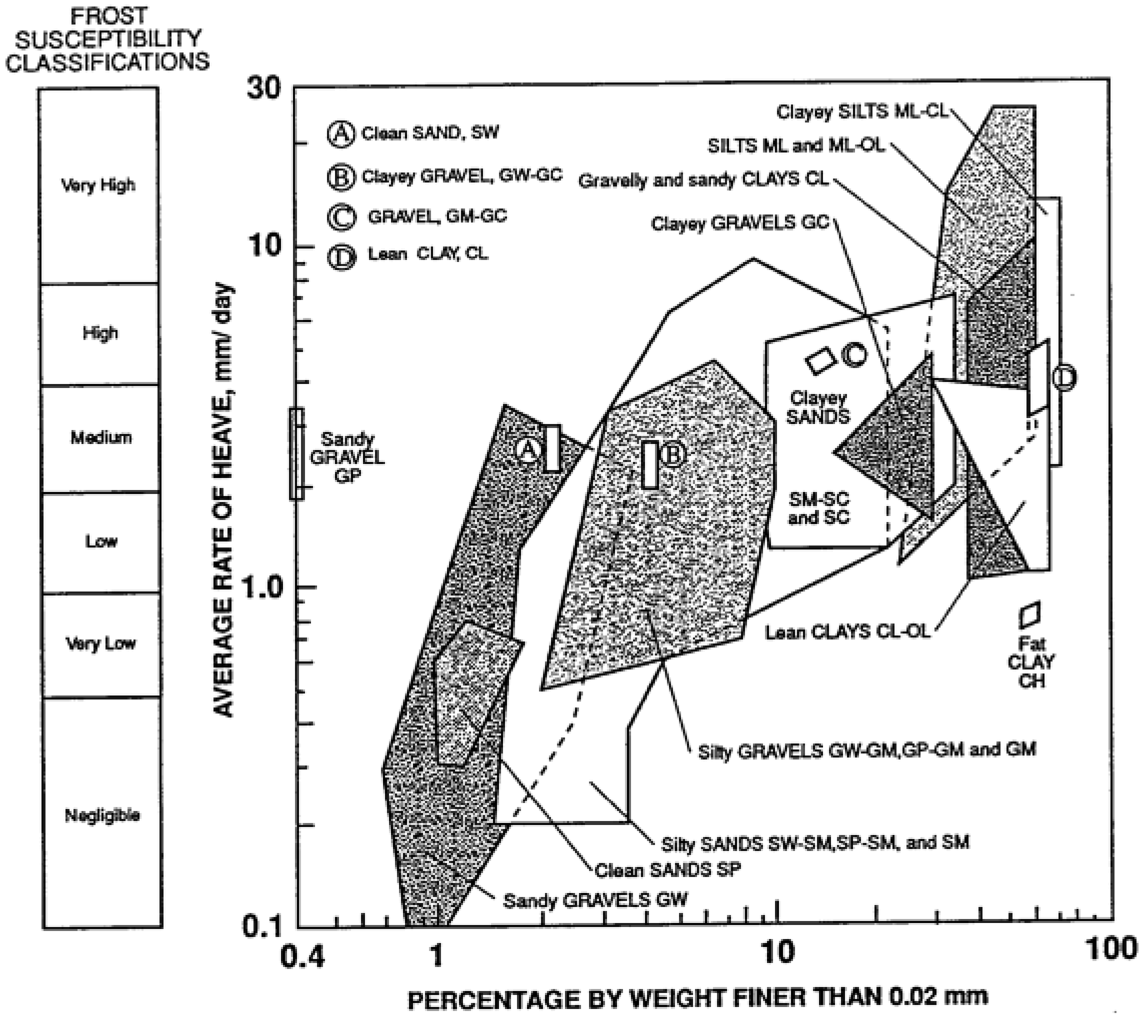
L = volumetric heat of latent fusion (BTU/ft^3)
Y_d = dry density (pcf)
w = moisture content (%)

$$F = \frac{h^2 L}{48k} \qquad \text{Eq. 3-2.10}$$

where:

F = freezing index (degree days)
h = layer thickness (in.)
L = volumetric heat of latent fusion (BTU/ft^3)
k = thermal conductivity ($BTU/ft^2/hr./{}^\circ F/in.$) (Values of 6.5 and 10.0 are generally used as constants for Portland cement concrete and bituminous pavements, respectively.)

Figure 3-2.5. Frost susceptibility of various soil types tested by the Corps of Engineers, 1950-1970 (from FHWA-TS-80-224, "Highway Subdrainage Design," August 1980).



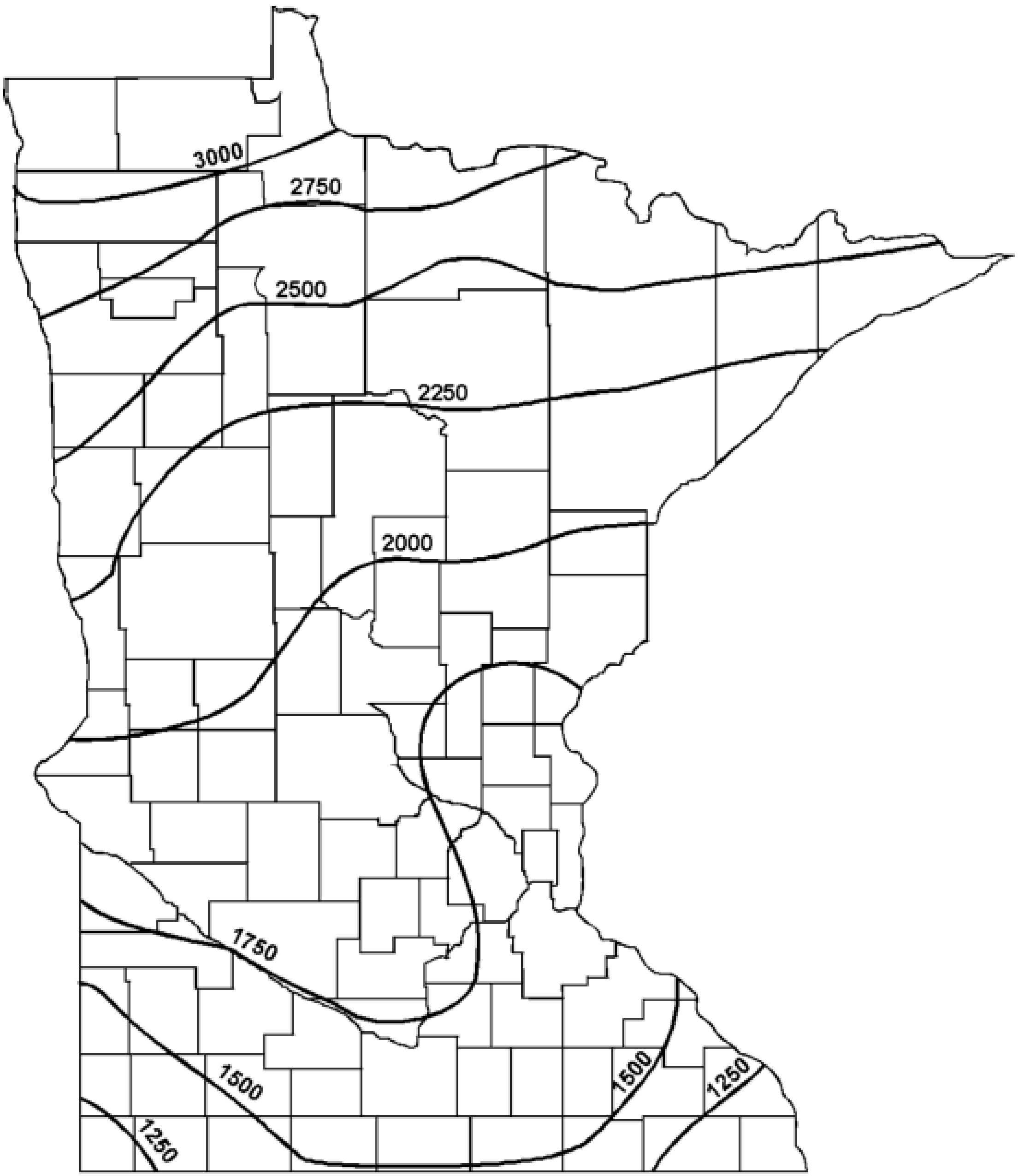
$$z = \sqrt{\frac{48kF}{L}} \qquad \text{Eq. 3-2.11}$$

