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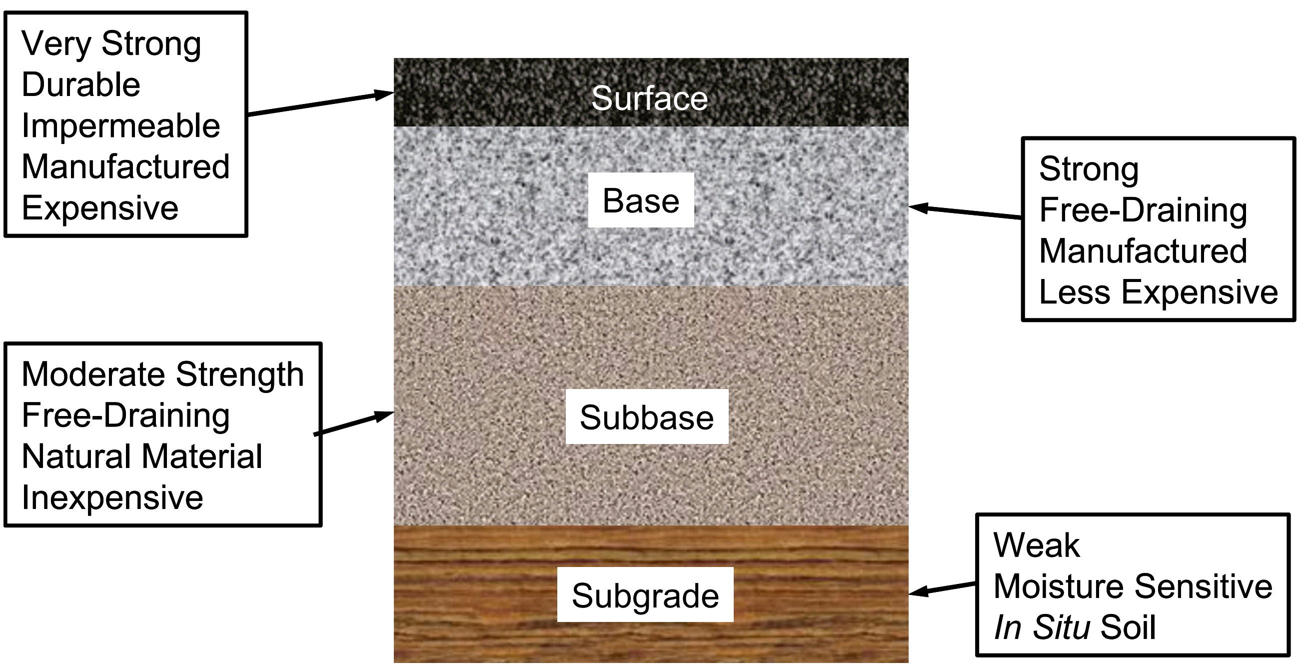
Geotechnical Aspects of Pavements

Reference Manual / Participant Workbook



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16. ABSTRACT This is the Reference Manual and Participants Workbook for the FHWA NHI's Course No. 132040 – **Geotechnical Aspects of Pavements**. The manual covers the latest methods and procedures to address the geotechnical issues in pavement design, construction and performance for new construction, reconstruction, and rehabilitation projects. The manual includes details on geotechnical exploration and characterization of in place and constructed subgrades as well as unbound base/subbase materials. The influence and sensitivity of geotechnical inputs are reviewed with respect to the requirements in past and current AASHTO design guidelines and the mechanistic-empirical design approach developed under NCHRP 1-37A, including the three levels of design input quality. Design details for drainage features and base/subbase material requirements are covered along with the evaluation and selection of appropriate remediation measures for unsuitable subgrades. Geotechnical aspects in relation to construction, construction specifications, monitoring, and performance measurements are discussed.			
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CHAPTER 1.0 INTRODUCTION

1.1 INTRODUCTION

This is the reference manual for FHWA NHI's Course #132040 – Geotechnical Aspects of Pavements. Many groups within an agency are involved with different aspects of definition, design use and construction verification of pavement geomaterials. These groups include

- pavement design engineers,
- geotechnical engineers,
- specification writers, and
- construction engineers.

