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Guide for Mechanistic-Empirical Design

OF NEW AND REHABILITATED PAVEMENT STRUCTURES

FINAL REPORT

PART 2. DESIGN INPUTS

CHAPTER 1. SUBGRADE/FOUNDATION DESIGN INPUTS

NCHRP

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ARA, Inc., ERES Consultants Division
505 West University Avenue
Champaign, Illinois 61820

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PART 2—DESIGN INPUTS

CHAPTER 1
SUBGRADE/FOUNDATION DESIGN INPUTS

This chapter provides procedures and recommended guidelines for determining the design parameters of the subgrade soils or foundation for use in new pavement designs. The chapter is divided into four sections: characterization of pavement foundations, subsurface explorations, laboratory testing, and foundation improvements and strengthening.

2.1.1 CHARACTERIZATION OF THE PAVEMENT FOUNDATION

The foundation must be characterized, regardless of whether the design procedure is to be applied to an existing pavement or a new pavement. The support for new and existing pavements is the roadbed soil or embankment, since the thickness and properties of all layers above this level are to be determined or analyzed as part of the design process.

The basic input data set for characterizing the subgrade or foundation is the same for the design of both flexible and rigid pavements. If sufficient data are unavailable for characterizing the foundation, the pavement designer may use the default values provided in the Guide. This provision allows for the use of hierarchical design methodologies that were discussed in the previous section, thus minimizing agency design costs, but at the increased risk of over-designing the pavement structure.

Different means for subgrade or foundation characterization alternatives exist, including:

- Laboratory testing of undisturbed or reconstituted field samples recovered from the subsurface exploration process.
- Nondestructive testing of existing pavements found to have similar subgrade materials.
- Intrusive testing such as the Dynamic Cone Penetrometer (DCP)
- Reliance on an agency's experience with the subgrade type.

All of these alternatives are covered in the Guide; however, laboratory testing and nondestructive deflection testing (NDT) are recommended as the primary characterization methods. An agency's experience can and should supplement these two methods. NCHRP Synthesis 278, *Measuring In Situ Properties of Pavement Subgrade Soils (1)*, and NCHRP Synthesis 247, *Stabilization of Existing Subgrades to Improve Constructability During Interstate Pavement Reconstruction*, can be used to supplement the information presented *(2)*. The agency's experience should also be used to select subgrade improvement techniques for problem soils unique to their region, such as collapsible, expansive, frost-susceptible, and saturated soils.

This Guide also addresses potential differences between the alternative methods of characterizing the foundation. The potential differences between the resilient modulus of the foundation or subgrade soils, as determined by backcalculation techniques or measured in the laboratory, are presented and discussed in two FHWA reports (*3, 4*). The important point for characterizing the foundation is that the design values or inputs determined by different methods are different, and that difference must be recognized in the design process.

Support characterization, as used in this context, refers to the process of characterizing the properties of the existing soil strata and any new or existing materials that make up the pavement. These include the surface layers, base and subbase layers, and other special pavement features. Special details of characterization requirements for each pavement material are described in greater detail in PART 2, Chapter 2. The characterization techniques for the pavement materials and subgrade soils will be hierarchical, ranging from default values for the different materials and soils to comprehensive laboratory and field testing for critical project types.

Layered resilient modulus (specifically, resilient modulus or approximations of the modulus of elasticity or Young's modulus) is the property needed for pavement design and analysis. Three basic methods can be used to obtain the resilient modulus of each structural layer in the pavement:

- Laboratory repeated load resilient modulus tests.
- Analysis or backcalculation of NDT data.
- Correlations with other physical properties of the materials.

For all new designs, particularly for critical projects, repeated load resilient modulus tests are needed to evaluate the effects of changes in moisture on the resilient modulus of a particular soil. The latest version of AASHTO T 307, *Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils*, should be used for measuring the resilient modulus of a soil in the laboratory. However, NCHRP Project 1-28A resulted in a resilient modulus test procedure which can also be used. There are differences between the two procedures and the designer should be cautious to ensure that the values used are consistent with their local calibration procedures. For rehabilitation designs, however, the use of backcalculated elastic modulus to characterize the existing structure and foundation is suggested because it provides data on the response characteristics of the in situ soils and conditions. ASTM D4694 (*Deflections with a Falling Weight Type Impulse Load Device*) and D4695 (*Guide for General Pavement Deflection Measurements*) are standards that can be used for measuring the deflection basins along an existing roadway. ASTM D5858 (*Guide for Calculating In Situ Equivalent Elastic Moduli of Pavement Materials Using Layered Elastic Theory*) is a standard that can be followed for backcalculating layer elastic modulus from deflection basin data. Both laboratory and NDT procedures can be used to produce the resilient modulus of the foundation soils needed for design. The method used to determine the resilient modulus is discussed in PART 2, Chapter 2.

2.1.2 SUBSURFACE CHARACTERIZATION FOR PAVEMENT DESIGN

The horizontal and vertical variations in subsurface soil types, moisture contents, densities, and water table depths must be considered during the pavement design process. These elements can be quantified through the implementation of proper field (subsurface investigation) and laboratory testing programs. More importantly, special subsurface conditions, such as swelling soils and frost-susceptible soils, must be identified and considered in pavement design. This section of the Guide provides guidelines on how to identify and address these special subsurface conditions. Specifically, minimum recommendations are provided for determining the subsurface soil profile, conditions, and the design resilient modulus.

2.1.2.1 Subsurface Exploration

The objective of subsurface investigations or field exploration is to obtain sufficient subsurface data to permit the selection of the types, locations, and principal dimensions of foundations for all roadways comprising the proposed project, thus providing adequate information to estimate their costs. More importantly, these explorations should identify the site in sufficient detail for the development of feasible and cost-effective pavement designs.

The subsurface investigation for any project should be sufficiently detailed to define the depth, thickness, and area of all major soil and rock strata that will be affected by construction or impose special problems for the construction and long-term performance of the pavement structure. Disturbed and undisturbed samples of the subsurface materials must be obtained for laboratory analyses (and/or tested in the field) to determine their engineering properties. The extent of the exploration program depends on the nature of both the project and the site-specific subsurface conditions. Thus, to begin the process, a boring layout and sampling plan should be established to ensure that the vertical and horizontal profile of the different soil conditions can be prepared.

To acquire reliable engineering data, each job site must be explored and analyzed according to its subsurface conditions. The engineer in charge of the subsurface exploration must furnish complete data so that an impartial and thorough study of practical pavement designs can be made. Suggested steps are listed below:

1. Make a thorough investigation of the topographic and subsurface conditions.
2. Conduct exploratory borings at spacing and depth prescribed by the engineer. The spacing and depth of these borings depend on the variability of the existing soil conditions, both vertically and horizontally. These borings should also be used to determine the water table depth. Take sufficient and appropriate auger, split tube, or undisturbed samples of all representative subsoil layers. The soil samples must be properly sealed and stored to prevent moisture loss prior to laboratory testing. Prepare boring logs and soil profiles from this data.
3. Classify all soils using the AASHTO (or Unified) soil classification system. Table 2.1.1 relates the Unified soil classification of a material to the relative value of a material for use in a pavement structure.
4. Moisture-density tests should be used to determine the compaction characteristics for embankment and/or surface soils and untreated pavement materials. AASHTO T99, *Moisture-Density Relations of Soils Using a 2.5 kg (5.5 lb) Rammer and a 305 mm (12 in) Drop*, should be used for medium to high plasticity fine-grained soils, whereas, AASHTO T180, *Moisture/Density Relations of Soils Using a 4.54-kg [10-lb] Rammer and 457-mm [18-in] Drop*, should be used for coarse-grained and low plasticity fine-grained soils. The degree of compaction required for the in-place density should be expressed as a percentage of the maximum density from the specified test procedure.

Table 2.1.1. Summary of soil characteristics as a pavement material[1].

Major Divisions	Name	Strength when Not Subject to Frost Action	Potential Frost Action	Compressibility & Expansion	Drainage Characteristics
GW	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	None to very slight	Almost none	Excellent
Gravel and Gravelly	Poorly graded gravels or gravel-sand mixtures little or no fines	Good to excellent	None to very slight	Almost none	Excellent
GP Soils	Silty gravels, gravel-sand silt mixtures	Good to excellent	Slight to medium	Very slight	Fair to poor
*d		Good	Slight to medium	Slight	Poor to practically impervious
GM --- u GC	Clayey gravels, gravel-sand-clay mixture	Good	Slight to medium	Slight	Poor to practically impervious
SW	Well-graded sands or gravelly sands, little or no fines	Good	None to very slight	Almost none	Excellent
Sand and	Poorly graded sands or gravelly sands, little or no fines	Fair to good	None to very slight	Almost none	Excellent
SP Sandy Soils	Silty sands, sand-silt mixtures	Fair to good	Slight to high	Very slight	Fair to poor
		Fair	Slight to high	Slight to medium	Poor to practically impervious
*d SM --- u SC	Clayey sands, sand-clay mixtures	Poor to fair	Slight to high	Slight to medium	Poor to practically impervious
Silts & Clays with the Liquid Limit Less Than 50	Inorganic silts & very fine sand, rock flour, silty or clayey fine sand or clayey silts with slight plasticity	Poor to Fair	Medium to Very High	Slight to medium	Fair to Poor
	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Poor to Fair	Medium to High	Slight to medium	Practically Impervious
	Organic silts & organic silt-clays or low plasticity	Poor	Medium to High	Medium to high	Poor
Silts & Clays with Liquid Limit Greater Than 50	Inorganic silts, micaceous or diatomaceous fine sand or silty soils, elastic silts	Poor	Medium to Very High	High	Fair to Poor
	Inorganic clays of high plasticity, fat clays	Poor to Fair	Medium to Very High	High	Practically Impervious
	Organic clays of medium to high plasticity, organic silts	Poor to Very Poor	Medium	High	Practically Impervious
Highly Organic Soils	Peat & other highly organic soils	Not Suitable	Slight	Very high	Fair to Poor

[1] The information presented here is adapted after publications of the U.S. Army Corps of Engineers, Federal Aviation Administration, and the Federal Highway Administration.