where:

z	=	frost depth (in.)
F	=	freezing index (degree days)
L	=	volumetric heat of latent fusion (BTU/ft^3)
k	=	thermal conductivity (BTU/ft^2/hr./ ° F/in.)

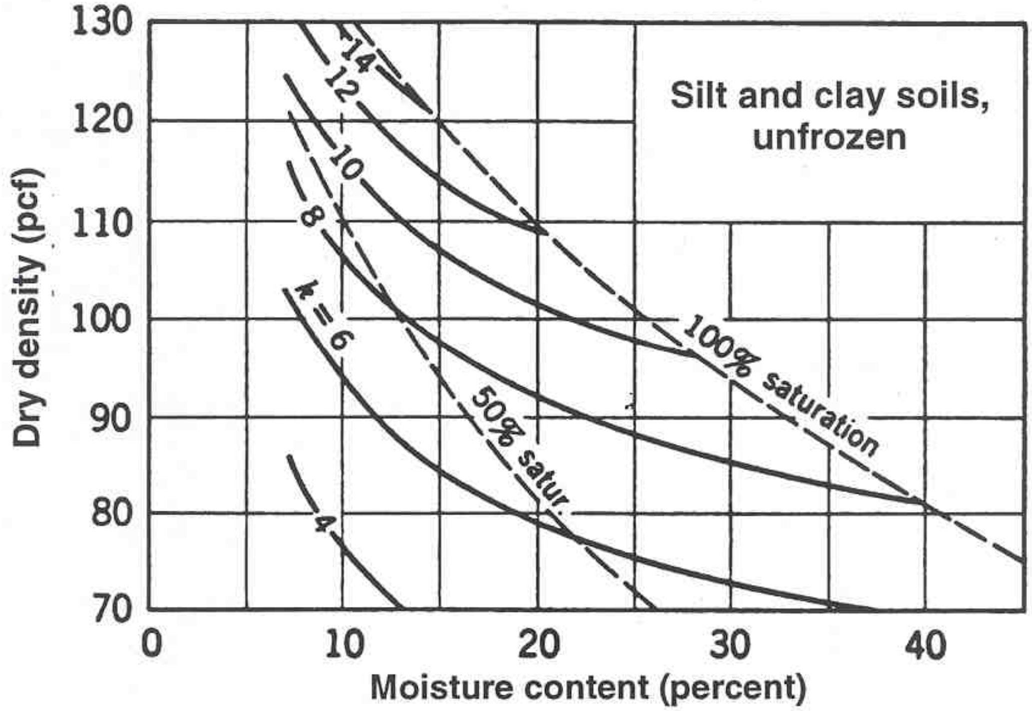
Values for w and $_d$ are determined through field investigations. Values for freezing index and k are determined from Figures 3-2.6 and 3-2.7, respectively. The freezing index is determined from a cumulative plot of degree-days versus time. A degree-day represents one day with a mean air temperature one degree below freezing. Thus, 10 degree-days would result when the air temperature is either 31°F for 10 days or 22°F for one day. Figure 3-2.6 shows a map of the average annual freezing index for the state of Minnesota.

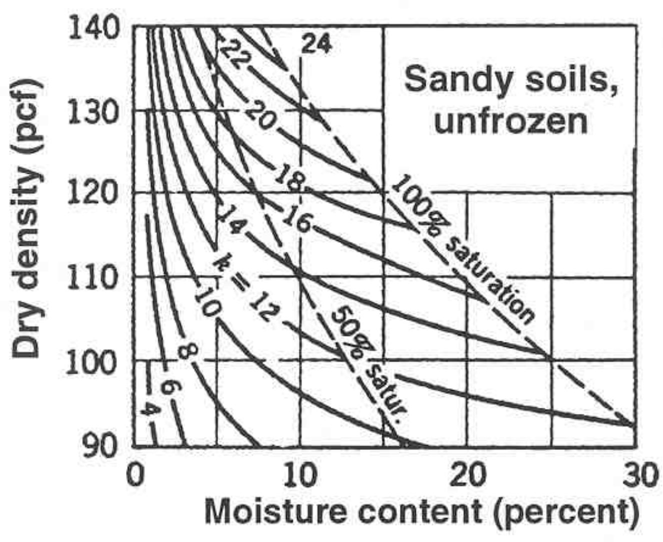
Figure 3-2.6. Freezing Index (from Corps of Engineers, EM 1110-345-306).



NOTE: MEAN FREEZING INDEX VALUE IN DEGREE DAYS BELOW 32° F

Figure 3-2.7. Thermal conductivity (from Kersten, 1952).





The following is an example frost depth calculation:

<u>Example</u>. Assume a 100cm (4-in.) thick PCC pavement overlies 250cm (10 in.) of base rock above a silty clay subgrade. The PCC has a dry density of 140 pcf and a moisture content of 2%. The base rock has a dry density of 120 pcf and a moisture content of 10%. The silty clay has a dry density of 110 pcf and a moisture content of 18%. The project location has a freezing index of 2,000 degree-days (Figure 3-2.6).