The three-day training course was developed as a format for these various personnel to meet and together develop a better understanding of the geotechnical aspects of pavements. The overall goal is for this group of personnel to work together to enhance current procedures to build and maintain more cost-efficient pavement structures. The geotechnical aspects are particularly important today, as longer pavement performance (analysis) periods are being used in design. The maintenance and rehabilitation activities used in the pavement management process to achieve the design performance period require a competent support from the underlying geomaterials.

Thus, this manual has been prepared to assist pavement design engineers, geotechnical engineers, specification writers, and construction engineers in understanding the geotechnical aspects of pavement design, and as a common tool for future reference. This manual covers the latest methods and procedures to address the geotechnical issues in pavement design, construction, and performance, including

- a review of the geotechnical parameters of interest in pavement design, construction, and performance of different types of pavements.
- the influence of climate, moisture, and drainage on pavement performance.
- the impact of unsuitable subgrades on pavement performance.
- the determination of the geotechnical inputs needed for the design, construction, and performance of pavements.
- evaluation and selection of appropriate remediation measures for unsuitable pavement subgrades.
- the geotechnical aspects of pavement construction specifications and inspection requirements.

- a review of typical subgrade problems during construction and the recommended solutions.

This manual covers the latest methods and procedures for

- new construction,
- reconstruction, and
- rehabilitation projects (*e.g.,* widenings, overlays, and treatments).

The manual covers designing and constructing pavement subgrades and unbound materials for paved and unpaved roads with emphasis on

- the current AASHTO, 1993 design guidelines, and
- the mechanistic-empirical design approach, including the three levels of design inputs being developed under the NCHRP 1-37A.

Previous AASHTO design methods are also reviewed in relation to the sensitivity of geotechnical inputs. The design details section also provides a review of the overall geotechnical and drainage aspects of bases, subbases, and subgrades.

The manual is divided into modules, with each chapter representing a module for specific geotechnical aspects of pavement system design, including

- introduction to geotechnical aspects of pavements,
- basic concepts and special conditions,
- subsurface exploration,
- determination of key geotechnical inputs,
- actual design (and sensitivity of design/performance to geotechnical inputs),
- design details and special problems, and
- construction and QC/QA issues.

Each module contains a short background section, which provides an overview of the specific design or construction element. Following the background section, the design element and, subsequently, the construction process, is presented. Examples used to demonstrate the method and case histories of both successes and failures are presented to support the use of the design and construction concepts.

1.2 A HISTORICAL PERSPECTIVE OF PAVEMENT DESIGN

Pavements with asphalt or concrete surface layers have been used in the United States since the late 1880s. The historical development of asphalt and concrete pavement design is discussed in more detail in Chapter 3. Although pavement materials and construction methods have advanced significantly over the past century, until the last decade, pavement design has been largely empirical, based on regional experience. Even the empirically based designs of the 1980s and 1990s, as expressed in the AASHTO 1986, 1993, and 1998, guidelines have been, for most cases, modified by state agencies, based on regional experience. For example, several agencies still use their own modifications of the 1972 AASHTO design guidelines. Currently, approximately one-half of the state agencies are using the 1993 guide, albeit usually with some modification. This close reliance on empirical evidence makes it difficult to adopt new design concepts. Empirical designs are significantly challenged by constantly changing design considerations (*e.g.,* traffic loads and number of applications, types of pavements, road base aggregate supply, etc.). An additional change is the type of pavement construction, which has shifted over the past several decades from new construction to rehabilitation. Recycled materials now often replace new construction materials. During the past ten years, a major thrust has been to develop a more scientific explanation of the interaction between the pavement structure, the materials, the environment and the wheel loading. The need for a more sophisticated design method becomes even more apparent when considering the number of variables, with more than twenty just for the geotechnical features (*e.g.,* unit weight, moisture content, gradation, strength, stiffness, and hydraulic conductivity, as described in Chapter 6) that influence the design in a modern pavement system.

Fortunately, the tools available for design have also significantly advanced over the past several decades. Specifically, computerized numerical modeling techniques (*i.e.,* mechanistic models) are now available that can accommodate the analysis of these complex interaction issues and, at the same time, allow the models to be modified based on empirical evidence. The development of mechanistic-empirical models is described in Chapter 3 and their use in design of unbound pavement materials is detailed in Chapter 6. The new national Pavement Design Guide development under NCHRP Project 1-37A (NCHRP 1-37A Design Guide) provides the basis for the information in these chapters. Several agencies have already adopted mechanistic-empirical analysis, at least as a secondary method for flexible pavement design (Newcomb and Birgisson, 1999). The newer, more sophisticated design models for flexible and rigid pavements rely heavily on accurate characterization of the pavement materials and supporting conditions for design input. As a result, there is a greater reliance on geotechnical inputs in the design models. Geotechnical exploration and testing programs are essential components in the reliability of pavement design and have also advanced significantly in the past several decades.