5. Examine the boring logs, soil profiles, and classification tests and select representative soil layers for laboratory testing. Conduct repeated load resilient modulus tests to measure the resilient modulus over a range of stress states, in accordance with the latest version of AASHTO T 307. Determine the in situ resilient modulus for each major soil type encountered using the procedure documented in report FHWA-RD-97-083 *(5)*.
6. Use the soil profile along the roadway alignment to relate resilient modulus to each soil strata encountered. Select a design subgrade resilient modulus that is representative of each soil type and depth. The design resilient modulus and other design inputs are dependent on the response model used and subgrade soil constitutive equation. These are discussed in PART 2, Chapter 2 of this Guide. The standard penetration and dynamic cone penetrometer tests are also allowed because they can provide additional information to determine the in situ strength characteristics of the subsurface soils.

2.1.2.2 Boring Location and Depth

Regardless of the type of project, the borings should be spaced to establish in reasonable detail the stratigraphy of the subsurface materials. Borings should also be located to obtain a basic knowledge of the engineering properties of the overburden and bedrock formations that will be affected by, or will have an effect on, the proposed pavement structure, and to locate and determine the quality and approximate quantity of construction materials, if required.

2.1.2.3 Number or Spacing of Borings

The number and spacing of the borings should be consistent with the type and extent of the project and with the nature of the subsurface conditions. Rigid rules for the number and spacing of the borings cannot and should not be established. In general, emphasis should be placed on locating the borings to develop typical and representative geologic cross sections. The spacing of the borings is dependent on the subsurface variability of the project site, and it typically varies from 500 to 1,500 ft.

The U.S. Department of Agriculture's Natural Resources Conservation Service, in cooperation with State agricultural experiment stations and other Federal and State agencies, has been making soil surveys and publishing them since 1899. An important product of this effort is county soil maps, which provide an overview of the spatial variability of the soil series within a county. Such information will be of help in planning soil exploration activities.

2.1.2.4 Depth of Borings

Just as rigid rules cannot be established for the spacing of borings, one also cannot establish hard-and-fast rules for determining the depth to which the borings are drilled. However, general guidelines are available for planning explorations. Two major factors control the depth of exploration: the magnitude and distribution of the traffic loads imposed on the pavement structure under consideration, and the nature of the subsurface conditions.

The planned exploration depths along the alignment of a highway depend on the knowledge of the subsurface conditions as based on geological soil surveys and previous explorations and the planned profile of the pavement surface. In areas of light cut and fill with no special problems, explorations should extend to a minimum of 5 ft below the proposed subgrade elevation. Some borings should extend to a depth 20 ft below the planned surface elevation. However, where deep cuts are to be made, large embankments are to be constructed, or subsurface information indicates the presence of weak (or water-saturated) layers, the boring depth should be increased. In those cases, the borings should be deep enough to provide information on any materials that may cause problems with respect to stability, settlement, and drainage.

All borings should extend through unsuitable foundation strata (for example, unconsolidated fill, highly organic materials, or soft, fine-grained soils) to reach relatively hard or compact materials of suitable bearing capacity. Borings in potentially compressible fine-grained strata of great thickness should extend to a depth where the stress from superimposed traffic loads or a thick embankment is so small that consideration will not significantly influence surface settlement. Where stiff or compact soils are encountered at the surface and the general character and location of rock are known, borings should extend into sound rock. Where the location and character of rock are unknown or where boulders or irregularly weathered materials are likely to be found, the boring penetration into rock should be increased.

2.1.2.5 Type of Samples and Sample Recovery

The majority of the samples taken will be the disturbed type, such as those obtained by split barrel samplers. This will permit visual identification and classification of the soils encountered, as well as identification by means of grain size, water content, and Atterberg limit tests.

Sampling at each boring location may be either continuous or intermittent. In the former case, samples are obtained throughout the entire length of the hole; in the latter (primarily used in areas of deep cuts), samples are taken about every 5 ft and at every change in material. Initially, it is preferable to have a few holes with continuous sampling so that all major soil strata present can be identified. Every attempt should be made to obtain 100 percent recovery where conditions warrant. The horizontal and vertical extent of these strata can then be established by intermittent sampling in later borings, if needed.

To obtain a basic knowledge of the engineering properties of the soils that will have an effect on the design, undisturbed samples (such as those obtained with thin-wall samplers or double tube core barrel rock samplers) should be taken, if at all possible. The actual number taken should be sufficient to obtain information on the shear strength, consolidation characteristics, and resilient modulus of each major soil stratum. If undisturbed samples cannot be recovered, disturbed samples should be recovered. Disturbed samples are obtained with split barrel samplers. Disturbed samples permit visual identification and classification of the materials encountered, as well as identification by means of grain size, water content, and Atterberg limit tests.

Undisturbed samples should comply with the following criteria:

1. The sides of the samples should be straight or perpendicular to a horizontal plane, and contain no visible distortion of strata, horizontal cracks from the extrusion process, or softening of materials.

2. Specific recovery ratio (length of undisturbed sample recovered divided by length of sampling push) should exceed 95 percent.
3. The samples should be taken with a sampler with an area ratio (cross sectional area of sampling tube divided by full area or outside diameter of sampler) less than 15 percent.

At least one representative undisturbed sample should be obtained in cohesive soil strata, in each boring for each 5 ft depth, or just below the planned surface elevation of the subgrade. Recommended procedures for obtaining undisturbed samples are described in AASHTO Standard T 207, *Thin-Walled Tube Sampling of Soils*. All samples (disturbed and undisturbed) and cores should be wrapped or sealed to prevent any moisture loss, placed in protective core boxes, and transported to the laboratory for testing and visual observations.

2.1.3 LABORATORY TESTING OF SUBGRADE SOILS

Once in the laboratory, soil samples should be reviewed and identified for classification and resilient modulus testing. Undisturbed specimens should be free of visual defects and represent their natural conditions (moisture content and density). For disturbed or reconstituted specimens, bulk material should be recompacted to as close to the natural conditions as possible.

A program of laboratory tests shall be carried out on representative samples of the foundation soils or soils to be used as construction materials so that pertinent properties can be determined. The extent of the laboratory program depends on criticality of the design and on the complexity of the soil conditions. Those laboratory tests and analyses that are typically performed or required for an analysis and selection of the pavement type and thickness are listed in table 2.1.2.

Table 2.1.2. Minimum laboratory testing requirements for pavement designs.

Type of Laboratory Test	Deep Cuts	High Embankments	At-Grade
Moisture Content and Dry Unit Weight	X		X
Atterberg Limits	X	X	X
Gradation		X	X
Shrink Swell	X		X
Permeability	X		
Consolidation		X	
Shearing and Bearing Strength	X	X	X
Resilient Modulus	X	X	X

2.1.3.1 Number of Test Specimens

The number of test specimens depends on the number of different soils identified from the borings, as well as the condition of those soils. Most of the test specimens should be taken from as close to the top of the subgrade as possible to a depth of 2 ft below the planned subgrade elevation. However, some tests should be performed on the soils encountered at a greater depth, especially if those deeper soils are softer or weaker. No guidelines are provided regarding the number of tests, except that all of the major soil types encountered near the surface should be tested with replicates, if possible. Stated simply, resilient modulus tests should be performed on any soil type that may have a impact on pavement performance.

Another important point to remember in selecting the number of specimens to be tested is that the resilient modulus measured from repeated load tests can be highly variable. A coefficient of variation exceeding 25 percent for the resilient modulus measured at the same stress-state is not uncommon. This potential high variability in test results requires a greater number of tests (i.e., more than two resilient modulus tests for the same soil type). As a general guide, three resilient modulus tests should be performed on each major subgrade soil found along the highway alignment. If the variability of test results (resilient modulus measured at the same stress-state) exceeds a coefficient of variation of 25 percent, then additional resilient modulus tests should be performed to obtain a higher confidence in the data.

2.1.3.2 Types of Laboratory Tests

<u>Classification Tests</u>

All samples should be visually classified when they are received in the laboratory, and the natural water contents should be measured unless the samples are clean sands and gravels. The descriptive classification and natural water content data are the basis for plotting boring log profiles. Normally, identification tests consisting of Atterberg limits and sieve analysis will be performed on a sufficient number of representative samples from the borings to show the general variation of these properties within the foundation strata. The following lists the tests to be conducted, as a minimum, to classify each major soil stratum:

- AASHTO T 87, *Dry Preparation of Disturbed Soil and Soil Aggregate Samples for Test*.
- AASHTO T 88, *Particle Size Analysis of Soils*.
- AASHTO T 89, *Determining the Liquid Limit of Soils*.

These identification tests also enable the data to be used in correlating the results of more expensive shear and consolidation tests. Samples selected for Atterberg limits tests and sieve analyses should be from representative locations to ensure that the optimum amount of information is obtained from the tests. At least one Atterberg limits and gradation test should be performed on each major soil stratum.

<u>Shrink-Swell Tests</u>

When pavement distress resulting from the swelling or shrinkage of subgrade soils may be a problem, the tests should be performed to simulate, as closely as possible, the loading sequence anticipated in the field. Thus, to determine the swell at any depth, the specimen is permitted to swell. Tests to determine volume changes due to shrinkage are normally performed as a volumetric or linear-shrinkage determination. AASHTO T 92, *Determining the Shrinkage Factors of Soils*, can be used to measure the linear shrinkage of the soil while AASHTO T 258, *Determining Expansive Soils*, can be used to identify expansive soils and determine the amount of swell under different conditions.

Permeability Tests

Laboratory permeability tests are very seldom justified in pavement foundation problems. A possible exception is when assessing the free-draining capabilities of the existing foundation for making decisions on whether horizontal drainage is required. When laboratory values of permeability are required (primarily for conditions where de-watering may be needed), tests should be performed on representative samples of the foundation strata. These tests should be performed in accordance with AASHTO T 215, *Permeability of Granular Soils (Constant Height)*.

Consolidation Tests

Consolidation tests should be performed on representative samples from the various compressible foundation strata whenever settlement is a significant factor and the amount of settlement cannot be estimated from existing correlations. Samples for consolidation tests should be selected from the middle of each compressible stratum. Information should be given as to the estimated overburden pressure, excavation pressure, and loading sequence. Test loads should be sufficiently high to define the straight-line portion of the pressure–void ratio curve on a semi-logarithmic diagram. These tests should be performed in accordance with AASHTO T 216, *One Dimensional Consolidation Properties of Soils*.

Shearing and Bearing Strength Tests

Samples selected for shear tests should be located whenever possible near zones in which failure may be expected to occur. To analyze the stability of proposed excavation slopes, the tests should be performed on those strata in which the critical failure surface is assumed to be located. In deep clay deposits, shear failures may occur to great depths, and a sufficient number of shear tests should be performed to determine the strength of the deeper strata.