Using Equation 3-2.9:

$L_{\text{PCC}} = (1.43)\ (2)\ (140) = 400,$

$L_{\text{base rock}} = (1.43)\ (10)\ (120) = 1{,}716,$ and

$L_{\text{silty clay}} = (1.43)\ (18)\ (110) = 2{,}831.$

For PCC, k = 6.5 (constant). For base rock, k = 18 (Figure 3-2.7). For silty clay, k = 12 (Figure 3-2.7). Then, using Equation 3-2.10:

$F_{\text{PCC}} = (4)^2\ (400) \div (48)\ (6.5) = 21,$ and

$F_{\text{base rock}} = (10)^2\ (1{,}716) \div (48)\ (18) = 199.$

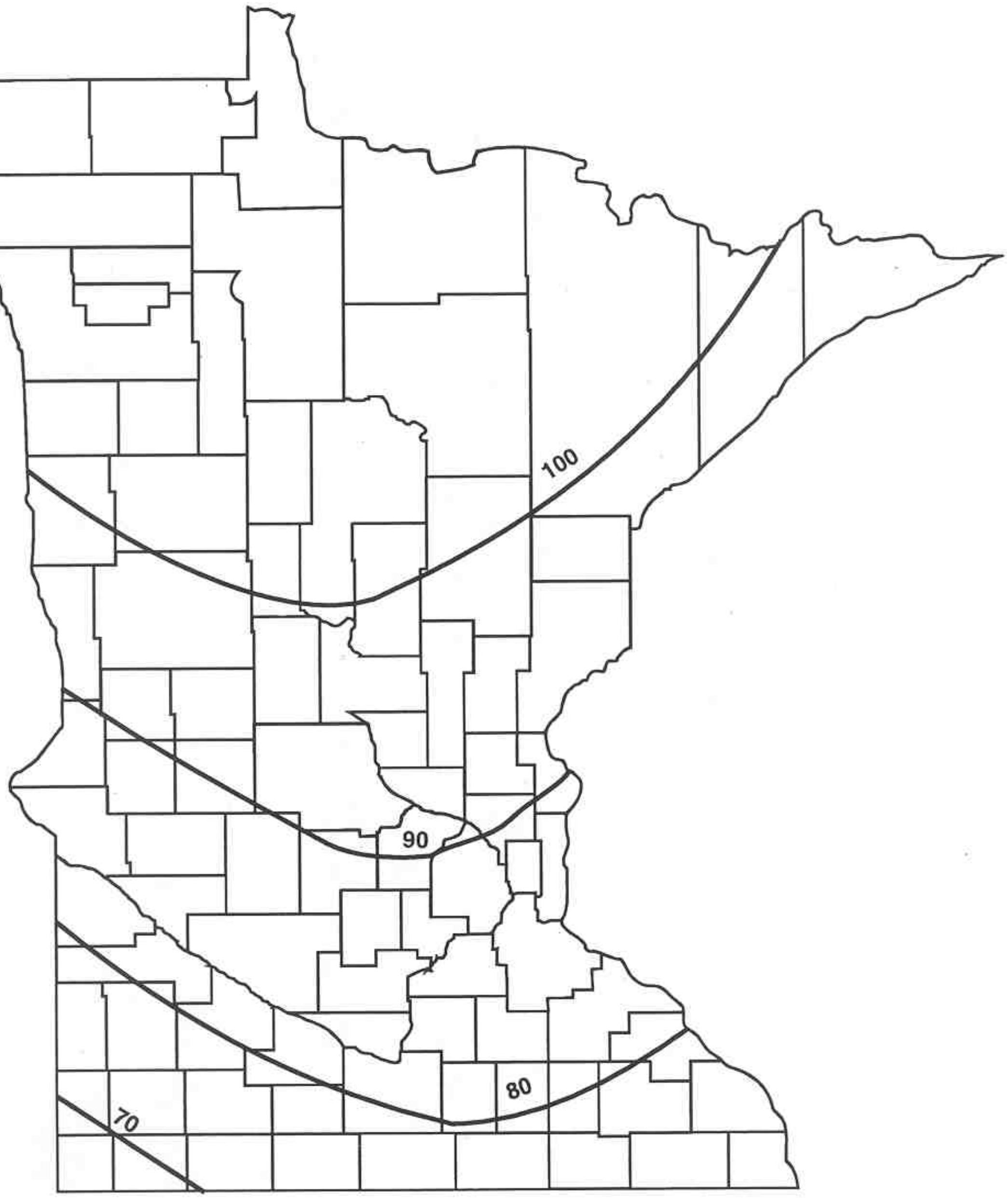
Therefore, $F_{\text{silty clay}} = \underline{2{,}000 - (21 + 199) = 1{,}780}$, and using Equation 3-2.11:

$z_{\text{silty clay}} = \sqrt{(48)\ (12)\ (1{,}780) \div (2{,}831)} = 19.0$ in.

And, the frost depth = 4 + 10 + 19.0 = <u>33.0 in.</u>

Figure 3-2.8 can be used to obtain an estimate of the maximum depth of frost penetration in Minnesota.

Figure 3-2.8. Estimated maximum depth of frost penetration (from FHWA-TS-80-224, "Highway Subdrainage Design," August 1980).



NOTE: DEPTH IN INCHES

3-2.03.05 STRENGTH

The strength (resistance to deformation) properties of soils for use in empirical and mechanistic-empirical pavement design models are usually represented by one of the following values.

1. R-Value. The R-Value test method was developed by Francis Hveem and R. M. Carmary of the California Division of Highways in the late 1940's to evaluate materials to be used for bases, subbases, and subgrade soils. The test was developed at time when rutting (or shoving) in the wheel tracks was the primary design concern. The surface courses were not as thick as they are today (hence fatigue has become a more serious design criterion).

   The test is conducted using a device called a stabilometer, where the material's resistance to deformation is expressed as a function of the ratio of the transmitted lateral pressure to that of the applied vertical pressure (which is 1,103 kPa (160 psi)).

   The results of the test are normalized to a zero to 100 scale, where water would have an R-Value of 0 and steel would have an R-Value of 100. Subgrade soils and pavement materials fall within this range: heavy clays may have R-Values as low as 4 and granular soils often have R-Values of 70 – 75.

   Assumed values, based on the Department's laboratory testing and past experience, are presented in Table 5-3.2(a) and (b) in Section 5-3.05.

2. California Bearing Ratio (CBR).

   This CBR test was developed by the California Division of Highways using survey results from 1928 and 1929. The test procedures were standardized in 1929 and revised and adopted by the U. S. Army Corps of Engineers in the 1940s.

   This test is a comparative measure of the shearing resistance of unbonded materials (i.e. base, subbase and subgrade) and is used with empirically derived curves to design flexible pavement structures. Physically, it consists of measuring the load required to cause a plunger of a standard size to penetrate a specimen at a specified rate. The CBR is the percentage of the load in kPa (psi) required to force a piston a particular depth into a standard sample of well-graded crushed stone needed to force the piston the same depth into a specimen. (Additional information on the CBR is provided in Section 4-2.06.05). Therefore, a high quality crushed stone base material should have a CBR near 100%.