Better methods for subsurface exploration and evaluation have been developed over time. Standard penetration tests (SPT), where a specified weight is dropped from a specific height on a thick-walled tube sampler to obtain an index strength value and disturbed sample of the subgrade, was developed in the 1920s. A typical practice is to locate the sampling intervals at a standard spacing along the roadway alignment. However, subgrade conditions can vary considerably both longitudinally and transversely to the alignment. This approach evaluates and samples less than a billionth of the soil along the roadway alignment, often missing critical subsurface features and/or variations. In addition, the SPT value itself has a coefficient of variation of up to 100% (Orchant et al., 1988). Based on these considerations, one must question the use of this approach as the sole method for subsurface evaluation. As discussed in Chapter 4, geophysical methods (*e.g.,* ground penetrating radar (GPR) and falling weight deflectometer (FWD)) and rapid in-situ testing (*e.g.,* cone penetration test - CPT) now allow for economical spatial characterization of subsurface conditions such that soil borings for sampling can be optimally located. The use of FWD to directly evaluate the dynamic response of existing pavement materials and support conditions for reconstruction and/or rehabilitation – now a standard of practice by a number of agencies – is also reviewed.

Empirical design methods of the past often relied on index tests such as CBR or R-value for characterizing the supporting aggregate and subgrade materials. Just as the design methods were modified for local conditions, agencies have modified the test methods to the extent that there are currently over ten index methods used across the United States to characterize these materials. Tests include the IBR (Illinois), the LBR (Florida), the Washington R-value, the California R-value, the Minnesota R-value, and the Texas triaxial, to name a few. Rough correlations between many of these methods are reviewed in Chapter 5. Considering that both the design and the input values may rely upon local knowledge, it is not surprising that comparison of test sections constructed by different agencies is often difficult. A method that allows for direct modeling of the dynamic response of subsurface soils and base course aggregate materials is the resilient modulus test. Advancements in the resilient modulus equipment and test procedures are reviewed in Chapter 5.

Throughout the history of pavement design, special problems have been encountered in relation to pavement support. These include expansive soils, frost susceptible materials, caliche, karst topography, pumping soils and highly fluctuating groundwater conditions. The solution to these problems is often to remove and replace these materials, often at great expense to the project. Today there are a number of alternate techniques available to resolve these issues, as discussed in Chapter 7.

Finally, verification of construction over the past century has used an array of methods for spot-check detection techniques, including evaluation of density using sand cone, balloon and, more recently, nuclear gauge techniques and/or load tests using plate load, field CBR, and drop cone methods. As with many of the older laboratory testing techniques, these field quality control (QC) methods are usually index tests and do not directly measure the dynamic response of the inplace materials. Many of these methods are reviewed in Chapter 8, along with some newer rapid assessment techniques (*e.g.,* the Geogage) that provide a direct assessment of the dynamic support conditions. While most of the index methods, as well as the dynamic response methods, usually produce reliable results, the small sample area evaluated does not allow an assessment of the overall project uniformity. Also, construction methods have developed to the extent that it is often impossible to keep up with the placement of materials. One of the oldest methods that does provide good area coverage and which is still widely used in current practice is proof rolling. However, this method is often subjective. Improvements in evaluation of proof rolling through the use of modern survey techniques now allow for a more quantitative, less subjective evaluation of the subgrade conditions, and are discussed in Chapter 8. The other disadvantage of all these methods is that the work is checked after the fact, often requiring rework and slowing the project. A new, more rapid technique that allows for real time evaluation of each pavement layer as it is constructed is now available, and is reviewed in Chapter 8. These "intelligent compaction" methods can also be directly tied into pavement guarantee and warranty programs, also discussed in Chapter 8.

1.3 THE PAVEMENT SYSTEM AND TYPICAL PAVEMENT TYPES

The purpose of the pavement system is to provide a smooth surface over which vehicles may safely pass under all climatic conditions for the specific performance period of the pavement. In order to perform this function, a variety of pavement systems have been developed, the components of which are basically the same.