For most pavement foundation problems, it is suggested that unconfined compression or unconsolidated undrained triaxial tests be conducted on samples of cohesive soils; consolidated undrained triaxial tests should be conducted on silts and soils intermediate between sands and clay to determine strength characteristics. For clean sands, the cohesion can be assumed equal to 0, and the friction angle determined with a reasonable degree of accuracy from the results of standard penetration test data. The exact values of cohesion and the friction angle can also be determined from the results of triaxial testing on clean sands.

The following lists those tests that can be used to measure the strength properties of the soil strata:

- AASHTO T 223, *Field Vane Shear Test in Cohesive Soils*.
- AASHTO T 296, *Standard Method of Test for Unconsolidated, Undrained, Compressive Strength of Cohesive Soils in Triaxial Compression*.
- AASHTO T 297, *Standard Method of Test for Consolidated Undrained Triaxial Compression Test on Cohesive Soil*.

Another type of test that can be performed in the field to measure the strength of soils in-place is the dynamic cone penetrometer (DCP) test. This test is being used more commonly to estimate the in-place strength of both fine- and coarse-grained soils. The principle behind the DCP is that a direct correlation exists between the strength of a soil and its resistance to penetration by solid objects (*1*). The DCP consists of a cone attached to a rod that is driven into soil by means of a drop hammer that slides along the penetrometer shaft. The mass of the hammer can be adjusted to between 10 and 17.6 lb, with the lighter weight applicable for weaker soils. More recent versions of the DCP have a cone angle of 60 degrees and a diameter of 0.787 inches (after (*1*)). A number of relationships exist that relate the DCP penetration index (DPI) to subgrade strength parameters required for mechanistic-empirical design. ASTM recently standardized this test, as ASTM D6951-03, *Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications*.

Repeated Load Resilient Modulus Tests

Repeated load resilient modulus tests are conducted on each major soil stratum to characterize the subgrade soils for pavement design purposes. This test should be performed in accordance with the latest version of AASHTO T 307 or the NCHRP 1-28A, *Harmonized Test Method for Soils and Unbound Materials*. The purpose of this test procedure is to determine the nonlinear modulus properties (stress-sensitive modulus) for the soils or foundation materials in a condition that simulates the actual response of the soils to applied wheel loads.

The test is similar to the standard triaxial compression test, except that the vertical stress is cycled at several levels to model wheel load intensity and duration typically encountered in pavements under a moving load. A repetitive load triaxial machine should be used, as it provides a capability to simulate field conditions. Parameters that are varied include vertical stress, lateral (or confining) pressure, load period from 0.1 second upward, rest period between cyclic loads on the specimen, sequence of loading, and cycles of loading prior to reading the test values.

The nonlinear elastic coefficient and exponents of the soil constitutive equation should be determined for each test specimen using standard regression techniques to ensure that the multiple correlation coefficient exceeds 0.90. The constitutive equation for all unbound granular base and subbase materials and subgrade soils is given in PART 2, Chapter 2. The repeated load resilient modulus test results from similar soils and test specimen conditions can be combined. The condition of the test specimen is discussed in the next section.

When a finite element method is used for the structural response model, the K-values derived from the laboratory tests are the inputs to that structural response model. When elastic layer theory is used, however, one resilient modulus value is used to characterize the subgrade soil. To determine the in situ resilient modulus from repeated load triaxial compression tests, the total lateral and vertical stresses must be estimated and include the at-rest earth pressure. To determine these values, the density and thickness of each pavement layer and soil stratum in the trial cross section must be assumed. A step-by-step procedure that can be used to estimate the in situ stress condition is provided later in this chapter.

Special Tests

It is often necessary to determine special soil properties, such as organic content and carbonate content. The scope of these tests will depend largely on the amount and type of information required, and no general rules can be established for these types of tests. Ground water and soil in some areas may contain sulfates in amounts sufficient to cause damage to portland cement concrete (PCC) in rigid pavements. Sulfates usually are found in clayey soils and in acidic waters found in peat. A chemical analysis is desirable to determine whether special precautions are necessary. Some soils have a corrosive action on metals due to chemical and bacterial agents, and special laboratory tests are available for determining this property.

2.1.3.3 Condition of Resilient Modulus Laboratory Test Specimens

The condition of test specimens refers to the dry density and moisture content of the specimen. Two types of laboratory test specimens can be used in determining the resilient modulus of the foundation: undisturbed and disturbed (recompacted). Undisturbed soil samples taken with thin-walled Shelby tubes should be used whenever possible, especially for soil strata below a depth that is not altered by construction operations. For undisturbed test specimens, the dry density and moisture content are the same as found during the sampling operation. Unfortunately, the variability in test results between undisturbed specimens of the same soil type can be quite high, because of the difference in dry densities and moisture contents of the soil that can exist along a roadway (both vertically and horizontally). Increased variability will require increased testing frequencies to be confident in the data.

More importantly, the moisture content of some fine-grained soils may increase significantly after pavement construction. For this case, the resilient modulus measured at the moisture content during sampling may not be representative of the actual condition several years after construction. This potential change must be considered in the assessment of the design resilient modulus for pavement structural design. This condition is discussed in more detail in PART 2, Chapter 3.

For the surface soil strata that are remixed and recompacted prior to the placement of a pavement layer, disturbed samples should be used in the resilient modulus test program. Remixing and recompacting undisturbed test specimens (especially for some clays), even at the same moisture content and dry density, can significantly alter the resilient modulus test results, as compared to undisturbed test specimens. Test specimens can be compacted in the laboratory to the same dry density, but at different moisture contents for resilient modulus testing. The resilient modulus can then be determined directly for varying moisture contents.

Obviously, the moisture content can be measured on soil samples recovered from the borings. The important question to be answered is: What will the moisture content be for a particular season or time? This is a difficult question to answer at even a moderate confidence level.

The density used in the resilient modulus test program should be the in situ density after construction. The moisture content of soils beneath pavement structures does vary seasonally, and it is the parameter most difficult to predict. For some cohesionless soils, the moisture content might decrease and increase from the optimum moisture content depending on the surface and subsurface drainage characteristics, and the amount of rainfall at the site. For some cohesive soils (such as expansive clays), the moisture contents below a pavement tend to increase to values above optimum. The Enhanced Integrated Climatic Model (EICM) estimates the seasonal variation in moisture content of the subgrade soil, as well as in an unbound pavement layer *(6)*. Thus, the moisture content to be used in the resilient modulus test should be representative of the moisture condition at construction.

2.1.3.4 Selection of In Situ Resilient Modulus for Soil Strata

To determine the in situ resilient modulus from laboratory repeated load triaxial compression tests, the total lateral and vertical stresses must be known and include the at-rest earth pressure. To determine these values, the density and thickness of each pavement layer and soil stratum above the point of resilient modulus determination must be known or assumed. The following is a step-by-step procedure that can be followed.

1. Estimate the at-rest earth pressure coefficient, k_o, for the soil stratum for which the resilient modulus is needed. For cohesive soils (such as clays), the at-rest earth pressure coefficient is normally considered to be a function of Poisson's ratio, μ, and is:

$$k_o = \frac{\mu}{(1-\mu)} \tag{2.1.1}$$

   For non-cohesive soils (such as gravel and sand), the at-rest earth pressure coefficient is a function of the angle of shearing resistance, ϕ, and is:

$$k_o = 1 - \sin\phi \tag{2.1.2}$$

   For compacted and heavily, over-consolidated soils, the at-rest earth pressure coefficient is generally greater than the values computed with the above equations.

2. Assume the elastic modulus and thickness for each layer in the trial pavement structure, including the soil strata tested in the laboratory.

3. Compute the total vertical stress, σ_Z, above the point of resilient modulus determination.

$$\sigma_Z = \sigma_l + p_o \tag{2.1.3}$$

   Where:
   σ_l = vertical stress from the wheel load computed with elastic layer theory.
   p_o = at-rest vertical pressure from the overburden of other layers.

$$p_o = (D\gamma)_n + \sum_{i=1}^{n-1}(D\gamma)_i \tag{2.1.4}$$

where:

D	=	thickness of layer; for layer n, D is the depth of characterization or where the resilient modulus is determined.
γ	=	density of layer.
i	=	layer above the soil strata, n, for which the resilient modulus is being estimated.

4. Compute the total lateral stress, σ_3, on the element of soils at the depth for determining the resilient modulus of the soil strata.

$$\sigma_3 = \sigma_{x,y} + k_o(p_o) \tag{2.1.5}$$

where:

$\sigma_{x,y}$	=	horizontal stress from the wheel loads applied at the pavement surface and computed with elastic layer theory.

5. Compute the resilient modulus for the total vertical and horizontal stresses using the constitutive relationship (equation 2.2.36).

6. Compare the assumed resilient modulus to the computed value. If the computed stresses result in a value within 5 percent of the resilient modulus measured in the laboratory, then the value can be used as the resilient modulus at construction.

2.1.3.5 Reporting of Test Results

The results of the explorations and laboratory testing are usually presented in the form of a geology and soils report. This report should contain sufficient descriptions of the field and laboratory investigations performed, the conditions encountered, typical test data, basic assumptions, and the analytical procedures utilized, to permit a detailed review of the conclusions, recommendations, and final pavement design. The amount and type of information to be presented in the design analysis report should be consistent with the scope of the investigation. For pavements, the following items (when applicable) should be included and used as a guide in preparing the design analysis report:

1. A general description of the site, indicating principal topographic features in the vicinity. A plan map showing surface contours, the locations of the proposed structure, and the location of all borings.
2. A description of the general geology of the site, including the results of any previous geological studies performed.
3. The results of field investigations, including graphic logs of all foundation borings, locations of pertinent data from piezometers (when applicable), depth to bedrock, and a general description of the subsurface materials, based on the borings. The boring logs or report should indicate how the borings were made, type of sampler used, and any penetration test results, or other field measurement data taken on the site.
4. Ground water conditions, including data on seasonal variations in ground water level and results of field pumping tests, if performed.

5. A general description of laboratory tests performed, the general range of test values, and detailed test data on typical samples. Laboratory test data should be summarized in tables, including the resilient modulus selected for each soil stratum. If laboratory tests were not performed, the basis for determination of soil properties should be presented, such as empirical correlations or reference to pertinent publications.
6. A generalized soil profile used for design, showing average or representative soil properties and values of design shear strength used for various soil strata. The profile may be described in writing or shown graphically.
7. Recommendations on the type of pavement structure and any special design feature to be used, including removal and replacement of certain soils, stabilization of soils or other foundation improvements and treatments.
8. Basic assumptions, imposed wheel loads, results of any settlement analyses, and an estimate of the maximum amount of swell to be expected in the subgrade soils. The effects of the computed differential settlement, and also the effects of the swell on the pavement structure should be discussed.
9. Special precautions and recommendations for construction techniques should be discussed. Locations at which material for fill and backfill can be obtained should also be stated. The amount of compaction required and procedures planned for meeting these requirements should be described.