   The strength of a given soil is affected by the moisture content and density of the tested samples. Therefore, CBR values will vary in the field depending on the moisture and density of the placed materials. Typical CBR values are shown in Table 3-2.9.

Table 3-2.9. Typical CBR values.

Mn/DOT Triangular Textural Classification	CBR Value
Gravel	20 - 80
Sand	10 - 40
Loamy Sand	5 - 40
Sandy Loam	5 - 40
Loam	< 15
Silt Loam	< 15
Sandy Clay Loam	5 - 40
Clay Loam	< 15
Silty Clay Loam	< 15
Sandy Clay	5 - 40
Silty Clay	< 15
Clay	< 15

The Department does not presently use CBR values for strength determination.

3. Triaxial Compression and Direct Shear Tests . The shear strength of a soil is its resistance to interparticle movement. This resistance is derived from two sources: cohesion and friction. Cohesion is the attraction of one particle to another. Friction results when one particle attempts to move past another. The shear strength is determined from the following formula.

$$s = c + (\sigma - u)\tan(\Phi) \qquad \text{Eq. 3-2.12}$$

where:

c = cohesion (kPa (psf))
s = shear strength (kPa (psf))
σ = normal stress (kPa (psf))
u = pore pressure (kPa (psf))
Φ = angle of internal friction (degrees)

The values of cohesion (c) and angle of internal friction (Φ) are dependent upon the stress history of the soil, the current stress state, and the type of test used to determine the values.

There are two forms of shear strength: drained and undrained. The drained shear strength represents the long-term condition where there is no increased water pressure due to the applied load. The undrained shear strength represents the short-term, or construction, condition where the water pressure does not have time to dissipate.

Unconfined compression tests can be used to determine the undrained strength of cohesive samples. Triaxial compression tests can be used to determine the drained and/or undrained strength of any soil sample. Direct shear tests are used primarily on rock samples but can also be used to determine the drained strength of soil samples. Undisturbed samples, obtained by the Foundations Unit, are required for all of the above strength tests.

Typical values of c and Φ are shown in Table 3-2.10.

Table 3-2.10. Typical cohesion and angle of internal friction values.

Mn/DOT Triangular Textural Classification	c Cohesion, kPa (psf) Compacted	Saturated	Φ Angle of Internal Friction (degrees)
Gravel	0	0	> 37
Sand	0	0	37 – 38
Loamy Sand	50 – 75 (1,000 - 1,500)	10- 20 (200 – 400)	31 – 34
Sandy Loam	50 – 75 (1,000 - 1,500)	10 - 20 (200 – 400)	31 – 34
Loam	60 – 90 (1,300 - 1,800)	10 - 20 (200 – 400)	28 – 32
Silt Loam	60 – 90 (1,300 - 1,800)	10 - 20 (200 – 400)	25 – 32
Sandy Clay Loam	50 – 75 (1,000 - 1,500)	10 – 20 (200 – 400)	31 – 34
Clay Loam	60 – 105 (1,300 - 2,200)	10 -20 (200 – 400)	18 – 32
Silty Clay Loam	60 – 105 (1,300 - 2,200)	10 - 20 (200 – 400)	18 – 32
Sandy Clay	50 – 75 (1,000 - 1,500)	10 - 20 (200 – 400)	31 – 34
Silty Clay	90 – 105 (1,800 - 2,200)	10 -20 (200 – 400)	18 – 32
Clay	90 – 105 (1,800 - 2,200)	10 - 20 (200 – 400)	18 - 28

While these values can be used in pavement design, they are generally more applicable to slope stability and bearing capacity calculations. Therefore, these are most often used by the Foundations Unit, rather than at the District level.

The method of determining each of the above strength values is presented in Section 4-2.06.05.

4. Dynamic Cone Penetrometer (DCP). This test is a field procedure that can be used to determine the in-situ strength properties of base and subgrade soils. The DCP is currently used for quality assurance testing by Mn/DOT.

5. Elastic Modulus of Subgrade Reaction (k-value). The k-value is used as a primary input in concrete pavement design thickness models, and can only be measured using a field test on top of the subgrade. There is no direct laboratory procedure for determining the k-value.

The k-value can be estimated using any of the following methods:

1) Plate bearing tests on subgrade. A 762mm (30 in.) diameter plate is loaded to a given pressure at a specified rate and the resulting deflection is measured.

$$k = p/\Delta \qquad \text{Eq. 3-2.13}$$

where:

p = unit pressure on the plate, typically 69kPa (10 psi)
Δ = vertical deflection of the plate (mm (in.))

This test requires expensive equipment and is costly to perform, therefore Mn/DOT rarely performs it.

2) Deflection testing (Falling Weight Deflectometer – FWD) and back calculation of subgrade k-value. The FWD can be used to estimate the k-value by conducting the test on the top of subgrade or on the in-situ pavement structure.

3) Correlation with soil type and other soil properties or tests. Subgrade k-values can be estimated using soil classification, moisture level, dry density, CBR, R-Value, DCP or Mr.

The Department uses a correlation developed from the results of plate load tests that conducted for Investigation No. 183. (Flexible Aggregate Base Design). The correlation is as follows:

$$\text{k-value} = -1.17 + 63\sqrt{\text{R-Value}} \qquad \text{Eq. 3-2.14}$$

where:

k-value is in psi/in or kPa/mm = psi/in * 0.271

3-2.03.06 ELASTIC MODULUS and POISSON'S RATIO

New pavement design models (such as Mn/DOT's MNPAVE) require that the subgrade soils be characterized mechanistically. Mechanistic characterization permits the application of the principles of engineering mechanics (namely, elastic theory) to pavement analysis problems. This involves predicting the states of stress, strain, and displacement within the pavement structure when subjected to a wheel load based on the pavement response properties. These properties are the modulus of elasticity and Poisson's ratio.

1. Resilient Modulus (Mr). Resilient Modulus is the material property used to characterize subgrade soil for mechanistic pavement design. This modulus is roughly the equivalent of the elastic modulus, however, it is calculated using the recoverable strain observed during cyclic loading. The modulus is highly influenced by the state of stress and in-situ moisture content. (It should be noted that Mr is not a measure of strength. Strength is the stress needed to break or rupture a material, where as elasticity means that the material returns to its original shape and size.)

   The Mr value used for design should represent the conditions of the finished compacted embankment upon which the pavement structure will be constructed, both in terms of expected field density and in-situ moisture content.