1.3.1 Components of a Pavement System

The pavement structure is a combination of subbase, base course, and surface course placed on a subgrade to support the traffic load and distribute it to the roadbed. Figure 1-1 presents a cross section of a basic modern pavement system, showing the primary components.

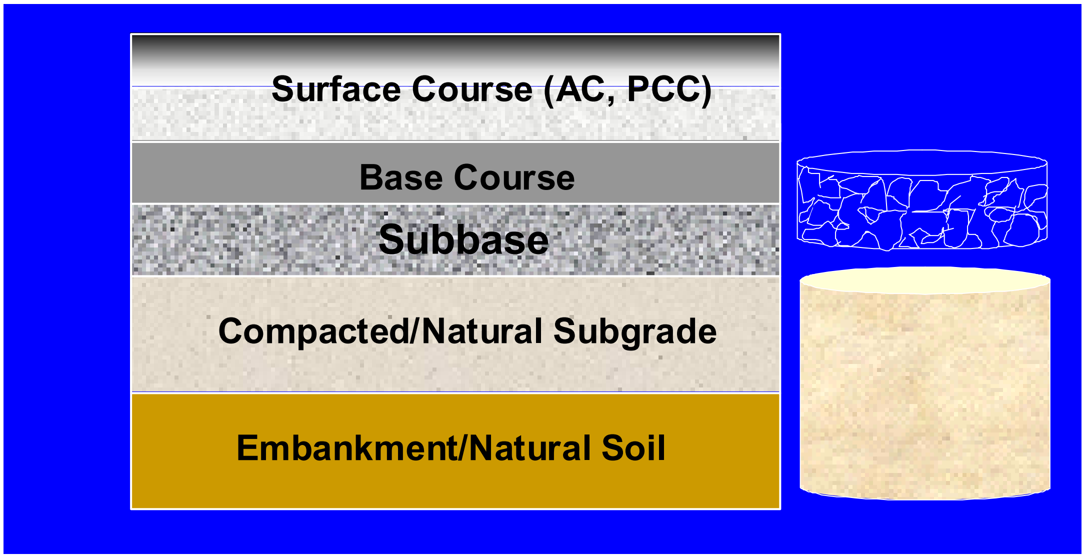


Figure 1-1. Basic components of a typical pavement system.

The **subgrade** is the top surface of a roadbed upon which the pavement structure and shoulders are constructed. The purpose of the subgrade is to provide a platform for construction of the pavement and to support the pavement without undue deflection that would impact the pavement's performance. For pavements constructed on-grade or in cuts, the subgrade is the natural in-situ soil at the site. The upper layer of this natural soil may be compacted or stabilized to increase its strength, stiffness, and/or stability.

For pavements constructed on embankment fills, the subgrade is a compacted borrow material. Other geotechnical aspects of the subgrade of interest in pavement design include the depth to rock and the depth to the groundwater table, especially if either of these is close to the surface. The actual thickness of the subgrade is somewhat nebulous, and the depth of consideration will depend on the design method, as discussed in Chapter 3.

The **subbase** is a layer or layers of specified or selected materials of designed thickness placed on a subgrade to support a base course. The subbase layer is usually of somewhat lower quality than the base layer. In some cases, the subbase may be treated with Portland cement, asphalt, lime, flyash, or combinations of these admixtures to increase its strength and stiffness. A subbase layer is not always included, especially with rigid pavements. A subbase layer is typically included when the subgrade soils are of very poor quality and/or suitable material for the base layer is not available locally, and is, therefore, expensive. Inclusion of a subbase layer is primarily an economic issue, and alternative pavement sections with and without a subbase layer should be evaluated during the design process.

In addition to contributing to the structural capacity of flexible pavement systems, subbase layers have additional secondary functions:

- Preventing the intrusion of fine-grained subgrade soils into the base layer. Gradation characteristics of the subbase relative to those of the subgrade and base materials are critical here.
- Minimizing the damaging effects of frost action. A subbase layer provides insulation to frost-susceptible subgrades and, in some instances, can be used to increase the height of the pavement surface above the groundwater table.
- Providing drainage for free water that may enter the pavement system. The subbase material must be free draining for this application, and suitable features must be included in the pavement design for collecting and removing any accumulated water from the subbase.
- Providing a working platform for construction operations in cases where the subgrade soil is very weak and cannot provide the necessary support.