In summary, the horizontal and vertical variations in subsurface soil types, moisture contents, densities and water table depths should be identified for both new and existing pavements. FHWA Report No. FHWA-RD-97-083 (*5*) provides general guidance and requirements for subsurface investigations for pavement design and evaluations for rehabilitation designs. Each soil stratum encountered should be characterized for its use to support pavement structures and whether the subsurface soils will impose special problems for the construction and long-term performance of pavement structures. The following provides guidance on identifying problem soils or conditions and different methods that can be used to treat or mitigate those detrimental effects.

2.1.4 IDENTIFICATION AND TREATMENT OF SPECIAL SUBSURFACE CONDITIONS

Proper treatment and preparation of the subgrade soil (or foundation) is extremely important for a long-lasting pavement structure. Rather than consider the increase in roughness from differential frost heave or from expansive soils, this Guide provides ways to minimize the effects from these problematic conditions. The Guide provides detailed guidelines for identifying and treating problem soils to achieve an adequate foundation on which to build the pavement structure. Four special subsurface conditions are addressed:

- Collapsible or highly compressible soils.
- Expansive or swelling soils.
- Subsurface water flow and saturated soils.
- Frost-susceptible soils.

2.1.4.1 Compressible Soils

<u>Effect of Compressible Soils on Pavement Performance</u>

Collapsible or highly compressible (very weak) soils are susceptible to large settlements and deformations with time that can have a detrimental effect on pavement performance. If these compressible soils are not treated properly, large surface depressions with random cracking can develop. The surface depressions can allow water to pond on the pavement's surface and more readily infiltrate the pavement structure, compounding a severe problem. More importantly, the ponding of water will create a safety hazard to the traveling public.

<u>Treatments for Compressible Soils</u>

Where compressible soils exist, the selection of a particular technique depends on the depth of the weak soil, and the difference between the in situ conditions and the minimum compaction or strength requirements to limit the amount of anticipated settlement to a permissible value that will not adversely affect pavement performance.

When constructing roadways in areas with deep deposits of highly compressible layers (very low density–saturated soils), the specific soil properties must be examined to calculate the estimated settlement. Under these conditions, a geotechnical investigation and detailed settlement analysis must be completed prior to the pavement design. When existing subgrade soils do not meet minimum compaction requirements and are susceptible to large settlements over time, consider the following alternatives:

- If the compressible layer is shallow, remove and process soil to attain the approximate optimum moisture content and replace and compact.
- Remove and replace subgrade soil with suitable borrow or select embankment materials. All granular fill materials should be compacted to at least 95 percent of the maximum density, as defined by AASHTO T 180. Cohesive fill materials should be compacted to no less than 90 percent, as defined by AASHTO T 99.
- Compact soils from the surface to increase the dry density through dynamic compaction techniques.
- If the soil is extremely wet or saturated, place vertical sand wicks or deep horizontal drains to de-water the soils.
- Consolidate deep deposits of very weak saturated soils with large fills prior to pavement construction. After construction, the fills can either be left in place or removed, depending on the final elevation.

2.1.4.2 Swelling Soils

<u>Effect of Swelling Soils on Pavement Performance</u>

Swelling or expansive soils are susceptible to volume change (shrink and swell) with seasonal fluctuations in moisture content. The magnitude of this volume change is dependent on the type of soil (shrink-swell potential) and its change in moisture content. A loss of moisture will cause the soil to shrink, while an increase in moisture will cause it to expand or swell. This volume change of clay type soils can result in longitudinal cracks near the pavement's edge and significant surface roughness (varying swells and depressions) along the pavement's length.

Expansive soils are a very significant problem in many parts of the United States and are responsible for the application of premature maintenance and rehabilitation activities on many miles of roadway each year. Expansive soils are especially a problem when deep cuts are made in a dense (over-consolidated) clay soil.

Identification of Swelling Soils

Various techniques and procedures exist for identifying potentially expansive soils. AASHTO T 258 can be used to identify soils and conditions that are susceptible to swell. Two of the more commonly used documents are *An Evaluation of Expedient Methodology for Identification of Potentially Expansive Soils* and *Design and Construction of Airport Pavements on Expansive Soils* (*7, 8*).

Clay mineralogy and the availability of water are the key factors in determining the degree to which a swelling problem may exist at a given site. Different clay minerals exhibit greater or lesser degrees of swell potential based on their specific chemistry. Montmorrillonitic clays tend to exhibit very high swell potentials due to the particle chemistry, whereas illitic clays tend to exhibit very low swell potentials. Identification of clay minerals through chemical or microscopic means may be used as a method of identifying the presence of high swell potential in soils. The soil fabric will also influence the swell potential, as aggregated particles will tend to exhibit higher swell than dispersed particles, and flocculated higher than deflocculated. Generally, the finer-grained and more plastic the soil, the higher the swell potential the soil will exhibit.

The identification of swelling soils beneath the pavement is a key component of the geotechnical investigation for the roadway. Soils at shallow depths beneath the proposed pavement elevation are generally sampled as part of the investigation, and their swell potential may be identified in a number of ways. Index testing is a common method for identifying swell potential. Laboratory testing to obtain the plastic and liquid limits and or the shrinkage limit will usually be conducted. The soil activity, defined as the ratio of the plasticity index to the percentage of the soil by weight finer than 80 mils is also used as an index property for swell potential, since clay minerals of higher activity exhibit higher swell. Activity calculation requires measurement of gradation using hydrometer methods, which is not typical in geotechnical investigations for pavement design in many jurisdictions. In addition to index testing, agency practice in regions where swelling soils are a common problem may include swell testing, for natural or compacted soil samples. Such testing generally includes measurement of the change in height (or volume) of a sample exposed to light loading similar to that expected in the field and then allowed free access to water.

Treatment for Swelling Soils

When expansive soils are encountered along a project in environments and areas where significant moisture fluctuations in the subgrade are expected, consideration should be given to the following alternatives to minimize future volume change potential of the expansive soil:

- For relatively thin layers of expansive clays near the surface, remove and replace the expansive soil with select borrow materials.
- Extend the width of the subsurface pavement layers to reduce the loss of subgrade moisture along the pavement's edge.
- Scarify, stabilize, and recompact the upper portion of the expansive clay subgrade. Lime stabilization is an accepted method for controlling the swelling of soils. (*Stabilization*, as used here, refers to the treatment of a soil with such agents as asphalt, portland cement, slaked or hydrated lime, and fly ash to limit its volume change characteristics. This can substantially increase the strength of the treated material.) Further discussion on soil stabilization is provided later in this chapter.
- In areas with deep cuts in dense, over-consolidated expansive clays, complete the excavation of the subsurface soils to the proper elevation and allow the subsurface soils to rebound prior to placing the pavement layers.
- If left in place, compact moderately to highly expansive soils above optimum moisture content.

2.1.4.3 Subsurface Water

Effect of Subsurface Water on Pavement Performance

It is important to identify any saturated soil strata, the depth to ground water, and subsurface water flow between soil strata. Subsurface water is especially important to recognize and identify in the transition areas between cut and fill segments. If allowed to saturate unbound base/subbase materials and subgrade soils, subsurface water can decrease the strength and modulus of these materials and soils significantly. Significant reductions in strength can result in premature surface depressions, rutting, or cracking. Seasonal moisture flow through selected soil strata can also significantly magnify the effects of differential volume change in expansive soils. Cut areas are particularly critical for subsurface water.

Treatments for Subsurface Water

When saturated soils or subsurface water are encountered, consideration should be given to the following alternatives for improving the foundation or supporting subgrade:

- For saturated soils near the surface, dry or strengthen the wet soils through the use of mechanical stabilization techniques to provide a construction platform for the pavement structure.
- Remove and replace the saturated soils with select borrow materials or soils.

- Place and properly compact thick fills or embankments to increase the elevation of the subgrade, or in other words, increase the thickness between the saturated soils or water table depth and pavement structure.
- Use subgrade drains whenever the following conditions exist:
  - High ground-water levels that may reduce subgrade stability and provide a source of water for frost action.
  - Subgrade soils of silts and very fine sands that may become quick or spongy when saturated.
  - Water seeps from underlying water-bearing strata or from subgrades in cut areas.

2.1.4.4 Frost-Susceptible Soils

Effect of Frost Action on Pavement Performance

Frost action can cause differential heaving, surface roughness and cracking, blocked drainage, and a reduction in bearing capacity during thaw periods. These effects range from slight to severe, depending on types and uniformity of subsoil and the availability of water.

One effect of frost action on pavements is frost heaving caused by crystallization of ice lenses in voids of soils containing fine particles. Three conditions must be present to cause frost heaving and other frost action problems:

- Frost-susceptible soils.
- Subfreezing temperatures in the soil.
- Source of water.

If these conditions occur uniformly, heaving will be uniform; otherwise, differential heaving will occur causing surface irregularities, roughness, and ultimately cracking of the pavement surface.

A second effect of frost action is thaw weakening. The bearing capacity may be reduced substantially during thawing periods. However, frost heaving can be more severe during freeze-thaw periods, because water is more readily available to the freezing zone. In more southerly areas of the frost zone, several cycles of freeze and thaw may occur during a winter season and cause more damage than one longer period of freezing in more northerly areas. Spring thaws normally produce a loss of bearing capacity to well below summer and fall values, followed by a gradual recovery over a period of weeks or months.

Pavement design for frost action often determines the required overall thickness of flexible pavements and the need for additional select material beneath both rigid and flexible pavements. Three design approaches have been used for pavement in seasonal frost areas:

- The Complete Protection approach—requires non-frost susceptible materials for the entire depth of frost.
- Limited Subgrade Frost Penetration approach—permits some frost penetration into the subgrade, but not enough to allow unacceptable surface roughness to develop.

- Reduced Subgrade Strength approach—allows more frost penetration into the subgrade, but provides adequate strength during thaw weakened periods.