   Different methods can be used to calculate modulus values for use in the design models:

   - Correlation with other laboratory test values.
     NCHRP Project 1-37A, "Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structure" provides the following:
     Correlation from CBR tests –
     $\text{Mr (psi)} = 2{,}550\ (\text{CBR})^{0.64}$ Eq 3.2.03.1
     $\text{Mr (kPa)} = 6.89 * \text{Mr (psi)}$

     Correlation from R-Value tests –

Mr (psi) = 155 + 555 (R-Value) Eq. 3.2.03.2
Mr (kPa) = 6.89 * Mr (psi)

Correlation from DCP tests –
Determine penetration resistance (PR) from DCP test.
$CBR = 292/PR^{1.12}$ Eq. 3.2.03.3
$Mr\ (psi) = 2{,}550\ (CBR)^{0.64}$ Eq. 3.2.03.1
Mr (kPa) = 6.89 * Mr (psi)

- Deflection Testing and Backcalculation of Moduli. Deflection testing is used to evaluate structural capacity and to estimate the subgrade soil support moduli for in-situ conditions. Backcalculated moduli values may need to be adjusted to values that are consistent with laboratory determined moduli for use in various pavement design models.

- Laboratory Resilient Modulus Test. The resilient modulus of subgrade soils can be determined directly from repeated load triaxial compression tests.

  These tests are preformed over a range of vertical stresses and confining pressures to evaluate the non-linear elastic behavior of soil. The Mr test does not result in a single modulus value, but it does define the modulus at different stress states. The modulus is dependent on the stress state.

- Soil Classification Correlations
  For subgrade soils, typical Mr values vary between 20 MPa (3,000 psi) and 100 MPa (15,000 psi) depending on the soil type, moisture content, deviator stress, and confining pressure.

  The method of predicting the Mr value should be commensurate with the priority and availability of resources of the pavement design under consideration. However, it should be kept in mind that the reliability of the pavement design is directly related to the level of effort in determining the Mr values.

  Additional information and equations are provided in section 4-2.06.05.

2. Poisson Ratio. This parameter is defined as the ratio of diametrical strain to longitudinal strain of a loaded specimen. It is usually determined during a resilient modulus test.

- $\mu = \varepsilon_D/\varepsilon_L$ Eq. 3-2.03-4

  where:
  μ = Poisson Ratio
  $\varepsilon_D = \Delta D/D$ = strain along the diametrical (horizontal) axis.
  $\varepsilon_L = \Delta L/L$ = strain along the longitudinal (vertical) axis.

However, because the Poisson Ratio has a relatively small effect on pavement responses, a reasonable value is assumed rather than measured. Typical values for subgrade soils are as shown in Table 3.2.11

Table 3.2.11 Typical Poisson Ratio Values

Soil	Range	Typical Value
Loose sand or silty sand	0.20 – 0.40	0.30
Dense sand	0.30 – 0.45	0.35
Fine-grained soils	0.30 – 0.50	0.40
Saturated soft clays	0.40 – 0.50	0.45

Generally, stiffer materials have lower ratios than softer materials.

3-2.03.06 PERMEABILITY / DRAINABILITY

Permeability is the rate of flow of a fluid through a porous medium. When that fluid is ground water, the terms **hydraulic conductivity** (K) and **coefficient of permeability** (k) are essentially equivalent. However, for fluids other than water the permeability coefficient includes additional factors relating to the viscosity of the fluid.

The terms hydraulic conductivity and coefficient of permeability are used interchangeably in much of the present literature. Although hydraulic conductivity is the preferred term in technical documents, the term **permeability** is often more easily understood. Both terms are expressed in units of length per time, such as feet per day (fpd) or centimeters per second (cps).

If water is present in the soil mass at or near an excavation elevation, then the water that will flow into the excavation must be accounted for. The greater the coefficient of permeability or hydraulic conductivity, the greater the volume of water that must be controlled. Therefore the value of k (or K) will impact both design and construction.

Table 3-2.12 from Terzaghi and Peck provides relative permeabilities that are helpful in interpreting the ability of water to flow through various soil types.

Table 3-2.12 Degree of Permeability

Degree of Permeability	**Coefficient of Permeability (k)**	
	cps	**fpd**
High	1×10^{-1}	10^{2}
Medium	1×10^{-1} to 1×10^{-3}	10^{2} to 10^{0}
Low	1×10^{-3} to 1×10^{-5}	10^{0} to 10^{-2}
Very Low	1×10^{-5} to 1×10^{-7}	10^{-2} to 10^{-4}
Practically Impermeable	$< 1 \times 10^{-7}$	$< 10^{-4}$

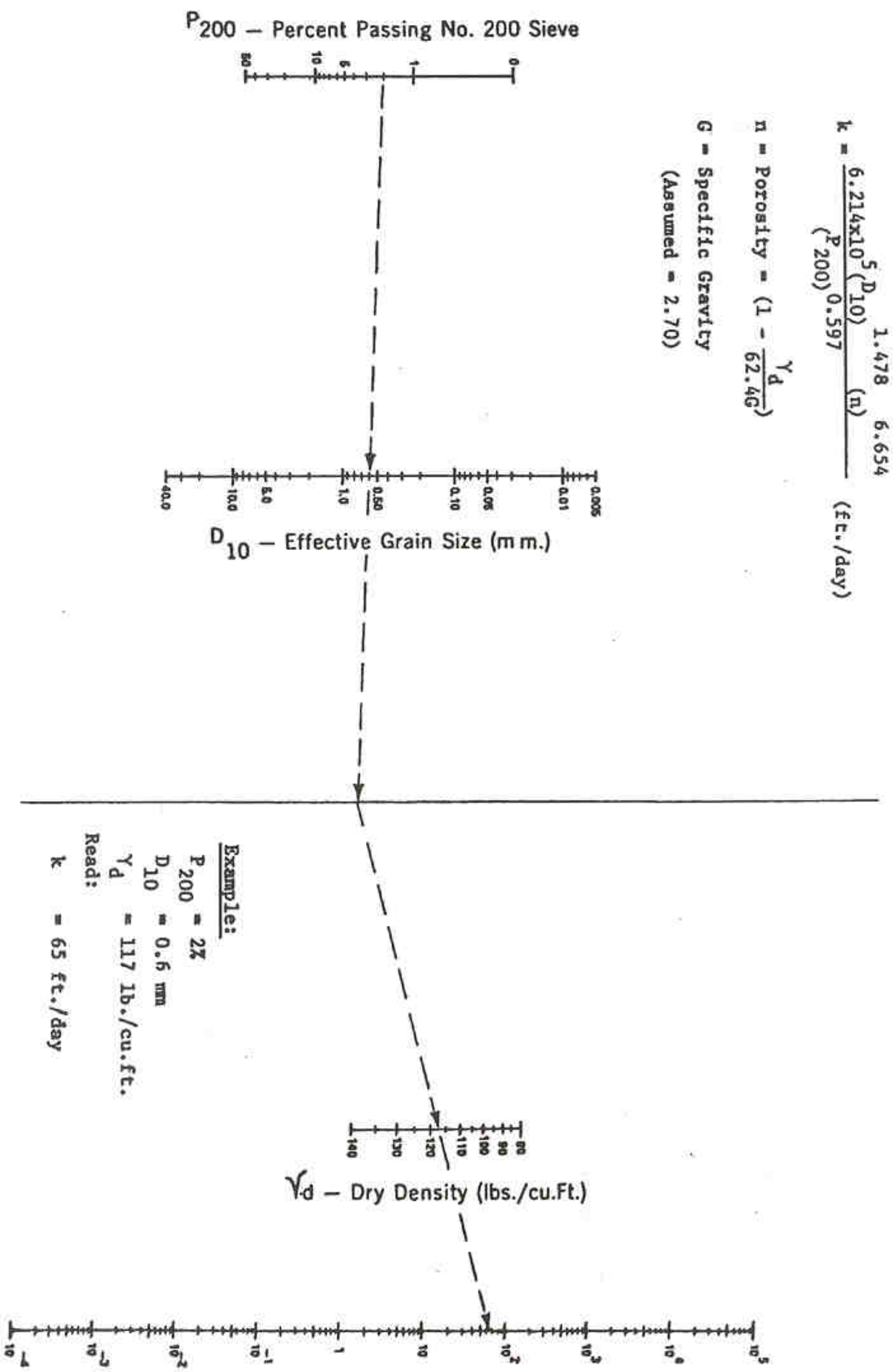
Finer soils can be expected to have lower permeabilities, and well-graded soils can be expected to be less permeable than more uniform soils. Furthermore, a decrease in permeability should be expected with increased dry density. Permeability values, as related to the soil's triangular textural and Unified classification, are given in Table 3-2.13. Figure 3-2.9 is a nomograph that may be used to estimate the coefficient of permeability of granular drainage and filter materials.