The **base** is a layer or layers of specified or select material of designed thickness placed on a subbase or subgrade (if a subbase is not used) to provide a uniform and stable support for binder and surface courses. The base layer typically provides a significant portion of the structural capacity in a flexible pavement system and improves the foundation stiffness for rigid pavements, as defined later in this section. The base layer also serves the same secondary functions as the subbase layer, including a gradation requirement that prevents subgrade migration into the base layer in the absence of a subbase layer. It usually consists of high quality aggregates, such as crushed stone, crushed slag, gravel and sand, or combinations of these materials. The specifications for base materials are usually more stringent than those for the lower-quality subbase materials.

High quality aggregates are typically compacted unbound – *i.e.,* without any stabilizing treatments – to form the base layer. Materials unsuitable for unbound base courses can provide satisfactory performance when treated with stabilizing admixtures, such as Portland cement, asphalt, lime, flyash, or a combination of these treatments, to increase their strength and stiffness. These stabilizing admixtures are particularly attractive when suitable untreated materials are in short supply local to the project site. Base layer stabilization may also reduce the total thickness of the pavement structure, resulting in a more economical overall design.

Finally, the **surface course** is one or more layers of a pavement structure designed to accommodate the traffic load, the top layer of which resists skidding, traffic abrasion, and the disintegrating effects of climate. The surface layer may consist of asphalt (also called bituminous) concrete, resulting in "flexible" pavement, or Portland cement concrete (PCC), resulting in "rigid" pavement. The top layer of flexible pavements is sometimes called the "wearing" course. The surface course is usually constructed on top of a base layer of unbound coarse aggregate, but often is placed directly on the prepared subgrade for low-volume roads. In addition to providing a significant fraction of the overall structural capacity of the pavement, the surface layer must minimize the infiltration of surface water, provide a smooth, uniform, and skid-resistant riding surface, and offer durability against traffic abrasion and the climate.

Figure 1-2 expands the basic components, showing other important features (*e.g.,* drainage systems) that are often included in a pavement design. The permeable base drainage layer in Figure 1-2 is provided to remove infiltrated water quickly from the pavement structure. Suitable features, including edgedrains and drain outlets, must be included in the pavement design for collecting and removing any accumulated water from the drainage layer. In order to function properly, the layer below the drainage layer must be constructed to grades necessary to promote positive subsurface drainage (*i.e.,* no undulations and reasonable crown or cross slope). Filter materials (*e.g.,* geotextiles) may also be required to prevent clogging of the drainage layer and collector system. Pavement drainage is discussed in more detail in Chapters 3, 6, and 7.

The **geotechnical components** of a pavement system as covered in this manual include surfacing aggregate, unbound granular base, unbound granular subbase, the subgrade or roadbed (either mechanically or chemically stabilized, or both), aggregate and geosynthetics used in drainage systems, graded granular aggregate and geosynthetic used as separation and filtration layers, and the roadway embankment foundation. These and other terms related to the components of the pavement system are defined in Appendix A.

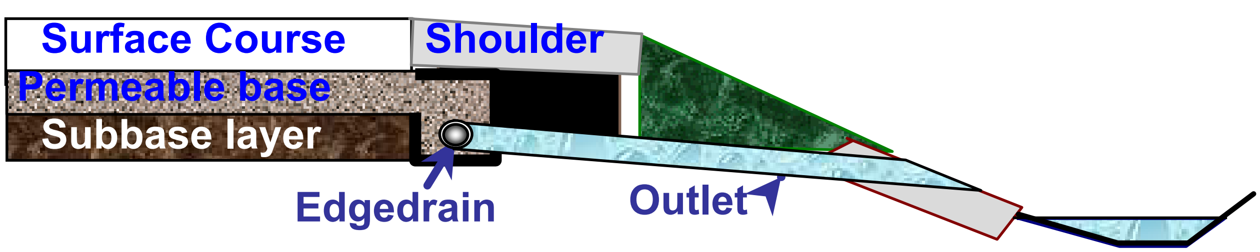


Figure 1-2. Pavement system with representative alternative features.

1.3.2 Alternate Types of Pavement

The most common way of categorizing pavements is by structural type: rigid, flexible, composite and unpaved.