Many agencies provide protection from frost heave and thaw weakening by including a minimum thickness of pavement, base, and selected materials above the frost susceptible subgrade soils. Blending of the upper portion of the subgrade may be required by some agencies to provide more uniform frost heave during the winter and more uniform support during the spring. However, some pavement designs in seasonal frost areas use the reduced subgrade strength approach, while those in areas with low freezing indexes use the complete protection or limited subgrade frost protection approaches. For the most part, these approaches were developed from experience rather than by application of some rigorous theoretical computational method.

In this design procedure, a more rigorous method is used to determine the layer thickness necessary to reduce the effects of seasonal freezing and thawing to acceptable limits. The EICM is used to determine the maximum frost depth for the pavement system at a particular location. Various combinations of layer thickness and material types can be used to determine their impact on the maximum frost depth and total amount of base and select materials necessary to protect the frost susceptible soils from freezing.

Identification of Frost-Susceptible Soils

Frost-susceptible soils have been classified into four general groups. Table 2.1.3 provides a summary of the typical soils in each of these four groups, and figure 2.1.1 graphically displays the expected average rate of frost heave for the different soil groups.

Little to no frost action occurs in sands, gravels, crushed rock, and similar granular materials, when clean and free-draining, under normal freezing conditions. The large void space permits water to freeze in place without segregation into ice lenses. Conversely, silts are highly frost-susceptible. The condition of relatively small voids, high capillary potential/action, and relatively good permeability of these soils accounts for this characteristic.

Clays are cohesive and, although their potential capillary action is high, their capillary rate is low. Although frost heaving can occur in clay soils, it is not as severe as for silts, since the impervious nature of the clays makes passage of water slow. The supporting capacity of clays must be reduced greatly during thaws, although significant heave has not occurred. Thawing usually takes place from the top downward, leading to very high moisture contents in the upper strata.

A ground water level within 5 ft of the proposed subgrade elevation is an indication that sufficient water will exist for ice formation. Homogeneous clay subgrade soils also contain sufficient moisture for ice formation, even with depth to ground water in excess of 10 ft.

Table 2.1.3. Frost susceptibility classification of soils (*9*).

Frost Group	Degree of Frost Susceptibility	Type of Soil	Percentage Finer than 0.075 in by wt.	Typical Soil Classification
F1	Negligible to low	Gravelly soil	6-10	GM, GW-GM, GP-GM
F2	Low to medium	Gravelly soils	10-20	GM, GW-GM, GP-GM
		Sands	6-15	SM, SW-SM, SP-SM
F3	High	Gravelly Soils	Greater than 20	GM, GC
		Sands, except very fine silty sands	Greater than 15	SM, SC
		Clays PI<12	—	CL, CH
F4	Very high	All Silts	—	ML, MH
		Very Fine Silty Sands	Greater than 15	SM
		Clays PI>12	—	CL, CL-ML
		Varied clays and other fine grained, banded sediments	—	CL, ML, SM, CH

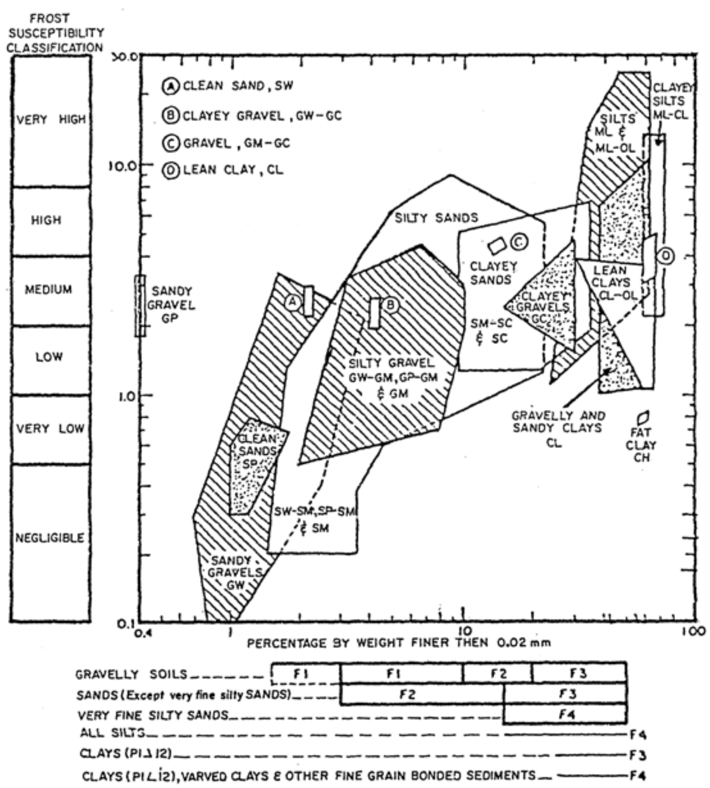

Figure 2.1.1. Average rate of heave versus percentage fines for natural soil gradations (*9*).

Treatment for Frost Action

When frost-susceptible soils are encountered, consideration should be given to the following alternatives for improving the foundation or supporting subgrade:

- Remove and replace the frost-susceptible soil (generally for groups F3 and F4) with select borrow that are non-frost susceptible to the depth of expected frost penetration.
- Place and compact select borrow materials that are non-frost-susceptible to a thickness or depth to prevent subgrade freezing for frost susceptible soil groups F2, F3, and F4.
- Remove isolated pockets of frost-susceptible soils to eliminate abrupt changes in subgrade conditions.
- Increase the pavement structural layer thickness to account for a strength reduction of the subgrade during the spring-thaw period for frost-susceptible groups F1, F2, and F3.
- Stabilize the frost-susceptible soil by eliminating the effects of soil fines by one of three processes: 1) mechanical removal or immobilization by means of physical-chemical means, such as cementitious bonding, 2) effectively reducing the quantity of soil moisture available for migration to the freezing plane, as by essentially blocking off all migratory passages, or 3) altering the freezing point of the soil moisture. Cementing agents such as portland cement, asphalt, lime, and lime-flyash effectively remove individual soil particles by bonding them together and also act to partially remove capillary passages thereby reducing the potential for moisture movement. Care must be taken when using lime and lime-flyash mixtures with clay materials in seasonal frost areas.

2.1.5 FOUNDATION IMPROVEMENT AND STRENGTHENING

Proper treatment of problem soil conditions and the preparation of the foundation are extremely important to ensure a long-lasting pavement structure that does not require excessive maintenance. In all cases, the provision for a uniform soil relative to textural classification, moisture and density in the upper portion of the subgrade cannot be over-emphasized. This uniformity can be achieved through soil sub-cutting or other techniques. Five techniques have been used to improve the strength and reduce the climatic variation of the foundation on pavement performance:

1. Stabilization of weak soils (highly plastic or compressible soils).
2. Thick granular layers.
3. Subsurface drainage systems.
4. Geosynthetics.
5. Soil encapsulation.

2.1.5.1 Stabilization

Objectives of Soil Stabilization

Soil that is highly susceptible to volume and strength changes can cause severe roughness and accelerate the deterioration of the pavement structure in the form of increased cracking and decreased ride quality when combined with truck traffic. Generally, the resilient modulus of some soils is highly dependent on moisture and stress state. In some cases, the subgrade soil can be treated with various materials to improve the strength and stiffness characteristics of the soil. Stabilization of soils is usually performed for two reasons:

1. As a construction platform to dry very wet soils and facilitate compaction of the upper layers—for this case, the stabilized soil is usually not considered as a structural layer in the pavement design process. This process is also sometimes referred to as soil modification.
2. To strengthen a weak soil and restrict the volume change potential of a highly plastic or compressible soil—for this case, the stabilized soil is usually given some structural value or credit in the pavement design process.

Lime, cement, and asphalt stabilization have been used for controlling the swelling and frost heave of soils and improving the strength characteristics of unsuitable soils. For stabilization or modification of cohesive soils, hydrated lime is most widely used. Lime modification is used in many areas of the U.S. to obtain a good construction platform in wet weather above highly plastic clays and other fine-grained soils. Lime is applicable in clayey soils (CH and CL type soils) and in granular soils containing clay binder (GC and SC), while portland cement is more commonly used in non-plastic soils. Lime reduces the Plasticity Index (PI) and renders a clay soil less sensitive to moisture changes. The use of lime should be considered whenever the PI of the soil is greater than 10. It is important to note that changing the physical properties of a soil through chemical stabilization can produce a soil that is susceptible to frost heave.

Some basic definitions of soil modification and stabilization using lime, cement, and asphalt are provided below. References *10*, *11*, *12*, and *13* and other related publications provide additional guidance on how stabilization is achieved using these three materials, respectively.

<u>Lime Treatment</u>

Lime treatment or modification consists of the application of 1 to 3 percent hydrated lime to aid drying of the soil and permit compaction. As such, it is useful in the construction of a "working platform" to expedite construction. Lime modification may also be considered to condition a soil for follow-on stabilization with cement or asphalt. Lime treatment of subgrade soils is intended to expedite construction, and no reduction in the required pavement thickness should be made.

<u>Lime Stabilization</u>

Lime or pozzolonic stabilization of soils improves the strength characteristics and changes the chemical composition of some soils. The strength of fine-grained soils can be improved significantly with lime stabilization, while the strength of coarse-grained soils is usually moderately improved. Lime has been found most effective with highly plastic clay soils containing montmorillonite, illite, and kaolinite.

Lime stabilization has been found to be an effective method to reduce the volume change potential of many soils. However, lime treatment of soils can convert the soil that shows negligible to moderate frost heave into a soil that is highly susceptible to frost heave, acquiring characteristics more typically associated with silts. It has been reported that this adverse effect has been caused by an insufficient curing period accompanied by an inadequate compaction effort. Adequate curing is also important if the strength characteristics of the soil are to be improved.

For successful lime stabilization of clay (or other highly plastic) soils, the lime content should be from 3 to 8 percent of the dry weight of the soil, and the cured mass should have an unconfined compressive strength increase of at least 50 psi after a 28-day curing period at 73 ^{o}F over the uncured material. The optimum lime content should be determined with the use of unconfined compressive strength and the Atterberg limits tests on laboratory lime-soil mixtures molded at varying percentages of lime. The lime-stabilized subgrade layer should be compacted to a minimum density of 95 percent, as defined by AASHTO T99. The minimum strength requirement for this material is a function of pavement type and the importance of the layer within the pavement structure. Additional guidance in this regard is provided in PART 2, Chapter 2.

When soils are treated properly with lime, it has been observed that the lime-soil mixture may be subject to durability problems caused by the cyclic freezing and thawing of the soil.