Table 3-2.13. Typical permeability values for various triangular textural classifications.

Triangular Textural Classification	Relative Permeability	Coefficient of Permeability (fpd)
Gravel	Pervious to very pervious	10^{2} - 10^{5}
Sand	Pervious	10^{0} - 10^{2}
Loamy Sand	Impervious to semi pervious	10^{-4} - 10^{-1}
Sandy Loam	Impervious to semi pervious	10^{-4} - 10^{0}
Loam	Impervious to semi pervious	10^{-4} - 10^{-3}
Silt Loam	Impervious to semi pervious	10^{-4} - 10^{-3}
Sandy Clay Loam	Impervious to semi pervious	10^{-4} - 10^{-2}
Clay Loam	Impervious	10^{-4} - 10^{-3}
Silty Clay Loam	Impervious	10^{-4} - 10^{-3}
Sandy Clay	Impervious	10^{-5} - 10^{-2}
Silty Clay	Very impervious to impervious	10^{-5} - 10^{-3}
Clay	Very impervious	10^{-7} - 10^{-5}

NOTE: Mn/DOT 3139, Class 5, Aggregate Base, typically has a coefficient of permeability between 1.1×10^{-4} and 1.8×10^{-4} cps (0.3 and 0.5 fpd.)

Figure 3-2.9. Chart for estimating coefficient of permeability of granular drainage and filter materials (from FHWA-TS-80-224, "Highway Subsurface Drainage," August 1980).



3-2.03.08 COMPRESSIBILITY

All soils compress when load is applied. Some, such as gravels and sands, compress very little. Others, such as those containing large amounts of silt and clay, compress significantly. When a load is applied to the latter group, water between the soil solids becomes pressurized and it flows out from the soil mass. As the water flows from the soil, the soil solids move closer together (Figure 3-2.4). As this happens the overall volume of the soil mass decreases, which is reflected as settlement or compaction on the surface. This movement is largely unrecoverable. Therefore, the soil mass will expand only a small percentage of the amount it had compressed should the load be removed.

Generally, natural silts and clays will not compress enough to be a major concern for highway projects. However, high fills, soft clays, peat, and muck are can settle to a problematic degree. Highly compressible areas should be studied by the Foundations Unit to determine the extent of the area and the compressibility of the material. Once the properties of the material are known, settlements can be calculated. If the anticipated settlements are too great, then the area can be pre-loaded (surcharged) by placing additional fill and monitoring the rate of settlement until it stops or slows to a tolerable pace.

Pre-loading is the surcharging of soft soils with additional fill to consolidate the underlying soils and offset settlements of the completed structure that would otherwise occur. Settlements of the original ground surface should be closely monitored during and after placement of the surcharge by the use of settlement plates. Settlement plates are installed at the ground surface before fill is placed. A section of pipe is attached to a flange on the plate and, as the fill is placed, additional sections of pipe are attached. A typical settlement plate installation is shown in Figure 3-2.10.

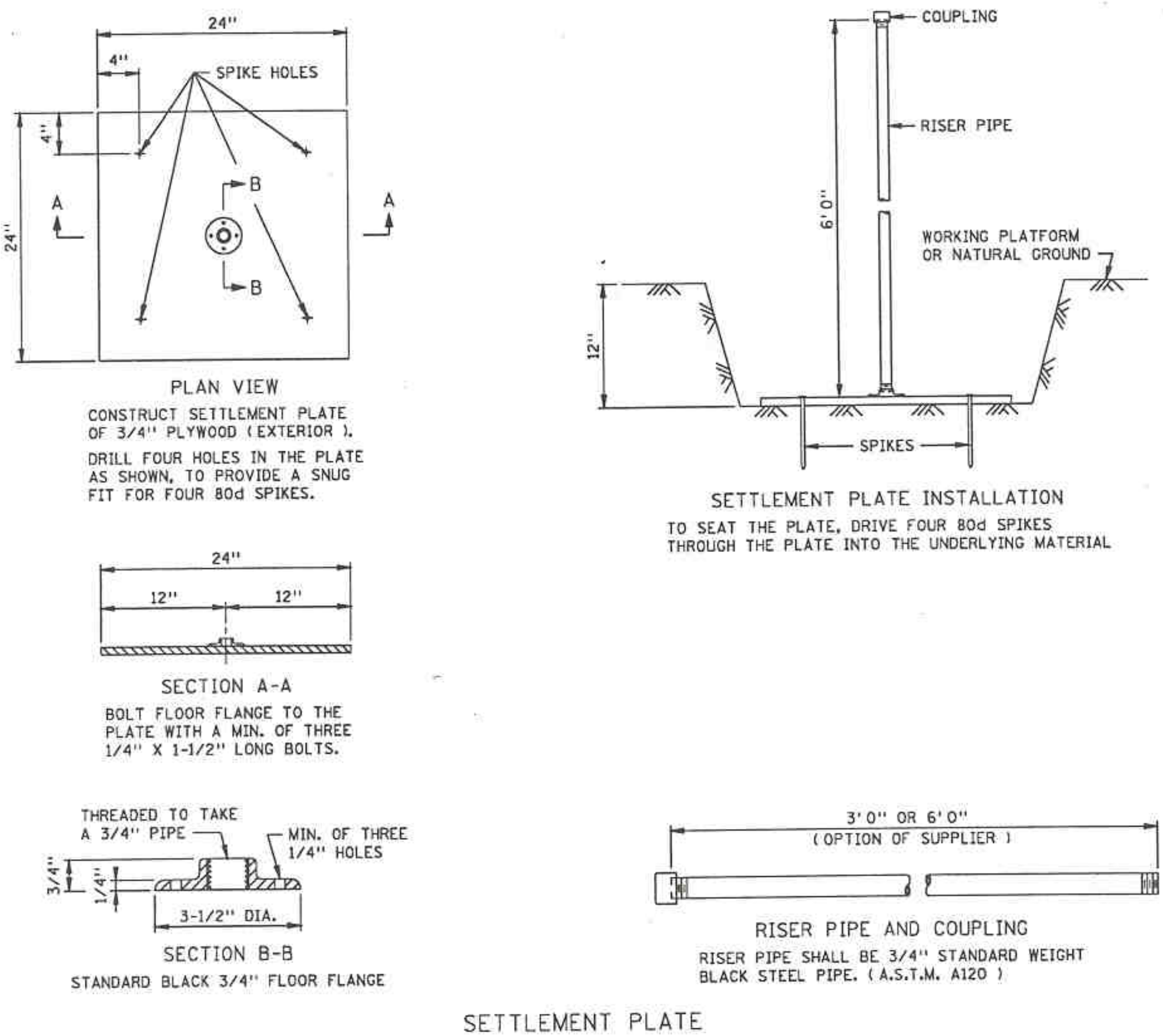
Field settlement data should be plotted against time in a graphical manner. Fill height at the settlement plate for the same reading date should be plotted on the same graph, but above the settlement data. Fill height will thus be a rising line, until the final height of fill is reached and the settlement data will be represented by a falling line with time.