Lime-flyash stabilization is applicable to a broader range of soils because the cementing action of the material is less dependent on the fines contained within the soil. However, long-term durability studies of pavements with lime-flyash stabilization are rather limited.

Soils classified as CH, CL, MH, ML, SM, SC, and GC with a plasticity index greater than 10 and with 25 percent passing the No. 200 sieve potentially are suitable for stabilization with lime. Hydrated lime, in powder form or mixed with water as slurry, is used most often for stabilization. To determine the design lime content for a subgrade soil, the designer shall follow guidelines provided by the National Lime Association.

Cement Stabilization

Portland cement is used widely for stabilizing low-plasticity clays, sandy soils, and granular soils to improve the engineering properties of strength and stiffness. Increasing the cement content increases the quality of the mixture. At low cement contents, the product is generally termed cement-modified soil. A cement-modified soil has improved properties of reduced plasticity or expansive characteristics and reduced frost susceptibility. At higher cement contents, the end product is termed soil-cement. Higher cement contents will unavoidable induce higher incidence of shrinkage cracking caused by moisture/temperature changes.

For soils to be stabilized with cement, proper mixing requires that the soil have a PI of less than 20 percent and a minimum of 45 percent passing the No. 40 sieve. However, highly plastic clays that have been pretreated with lime or fly ash are sometimes suitable for subsequent treatment with Portland cement.

For cement stabilization of granular and/or nonplastic soils, the cement content should be 3 to 10 percent of the dry weight of the soil, and the cured material should have an unconfined compressive strength of at least 150 psi within 7 days (see PART 2, Chapter 2 for additional guidance on minimum strength requirements). The Portland cement should meet the minimum requirements of AASHTO M 85. The cement-stabilized subgrade should be compacted to a minimum density of 95 percent as defined by AASHTO M 134. Only fine-grained soils can be treated effectively with lime for marginal strength improvement.

Asphalt Stabilization

Generally, asphalt-stabilized soils are used for base and subbase construction. Use of asphalt as a stabilizing agent produces different effects, depending on the soil, and may be divided into three major groups: 1) sand-asphalt, which produces strength in cohesionless soils, such as clean sands, or acts as a binder or cementing agent, 2) soil-asphalt, which stabilizes the moisture content of cohesive fine-grained soils, and 3) sand-gravel asphalt, which provides cohesive strength and waterproofs pit-run gravelly soils with inherent frictional strength. The durability of asphalt-stabilized mixtures generally can be assessed by measurement of their water absorption characteristics.

Characteristics of Stabilized Soils

The improvement of subgrade or unbound aggregate by application of a stabilizing agent is intended to cause the improvements outlined above. These improvements arise from several important mechanisms that must be considered and understood by the pavement designer. Subgrade stabilizing agents may fill or partially fill the voids between the soil particles. This reduces the permeability of the soil by increasing the tortuosity of the pathways for water to migrate through the soil. Reduction of permeability may be relied upon to create a waterproof surface to protect underlying, water-sensitive soils from the intrusion of surface water. This mechanism must be accompanied by other aspects of the geometric design into a comprehensive system. The reduction of void spaces may also tend to change the volume change under shear from a contractive to a dilative condition. The stabilizing agent also acts by binding the particles of soil together, adding cohesive shear strength and increasing the difficulty with which particles can move into a denser packing under load. Particle binding serves to reduce swelling by resisting the tendency of particles to move apart. The particles may be bound together by the action of the stabilizing agent itself (as in the case of asphalt cement), or may be cemented by byproducts of chemical reactions between the soil and stabilizing agent (as in the case of lime or portland cement). Additional improvement can arise from other chemico-physical reactions that affect the soil fabric (typically by flocculation) or the soil chemistry (typically by cation exchange).

The zone that may be selected for improvement depends upon a number of factors. Among these are the depth of soft soil, anticipated traffic loads, the importance of the transportation network, and the drainage characteristics of the geometric design and the underlying soil. When only a thin zone is subject to improvement, removal and replacement will usually be the preferred alternative by most agencies unless a suitable replacement soil is not economically available. Note that in this context, the use of the qualitative term “thin” is intentional, as the thickness of the zone can be described as thick or thin based primarily on the economics of the earthwork requirements and the depth of influence for the vehicle loads.

Pavement Design Considerations for Stabilized Subgrades

The application of the stabilizing agent will usually increase the strength properties of the soil. This increase will generally appear in the pavement design process as an increase in the modulus of the improved soil, reducing the pavement structural layer thicknesses. The cost of the stabilization process, therefore, can be offset by savings in the pavement structural layers. However, it is important that the actual increase used in the design process be matched in the constructed product, making construction quality control and quality assurance programs very important. When pavement design is performed using only a single parameter to describe the subgrade condition, the thickness of the stabilized zone is a critical component in determining the increased modulus to use in design.

The thickness of the improved subgrade zone is both a design and a construction consideration. From the design standpoint, it would of course be advantageous to stabilize and improve the properties of a zone as thick as may be reasonably stabilized. From a constructability perspective, there are practical and economic implications related to the thickness of the stabilized zone. Stabilization requires that the agent be thoroughly distributed into the soil matrix, and that the soil matrix be well pulverized to prevent unimproved clumps from remaining isolated within the mass. The construction equipment to be used to provide mixing must be capable of achieving high levels of uniformity throughout the depth of desired improvement. If the zone to be improved is very thick, it may be necessary to process the stabilized soil in multiple lifts, which will usually require the stripping and stockpiling of upper lifts within the subgrade. Stabilization therefore rarely exceeds a few inches in depth in transportation applications, except for deep mixing applications that might be used in the vicinity of bridge foundations or abutments to provide improved foundation support.

2.1.5.2 Thick Granular Layers

Many agencies have found that a thick granular layer is an important feature in pavement design and performance. Thick granular layers are generally greater than 18 inches in thickness. Thick granular layers provide several benefits, including increased load-bearing capacity, frost protection, and improved drainage. While the composition of this layer takes many forms, the underlying strategy of each is to achieve desired pavement performance through improved foundation characteristics. The following sections describe the benefits of thick granular layers, typical characteristics, and considerations for the design and construction of granular embankments.

Objectives of Thick Granular Layers

Thick granular layers have been used in design for structural, drainage, and geometric reasons. Many times, a granular layer is used to provide uniformity and support as a construction platform. In areas with large quantities of readily accessible, good quality aggregates, a thick granular layer may be used as an alternative to soil stabilization. Whatever the reason, thick granular layers aim to improve the natural soil foundation. By doing this, many agencies are recognizing that the proper way to account for weak, poorly draining soils is through foundation improvement, as opposed to increasing the pavement layer thicknesses. The following is a list of objectives and benefits of thick granular layers:

- To increase the supporting capacity of weak, fine-grained subgrades.
- To provide a minimum bearing capacity for the design and construction of pavements.
- To provide uniform subgrade support over sections with highly variable soil conditions.
- To reduce the seasonal effects of moisture and temperature variations on subgrade support.
- To promote surface runoff through geometric design.
- To improve subsurface drainage and the removal of moisture from beneath the pavement layers.
- To increase the elevation of pavements in areas with high water tables.
- To provide frost protection in freezing climatic zones.
- To reduce subgrade rutting potential of flexible pavements.
- To reduce pumping and erosion beneath PCC pavements.
- To meet elevation requirements of geometric design.

Characteristics of Thick Granular Layers

Thick granular layers have been incorporated in pavement design in several ways. They can be referred to as fills or embankments, an improved or prepared subgrade, and select or preferred borrow. Occasionally, a thick granular layer is used as the pavement subbase. The two most important characteristics for all of these layers are the material properties and thickness. While geometric requirements (e.g., vertical profile) and improved surface runoff can be achieved by embankments constructed of any soil type, the most beneficial effects are produced through utilization of good quality, granular materials. Several methods are used to characterize the strength and stiffness of granular materials, including the California Bearing Ratio (CBR) and resilient modulus testing. In addition, several types of field plate load tests have been used to determine the composite reaction of the embankment and soil combination. In general, materials with CBR values of 20 percent or greater are used, corresponding to resilient moduli of approximately 17,500 psi. These are typically sand or granular materials, or coarse-grained materials with limited fines, corresponding to AASHTO A-1 and A-2 soils.

Aggregate gradation and particle shape are other important properties. Typically, embankment materials are dense-graded with a maximum top-size aggregate that varies depending on the height of the embankment. Many times, the lowest embankment layer may contain cobbles or aggregates of 4 to 8 inches in diameter. Granular layers placed close to the embankment surface have gradations, including maximum size aggregates, similar to subbase material specifications. Although dense-graded aggregate layers do not provide efficient drainage relative to open-graded materials, a marginal degree of subsurface seepage can be achieved by limiting the fines content to less than 10 percent. The type of granular material used is normally a function of material availability and cost. Pit-run gravels and crushed stone materials are the most common. The high shear strength of crushed stone is more desirable than rounded, gravelly materials; however, the use of crushed materials may not always be economically feasible.

The thicknesses of granular layers vary, depending upon their intended use. Granular layers 6 to 12 inches thick may be used to provide uniformity of support or act as a construction platform for paving of asphalt and concrete layers. To increase the composite subgrade design values (i.e., combination of granular layer over natural soil), it is usually necessary to place a minimum of 1.5 to 5 ft of embankment material, depending on the strength of the granular material relative to that of the underlying soil. Likewise, granular fills placed for frost protection may also range from 1.5 to 5 ft. In most cases, embankments greater than 6 ft thick have diminishing effects in terms of strength. For example, the required thickness of the HMA layers above thick granular layers (greater than 6 ft) does not decrease with increasing granular thickness. Granular embankments greater than 6 ft thick are usually constructed for purposes of geometric design.

Considerations for Pavement Structural Design

The use of a thick granular layer presents an interesting situation for design. The placement of a granular layer of substantial thickness over a comparatively weak underlying soil forms, essentially, a non-homogeneous subgrade, at least at the bottom of the granular layer. Pavement design requires a single subgrade design value, for example CBR, resilient modulus, or k-value. This is generally determined through laboratory or field tests, when the soil mass in the zone of influence of vehicle loads is of the same type, or exhibits similar properties. In the case of a non-homogeneous subgrade, the composite reaction of the embankment and soil combination can vary from that of the natural soil to that of the granular layer. Most commonly, the composite reaction is a value somewhere between the two extremes, dependent upon the relative difference in moduli between the soil and embankment, and the thicknesses of the granular layer. The actual composite subgrade response is not known until the embankment layer is placed in the field, and it may be different once the upper pavement layers are placed.

To account for non-homogenous subgrades in pavement structural design, it is recommended to characterize the individual material properties by traditional means, such as CBR and resilient modulus testing, and to compare these results to field tests performed over the constructed embankment layers, as well as the completed pavement section. Analytical models, such as elastic layer programs, can be used to make theoretical predictions of composite subgrade response, and these predictions can then be verified by field testing. Some agencies use in situ plate load tests to verify that a minimum composite subgrade modulus has been achieved. Deflection devices, including the Falling Weight Deflectometer (FWD), can be used for testing over the compacted embankment layer and over the constructed pavement surface.

It is advisable to use caution when selecting a design subgrade value for a non-homogenous subgrade. Experience has shown that a good-quality embankment layer must be of significant height, say 3.3 ft or more, before the composite subgrade reaction begins to resemble that of the granular layer. This means that, for granular layers up to 3.3 ft in height, the composite reaction can be much less than that of the embankment layer itself. If too high a subgrade design value is selected, the pavement will be under-designed. Granular layers less than 1.5 ft thick have minimal impact on the composite subgrade reaction, when loaded under the completed pavement section.

2.1.5.3 Subsurface Drainage

Subsurface drainage systems are used for three basic reasons:

- To lower the ground water level.
- To intercept the lateral flow of subsurface water beneath the pavement structure.
- To remove the water that infiltrates the pavement's surface.

Deep underdrains (greater than 3.3 ft deep) are usually installed to handle groundwater problems as indicated by the first two bullet items above. The design and placement of these underdrains should be handled as part of the geotechnical investigation of the site. Edgedrains placed in trenches under the shoulders at shallower depths are used to handle water infiltrating the pavement from above. The design and placement of these drainage systems is discussed in PART 3, Chapter 1.

2.1.5.4 Geosynthetics

Geosynthetics are a class of geomaterials that are used to improve soil conditions for a number of applications. They consist of manufactured polymeric materials used in contact with soil materials or pavements as an integral part of a man-made system (after ASTM D4439). The most common applications in general use are in pavement systems for both paved and unpaved roadways, for reinforcing embankments and foundation soils, for creating barriers to water flow in liners and cutoffs, and for improving drainage. The generic term geosynthetic is often used to cover a wide range of different materials, including geotextiles, geogrids, and geomembranes. Combinations of these materials in layered systems are usually called geocomposites.

A geotextile, as defined by ASTM D4439, is "a permeable geosynthetic comprised solely of textiles." These materials are also known as fabrics. Fabrics are usually created from polymers, most commonly polypropylene, but also potentially including polyester, polyethylene, or nylon *(14)*. Geotextiles are usually classified by their manufacturing process as either woven or nonwoven. Both kinds of geosynthetics use a polymer fiber as raw material. Depending on the application, the fibers may be used singly or spun into yarns by wrapping several fibers together, or created by a slit film process. Woven geosynthetics are manufactured by weaving fibers or yarns together in the same way as any form of textile, although generally only fairly simply weaving patterns are used. Nonwoven geosynthetics are made by placing fibers in a bed, either full length or in short sections. The fibers are then bonded together, either by raising the temperature, applying an adhesive chemical, or mechanically (usually by punching the bed of fabric with barbed needles, in essence tangling them into a tight mat).

Geogrids, as their name suggests, consist of a regular grid of plastic with large openings (called apertures) between the tensile elements. The function of the apertures is to allow the surrounding soil materials to interlock across the plane of the geogrid; hence, the selection of the size of the apertures is partially dependent on the gradation of the material into which it will be placed. The geogrid is manufactured using high-density polymers of higher stiffnesses than are common for geotextiles. These polymers are then punched in a regular pattern and drawn to create a wide grid. Geogrids are commonly described as either biaxial or uniaxial depending on whether the sheet is drawn in one or two directions. Alternatively, a weaving process may be used in which the crossing fibers are left wide apart and the junctions between them are reinforced.

Geomembranes are used to retard or prevent fluid and as such consist of continuous sheets of low permeability materials. These materials are made by extruding or calendaring the polymer into a flat sheet, which may have a roughened surface created to aid in the performance of the membrane by increasing friction with the adjacent soil layer.

There are also a number of other kinds of geosynthetic materials that may be made by slight variations of these general types. For example, geonets are similar in appearance to geogrids but are manufactured slightly differently so that the individual elements of the geonet are at acute angles to each other. These materials are usually used in drainage applications.

Geocomposite materials are often created by combining two or more of the specific types of products described previously to take advantage of multiple benefits. Further, geocomposites may be formed by combining geosynthetics with more traditional geomaterials, the most common example being the geosynthetic clay liner. A geosynthetic clay liner consists of a layer of bentonite sandwiched together with geomembrane or geotextile materials to create a very low permeability barrier.

There are six widely recognized functions for geosynthetic applications *(15)*. These are shown across the top of table 2.1.4. The typical classes of geosynthetic used for each function are also shown. Although the table indicates only primary functions, most geosynthetic applications call for the material to satisfy at least one secondary function as well as (for example, a separation layer under a pavement may also be required to reinforce the subgrade and influence drainage under the pavement).

Koerner *(14)* provides a summary of the most commonly used geosynthetic functions for transportation applications, which is presented in table 2.1.5. Comparison of table 2.1.4 and table 2.1.5 reveals that the geotextile and geogrid materials are the most commonly used in transportation, although certainly others are sometimes used. This generality is more accurate when only the pavement itself (not including the adjoining fill or cut slopes, retaining walls, abutments, or drainage facilities) is considered. The most common usage for geosynthetics in the United States has been for unpaved roads historically, but use in paved, permanent roads is increasing.

Each of these functional classes, while potentially related by the specific application being proposed, refers to an individual mechanism for the improvement of the soil subgrade. The separation function describes the maintenance of materials of different gradations as separate and distinct materials. In the specific case of the pavement application, separation relates to the maintenance of unbound granular base course materials as distinct from the subgrade *(14, 16)*. These materials may tend to become mixed in service due to pumping of the subgrade into the base or due to localized bearing capacity failures leading to migration of aggregate particles into the subgrade (*17*). This potential behavior has been confirmed in the field, as well as the ability

Table 2.1.4. Division of geosynthetic materials by primary function (after *15*).

Geosynthetic	Filtration	Drainage	Separation	Reinforcement	Fluid Barrier	Protection
Geotextile	x	x	x	x		x
Geogrid			x	x		
Geomembrane					x	
Geonet		x				
Geocomposites:						
Geosynthetic Clay liner					x	
Thin film Geotextile Composite					x	
Field coated Geotextile					x	

Table 2.1.5. Transportation uses of geosynthetic materials (after *14*).

General Category	Specific Use
Separation of Dissimilar Materials	Between subgrade and aggregate base in paved and unpaved roads and airfields Between subgrade and ballast for railroads Between old and new asphalt layers
Reinforcement of weak materials	Over soft soils for unpaved roads, paved roads, airfield, railroads, construction platforms
Filtration	Beneath aggregate base for paved and unpaved roads and airfields or railroad ballast
Drainage	Drainage interceptor for horizontal flow Drain beneath other geosynthetic systems

of geosynthetic materials to resist it *(18, 19)*. Once the unbound base is mixed with the subgrade, its strength and drainage properties may be detrimentally affected.

The reinforcement function is very similar to the reinforcement process in reinforced concrete elements. The geosynthetic is introduced to provide elements with tensile resistance into the unbound material, which on its own would exhibit very low tensile resistance. The specific improvements imparted to pavement designs include the potential for improved lateral restraint of the base and subgrade, modifications of bearing capacity failure surfaces, and tensile load transfer under the wheel load. The lateral restraint arises as the base material tends to move outward under load beneath the wheel. The geosynthetic tends to be pulled along as a result of friction or interlock with the aggregate particles, and resists that tendency through its own tensile strength. The particles are therefore held in place as well. Bearing capacity surfaces may be forced to remain above the geosynthetic, in the stronger base course. Finally, the tendency of the base to bend under the wheel loads introduces tensile stress at the base/subgrade interface, which may be taken by the geosynthetic. Careful consideration must be given to the mobilization behavior of the geosynthetic, which may require fairly large strains to provide the desired resistance *(15)*.

The filtration function is similar to the separation function, but in this case the reason for mixing or migration of particles is the seepage forces induced by water flowing through the unbound material. The function of the filter is to provide a means to allow water to flow through unbound material without excessive loss of soil due to seepage forces, and without clogging *(14)*. Zonal filters may offer the same protection, but may be less convenient or practical to install. The drainage function is related to the filtration function, in that once again the desired behavior is the movement of water out of or through the unbound material with sufficient maintenance of the fine particles in place. The difference arises in the focus and intent; filtration applications tend to be predicated on the maintenance of the soil, while drainage applications tend to attach more importance to the quantity of flow to be maintained or the desired reduction in pore water pressure. Further, the drainage function may be carried out by designing for drainage along the plane of the geotextile itself, rather than through surrounding unbound material.

The specific function to be provided by the geosynthetic in transportation applications is a function of the soil conditions. Holtz et al. *(15)* indicate that the following functions most commonly arise as a function of the soil strength (table 2.1.6).

Table 2.1.6. Function of the geosynthetic vs. subgrade properties (after *15*).

S_u (kPa)	**CBR**	**Function**
60-90	2-3	Filtration, some separation
30-60	1-2	Filtration, separation, some reinforcement
<30	Below 1	Filtration, separation, reinforcement

The range of functions potentially served by the geosynthetic thus increases as the subgrade strength decreases. In all cases reported in table 2.1.6, the soil conditions are rather poor. In fact, Holtz et al. *(15)* indicate that geosynthetics are most appropriate under the conditions outlined in table 2.1.7.

Table 2.1.7. Appropriate conditions for geosynthetic use (after 15).