Typical values of compressibility are shown in Table 3-2.14.

Table 3-2.14. Typical compressibility values.

Mn/DOT Triangular Textural Classification	Compressibility (%) @ 1.4 tsf	Compressibility (%) @ 2.8 tsf
Gravel	0.3 - 0.8	0.6 - 1.4
Sand	0.6 - 0.8	1.2 - 1.4
Loamy Sand	0.8 - 1.1	1.4 - 2.2
Sandy Loam	0.8 - 1.1	1.4 - 2.2
Loam	0.9 - 2.0	1.7 - 3.8
Silt Loam	0.9 - 2.0	1.7 - 3.8
Sandy Clay Loam	0.8 - 1.1	1.4 - 2.2
Clay Loam	0.9 - 2.6	1.7 - 3.9
Silty Clay Loam	0.9 - 2.0	1.7 - 3.8
Sandy Clay	0.8 - 1.1	1.4 - 2.2
Silty Clay	1.3 - 2.6	2.5 - 3.9
Clay	1.3 - 2.6	2.5 - 3.9

Figure 3-2.10. Typical settlement plate installation.



3-2.04 SAMPLE DESCRIPTION

A typical sample description includes the primary classification and additional descriptive information such as secondary descriptors, the AASHTO classification, and special terms describing the material's geologic origin. Each of these is discussed below.

3-2.04.01 PRIMARY CLASSIFICATION

The primary classification is taken from the triangular textural classification chart. It is one of the twelve groups described in Section 3-2.02.01.

3-2.04.02 SECONDARY DESCRIPTORS

The use of secondary descriptors should be restricted to color, moisture, consistency, compactness, structure, or cementation. Each of these items is highlighted below.

1. Color. A soil's color is usually described when the sample has a non-zero moisture content, because many soils fade to similar pastel shades when dry. The importance of color is

particular to a geologic area; however, certain colors, like black, gray, or green, are indicators of wet environments or organics and should be noted. Bright reds and yellows are indicative of particular mineral contents and a high oxidation environment. Acceptable colors are black, blue, brown, green, grey, red, tan, buff, white, and yellow optionally modified with intensity of the color (light or dark) or shading (reddish, brownish, etc.). Other descriptors include:

a. Mottled. Presence of spots, streaks or splotches of one or more colors in a soil mass of another predominant color. In mottled soils the colors are not mixed and blended, but each is more of less distinct in the general ground color.
b. Marbled. Presence of two or more distinct colors in approximately equal amounts but not blended. In a marbled soil, there is no general or predominant color, as is the case of mottled soil.

2. Moisture. The moisture of the soil should be described as dry, damp, moist, wet, or saturated. Dry soils are in a powdery condition, with no indication of moisture. Damp soils are well below their optimum moisture content. Moist soils are at or near their optimum moisture content and exhibit some color change with short periods of exposure to the air. Wet soils are between their optimum moisture content and saturation, with a high degree of moisture to the touch. Saturated soils cannot take on any additional water and may have free surface water.

3. Consistency. The strength of cohesive soils is quantified by their consistency. Terms utilized to describe consistency are very soft, soft, firm (sometimes referred to as medium stiff), stiff, very stiff, and hard. Consistency is often thought of as relating to plasticity, since in clays short term strength is based on cohesion; however, it is possible to have a very plastic soil (high cohesion) appear very soft.

The Standard Penetration Test (SPT), torvane, and pocket penetrometer tests are used to identify the consistency of clayey soils. However, the SPT (ASTM D-1586) is considered to be somewhat unreliable for cohesive soils. Table 3-2.15 provides the relationship for these test methods to determine the soil's consistency. It also includes a reference to the manual penetration test that can readily be performed by field personnel at the District level.

Table 3-2.15. Consistency of fine-grained soils.

Consistency Test	SPT (bpf)	Pocket Penetrometer (tsf)	Torvane (tsf)	Manual Penetration
Very Soft	< 2	0 - 0.25	0 - 0.125	Squeezes between fingers when fist is closed
Soft	2 - 4	0.25 - 0.50	0.125 - 0.25	Easily molded by finger
Firm	5 - 8	0.50 - 1.00	0.25 - 0.50	Molded by strong pressure of fingers
Stiff	9 - 15	1.00 - 2.00	0.50 - 1.00	Dented by strong pressure of fingers
Very Stiff	16 - 30	2.00 - 4.00	1.00 - 2.00	Dented only slightly by finger pressure
Hard	> 30	> 4.00	> 2.00	Dented only slightly by pencil point

4. Compactness. The strength of granular soils is quantified by their compactness. It is described as very loose, loose, medium dense (sometimes referred to as medium), dense, or very dense. Again, a sand with a high internal friction angle (indicative of high strength) may be encountered in a very loose condition, so associating strength, per se, with compactness is not necessarily correct.

   The SPT is widely used to evaluate the compactness of granular soils, as related in Table 3-2.16.

Table 3-2.16. Compactness of granular soils.

Compactness	SPT (bpf)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

   In the case of silts, it is probably better to associate terms of consistency rather than compactness, since silts are difficult to compact and behave, under many circumstances, similarly to low-plasticity clays.