Condition	**Related Measures**
Poor soils	USCS of SC, CL, CH, ML, MH, OL, OH, PT or AASHTO of A-5, A-6, A-7, A-7-6
Low strength	c_u<13 psi or CBR<3 or M_R < 4500 psi
High water table	Within zone of influence of surface loads
High sensitivity	High undisturbed strength compared to remolded strength

<u>Design Considerations for Geosynthetics</u>

Koerner *(14)* describes three potential design approaches—design by cost, design by specification, and design by function—to design geosynthetics for engineering application. The latter two approaches that relate to rational engineering design are described below in the order of increasing rigor and sophistication:

Design by Specification

In this case, the functions required for the geosynthetic are selected, and specifications are written to satisfy this function according to specific rules outlined in a guide specification or policy. The AASHTO M 288 standard is used in this way, as are the design approaches of many public agencies. In the AASHTO standard, AASHTO design classes are selected based on the properties of the soil to be improved and the specific primary function to be addressed by the geosynthetic, and then a geosynthetic is selected which exceeds the requirements of the design class. The AASHTO standard is predicated on the assumption that the survival of the construction process is the key issue in the design. In fact, pavement design methods using this specification *(15)* assume that the primary function of the geosynthetic is to reduce additional requirements for construction platform installation, and the potential benefits of the geosynthetic are not incorporated in the structural design of the pavement itself. This assumption is based on the expectation that unacceptably large strains which would be required to mobilize geosynthetic resistance if influence of the long-term subgrade performance were expected.

A list of potentially important properties and existing measurement methods is provided in table 2.1.8. Manufacturers will commonly report some of these values in their product literature. Some care must be used in comparing manufacturer's information with the minimum requirements presented in the AASHTO standard, as the manufacturers often report average values for a roll or lot rather than minimum values. Many manufacturers provide the AASHTO survival class with their product literature.

The design of the geosynthetic is completed using AASHTO M 288 in the following steps (taken from *15*). A key feature of this method is the assumption that the structural pavement design is not modified at all in the procedure. The pavement design proceeds exactly according to standard procedures as if the geosynthetic was not present. The geosynthetic instead replaces additional unbound material that might be placed to support construction operations, and replaces no part of the pavement section itself.

1. Identify properties of the subgrade, including CBR, location of groundwater table, AASHTO and/or Unified classification, and sensitivity.
2. Compare these properties to those in table 2.1.7 or with local policies. Determine if a geosynthetic will be required.
3. Design the pavement without consideration of a geosynthetic, using normal pavement structural design procedures.
4. Determine the need for additional imported aggregate to ameliorate mixing at the base/subgrade interface. If such aggregate is required, determine its thickness, t_1..
5. Determine additional aggregate needed for establishment of a construction platform. The FHWA procedure requires the use of USFS (United States Forest Service) curves for aggregate thickness vs. the expected single tire pressure and the subgrade bearing capacity. Alternatively, local policies or charts may be used. This thickness is t_2.
6. Select the greater of t_1 and t_2.
7. Check filtration criteria for the geosynthetic to be used. The important measures include the apparent opening size (AOS), the permeability and permittivity of the geotextile, and the 95% opening size, defined as the diameter of glass beads for which 95% will be retained on the geosynthetic. These values will be compared to a minimum standard or to the soil properties.

Table 2.1.8. Geosynthetic properties and relevant test methods (*14*).

Property Assessed		Test Method(s)	Typical Values
Abrasion Resistance		ASTM D1175, D4886 ISO 13427	—
Apparent Opening Size		ASTM D4751 ISO 12956	—
Basis Weight (Mass per Unit Area)		ASTM D5261 ISO 9864	0.2765-1.3825 lb/yd^2 (150 – 750g/m^2)
Clogging		ASTM D5084, D5101	—
Creep Resistance		ASTM D5262 ISO 13431	—
Flexure Stiffness		ASTM D1388	0.868 – 21.7 lb-in (1000 – 25000 mg·cm)
Frictional Properties (Mohr Coulomb)		ASTM D5321	—
Transmissivity (In plane permeability)		ASTM D4716 ISO 12958	3.229 x 10^{-8} – 2.153 x 10^{-5} ft^2/sec (3.0X10^{-9} – 2.0X10^{-6}m^2/s)
Permittivity (Cross plane perm.)	Unloaded	ASTM D4491 ISO 11058	—
	Loaded	ASTM D5493	—
Puncture Resistance		ASTM D4833 ISO 12236	—
Seam Strength		ASTM D4884 ISO 10321	—
Soil Retention		ASTM D5141	—
Specific Gravity		ASTM D792 ASTM D1505	PVC: G_s=1.69 Polyester: G_s=1.38-1.22 Nylon: G_s=1.38-1.22 Polyethylene: G_s=0.90-0.96 Polypropylene: G_s=0.91
Tear Strength	Elmendorf	ASTM D1424	—
	Tongue	ASTM D751	—
	Trapezoidal	ASTM D4533	—
Tensile Strength	Grab Test	ASTM D 4632	—
	Wide Width Test	ASTM D4595 ISO 10319	—
Thickness		ASTM D5199 ISO 9863	0.0098 – 0.2953 inch (0.25 – 7.5 mm)

8. Determine survival criteria. The design is based on the assumption that the geosynthetic cannot function unless it survives the construction process. The AASHTO M 288 standard categorizes the requirements for the geosynthetic based on the survival class. The requirements for the standard include the strength (grab, seam, tear, puncture, and burst), permittivity, apparent opening size, and resistance to UV degradation based on the survival class. The survival class is determined from table 2.1.9.
9. Select a geosynthetic that meets or exceeds the requirements of the M 288 standard for the appropriate Survivability Rating.

Table 2.1.9. Geosynthetic construction survivability rating (*15*).

Soil CBR[1]	**< 1**		**1-2**		**3 or more**	
Equipment Ground Contact Pressure, psi (kPa)	> 50 psi (350kPa)	< 50 psi (350kPa)	> 50 psi (350kPa)	< 50 psi (350kPa)	> 50 psi (350kPa)	< 50 psi (350kPa)
Compacted Cover Thickness[2], inch (mm)						
4 inch (100 mm)[3,4]	NR[5]	NR	1[5]	1	2[5]	2
6 inch (150 mm)	NR	NR	1	1	2	2
12 inch (300 mm)	NR	1	2	2	2	2
18 inch (450 mm)	1	2	2	2	2	2

Notes:
1 Assume saturated unless construction scheduling can be controlled.
2 Maximum aggregate size not to exceed half the compacted cover thickness.
3 For low-volume, unpaved roads (ADT <200).
4 The 100 mm minimum is limited to existing road bases and is not intended for use in new construction.
5 NR = Not recommended; 1= High survivability class (more stringent); 2 = Moderate survivability class (less stringent) per AASHTO M288.

Field installation procedures introduce a number of special concerns; the AASHTO M 288-99 standard includes a guide specification for construction. Holtz et al. *(15)* recommended that this specification be modified to suit local conditions and contractors. Concerns and criteria for field installation include, for example, the seam lap and sewing requirements and construction sequencing and quality control.

Design by Function

The design by function approach attempts to follow a more mechanistic design approach. The approach recommended by Koerner *(14)* is an allowable stress design (ASD) formulation. In this formulation, required strengths are developed for each potential failure mechanism, and the geosynthetic is selected so that it will meet or exceed the requirement. The allowable stress is determined using a relatively common approach, but with factors to consider a number of individual behaviors, including installation damage, creep, chemical degradation, and biological degradation. This design approach could lead to reductions in the pavement structural section, and in that case the decision to use geosynthetics is one part of a life cycle cost analysis for the pavement structure. The use of a geosynthetic generally decreases the need for other materials, and thereby offsets its own cost. The specific geosynthetic to be used is chosen to meet or exceed the needs of its specific application by calculation based on the desired performance of the geosynthetic.

For example, in a drainage application, the permeability of the geosynthetic might be the appropriate functional measurement, while in the reinforcement application the tensile strength of the geosynthetic might be more appropriate. The required strength is calculated for the particular application. A factor of safety can then be applied, so that the limiting value of the required functional measurement can then be assigned. A geosynthetic would then be selected which provides functional requirements for the specific application based on this allowable limit.

This approach requires that one be able to estimate the value of the primary and/or secondary functional measurements for the specific application being considered. In order to do this, one must be able to analyze the performance of the geosynthetic in the field for its intended application. At present, there does not seem to be agency confidence in the prediction of field performance. Koerner *(14)* provided a number of methods for estimating the factored or required values for properties of the geosynthetic, but does not provide insight into the analysis problem. At present, the most commonly used agency approach seems to be the design by specification.

2.1.5.5 Soil Encapsulation

Soil encapsulation is a foundation improvement technique that has been used to protect moisture sensitive soils from large variations is moisture content. However, this technique is rarely used to improve the foundations of higher volume roadways. It is more commonly used as a foundation or subbase layer for low-volume roadways, where the import of higher quality paving materials is restricted from a cost standpoint. The concept of soil encapsulation is to keep the fine-grained soils at or slightly below optimum moisture content, where the strength of these soils can support heavier trucks and traffic.

Fine-grained soils can provide adequate bearing strengths for use as structural layers in pavements and embankments, as long as the moisture content remains below the optimum moisture content. However, increases in moisture content above the optimum value can cause a significant reduction in the resilient modulus and strength of fine-grained materials and soils. Increased moisture content in fine-grained soils below pavements occurs over time, especially in areas subject to frost penetration and freeze-thaw cycles. Thus, fine-grained soils cannot be used as a base or subbase layer unless the soils are protected from any increase in moisture.

The soil encapsulation concept, sometimes referred to as membrane encapsulated soil layer (MESL), is a method for maintaining the moisture content of the soil at the desired level by encapsulating the soil in waterproof membranes. The waterproof membranes prevent water from infiltrating the moisture sensitive material. The resilient modulus measured at or below optimum conditions remains relatively constant over the design life of the pavement.

The prepared subgrade is normally sprayed with an asphalt emulsion before the bottom membrane of polyethylene is placed. This asphalt emulsion provides added waterproofing protection in the event the membrane is punctured during construction operations, and acts as a adhesive for the membrane to be placed in windy conditions. The first layer of soil is placed in sufficient thickness such that the construction equipment will not displace the underlying material. The completed soil embankment is also sprayed with an asphalt emulsion before placement of the top membrane. The top of the membrane is sprayed with the same asphalt emulsion and covered with a thin layer of clean sand to blot the asphalt and to provide added protection against puncture by the construction equipment used to place the upper paving layers.

The reliability of this method to maintain the resilient modulus and strength of the foundation soil over long periods of time is unknown. More importantly, roadway maintenance and the installation of utilities limit the use of this technique. Thus, this improvement technique is not suggested unless there is no other option available.

If this technique is used, the pavement designer should be cautioned regarding the use of the EICM to predict changes in moisture over time. Special design computations will be needed to characterize the change in moisture content of the MESL over time. The resilient modulus used in design for the MESL should be held constant over the design life of the pavement. The designer should also remember that any utilities placed after pavement construction could make that assumption invalid.

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