5. Structure. The feel and appearance of the soil mass is its structure. It indicates the arrangement of individual soil grains, which often is not directly discernable to the human eye. Mn/DOT's Grading and Base Manual describes eight different structures in Section 5-692-604. These are cloddy, crumb, hardpan, clay pan, massive, laminated, mealy, and fluffy. Several other possible soil structures are described as follows:

   - **Fissured** soils break along definite fracture planes.
   - **Slickensided** soils have striated, polished, or glossy fracture planes.
   - **Blocky** soils are cohesive, with many, small, flat-sided blocks. When the soil mass is broken apart, the small blocks tend to retain their individual identity.
   - **Lensed** soils are different from laminated soils in that the lenses are thicker, distinct, soil layers in a more massive soil layer. They are, however, thin enough that they would not be considered a separate stratigraphic layer.
   - **Homogeneous** describes a consistent feel or appearance throughout the soil mass, whereas **heterogeneous** describes a varying feel or appearance.

6. Cementation. The adhesion or binding together of soil grains or aggregates by a cementing agent, such as colloidal clay, iron aluminum hydrates, lime carbonate, etc. When this occurs, the degree of cementation, from firm to soft, should be indicated.

3-2.04.03 SPECIAL TERMS

Additional descriptors, such as geologic origin or commonly accepted vernacular, may be added using special terms. Some special terms identifying particular soil types are topsoil, peat, quicksand, gumbo, diatomaceous earth, marl, alluvium, colluvium, loess, blow sand, volcanic ash, till, or glacial outwash. Most of the above terms are discussed in detail in Chapter Two of this manual and also in Mn/DOT's Grading and Base Manual.

As a minimum, the sample description should include the primary textural classification and two secondary descriptors: color and moisture condition. More complete descriptions are desirable, including the additional secondary descriptors listed above, the AASHTO classification, and any applicable special terms. Examples of proper field sample descriptions are:

- Minimum: SILTY CLAY, gray, wet.
- More Desirable: SILTY CLAY, gray, wet, soft, with fine sand laminations, A-6, lacustrine.

3-2.05 ABBREVIATIONS

Standard terms and abbreviations are desirable, both for saving time when describing soil samples and for ease of interpretation of field notes, profiles, etc., by others. The following terms and abbreviations (Table 3-2.17) are approved by the Department for use in describing samples and preparing field notes.

Table 3-2.17. Approved terms and abbreviations.

Soils Nomenclature

TERMS USED IN BORING LOGS

Heading/Location Terms	Abbv.
Trunk Highway	TH
Station	Sta
Reference Point	RP
Offset	OS
Left	LT
Right	RT
Centerline	C/L
Plus	+
Northbound	NB
Southbound	SB
Eastbound	EB
Westbound	WB
Feet	FT
Tenths	Tenths
kilometers	km
meters	m
millimeters	mm
Months	Mo
Matls Engr's Name	MEngr
Crew's Name	Crew
Individual Letters A thru Z. . (Upper Case)	
Numbers 0 thru 9	
Ramp	Ramp
Loop	Loop
Frontage Road	FR
Service Drive	SrDr
Source Name/Number	Srce
Mainline	ML
Shoulder	SHLD
State Project No	SP
Control Section	CS
County Road	CR
County State Aid Highway	CSAH
Soils Engr's Name	SEngr
Section	Sec
Township	Twp
Range	Rng
Decimal Point	
One-quarter	1/4
One-half	1/2
Three-quarters	3/4
Plus/Minus	+/-
Approximate	Approx
Question Mark	?
Inplace	Inp

Surfacing Terms

Concrete	CONC
Bituminous	BIT
Aggregate	AGG
Bit Treated Base	BTB

Material Terms

Gravel	G
Sand	S
Sand and Gravel	S&G
Loamy Sand	LS
Loamy Sand and Gravel	LS&G
Sandy Loam	SL
Loam	L
Silt	Si
Silt Loam	SiL
Silty Clay Loam	SiCL
Clay Loam	CL
Sandy Clay Loam	SCL
Clay	C
Silty Clay	SiC
Sandy Clay	SC

Boulder Terms

Limestone	Lmst
Sandstone	Sst
Dolostone	Dolo
Shale	Shale
Boulder (over 3")	Bldr

Moisture Terms

dry	dry
damp	damp
moist	moist
wet	wet
saturated	sat

Color & Shade Terms

black	blk
brown	brn
grey	gry
yellow	yel
tan	tan
blue	blu
white	wht
green	grn
red	red
orange	orng
dark	dk
light	lt

Textural Terms

Very Fine	VF
Fine	F
Coarse	Cr

Plasticity Terms

slightly plastic................................ slpl
nonplastic...................................... nonpl
plastic.. pl
highly plastic. .... hpl

Consistency Terms

Very soft.. Vsoft
soft... soft
firm .... firm
stiff... stiff
Very stiff.. Vstiff
hard .... hard
Very hard. .... Vhard

Compactness Terms

Very loose .... Vloose
loose... loose
medium dense .... meddense
dense .... dense
Very dense .... Vdense

Water Condition Terms

water level H2O
Flowing Artesian .... FlArt
perched water .... perch

Peat Classification Terms

Peat .... peat
spongy .... spongy
fiberous peat .... fpeat
semi fiberous peat.......................... sfpeat
well decomposed peat..................... wdpeat
partially decomposed peat .... pdpeat

Organic Content Terms

non organic .... nonorg
slightly organic .... slorg
organic .... org
highly organic................................ horg

Descriptors

Deteriorated .... Det
Stripped .... Strpd
Sound... Snd
Unsound... UnSnd
weathered....................................... wx
Bedrock .... bedrock
debris .... debris
chips... chips
seams .... seams
layers .... layers
marbled .... mrbl
mottled .... mtld
fill .... fill
cut .... cut
fat.. fat
frozen... frzn
ice lenses .... icelns
ice .... ice
topsoil .... ts
slope dressing .... sd
wood .... wood
woody .... woody
roots .... roots
shells .... shells
Iron Oxide Stained.......................... IOS
till.. till

Miscellaneous

with .... w/
without... w/o
variable .... var
natural .... nat
Not Applicable................................ N/A
and .... &
or... or
to .... to
included .... inc
Gas Smell....................................... GasSm
Road Tar .... RdTar
sample.. smpl
Soil ID .... SID
R-Value .... RVal
Gradation .... Grad
Fertility .... Fert
Extraction .... Xtract
at .... @
Time Of Drilling .... TOD
hour .... hr
no return. .... noret
poor return. .... prret
fluid .... fluid
REFUSAL .... REFUSAL

Equipment

Auger Truck .... AT
Hand Auger .... HA
50# Sounding Hammer................... 50SH
20# Sounding Hammer................... 20SH
Portable Auger............................... PA
Dynamic Cone Penetrometer.......... DCP

Other comments:

- no periods
- keep UPPER & lower case letters