- *Rigid pavements* in simplest terms are those with a surface course of Portland cement concrete (PCC). The Portland cement concrete slabs constitute the dominant load-carrying component in a rigid pavement system.
- *Flexible pavements*, in contrast, have an asphaltic surface layer, with no underlying Portland cement slabs. The asphaltic surface layer may consist of high quality, hot mix asphalt concrete, or it may be some type of lower strength and stiffness asphaltic surface treatment. In either case, flexible pavements rely heavily on the strength and stiffness of the underlying unbound layers to supplement the load carrying capacity of the asphaltic surface layer.
- *Composite pavements* combine elements of both flexible and rigid pavement systems, usually consisting of an asphaltic concrete surface placed over PCC or bound base.
- *Unpaved roads or naturally surfaced roads* simply are not paved, relying on granular layers and the subgrade to carry the load. Seal coats are sometimes applied to improve their resistance to environmental factors.

Pavements can also be categorized based on type of construction:

- *New construction*: The design and construction of a pavement on a previously unpaved alignment. All pavements start as new construction.
- *Rehabilitation*: The restoration or addition of structural capacity to a pavement. Overlays (either asphalt or Portland cement concrete), crack and seat and full or partial depth reclamation are examples of rehabilitation construction.
- *Reconstruction*: The complete removal of an existing pavement and construction of a new pavement on the same alignment. Except for the demolition of the existing pavement (usually done in stages, *i.e.,* one lane at a time) and traffic control during construction, reconstruction is very similar to new construction in terms of design.

Categorization of pavements by structural type is generally the more useful approach for the overall pavement design, as well as performance monitoring and management of the pavement structure. The material types and structural behavior of flexible versus rigid pavements are sufficiently different to require fundamentally different design approaches. Unpaved roads also provide a unique set of challenges and correspondingly unique design requirements. Key features of flexible, rigid, composite pavement systems, and unpaved roads are described in the following subsections.

1.3.3 Flexible Pavements (Adapted from AASHTO 1993)

As was described in Figure 1-1, flexible pavements in general consist of an asphalt-bound surface course or layer on top of unbound base and subbase granular layers over the subgrade soil. In some cases, the subbase and/or base layers may be absent (*e.g.,* full-depth asphalt pavements), while in others the base and/or subbase layers may be stabilized using cementitious or bituminous admixtures. Drainage layers may also be provided to remove water quickly from the pavement structure. Some common variations of flexible pavement systems are shown in Figure 1-3. Full depth asphalt pavements (Figure 1-3 upper right corner) are used primarily for flexible pavements subjected to very heavy traffic loadings.

Hot mix asphalt concrete produced by an asphalt plant is the most common surface layer material for flexible pavements, especially for moderately to heavily trafficked highways. Dense-graded (*i.e.,* well-graded with a low void ratio) aggregates with a maximum aggregate size of about 25 mm (1 in.) are most commonly used in hot mix asphalt concrete, but a wide variety of other types of gradations (*e.g.,* gap-graded) have also been used successfully for specialized conditions. The Superpave procedure has become the standard for asphalt mixture design, although county and local government agencies may still use the older Marshall and Hveem mix design procedures (Asphalt Institute MS-2, 1984).

The asphalt surface layer in a flexible pavement may be divided into sub-layers. Typical sub-layers, proceeding from the top downward, are as follows:

- *Seal coat*: A thin asphaltic surface treatment used to increase (or restore) the water and skid resistance of the road surface. Seal coats may be covered with aggregate when used to increase skid resistance.
- *Surface course* (also called the *wearing course*): The topmost sublayer (in the absence of a seal coat) of the pavement. This is typically constructed of dense graded asphalt concrete. The primary design objectives for the surface course are waterproofing, skid resistance, rutting resistance, and smoothness.
- *Binder course* (also called the *asphalt base course*): The hot mix asphalt layer immediately below the surface course. The base course generally has a coarser aggregate gradation and often a lower asphalt content than the surface course. A binder course may be used as part of a thick asphalt layer either for economy (the lower quality asphalt concrete in the binder course has a lower material cost than the higher-asphalt content concrete in the surface course) or if the overall thickness of the surface layer is too great to be paved in one lift.

Figure 1-3. Some common variations of flexible pavement sections (NCHRP 1-37A, 2002).

Additionally, thin liquid bituminous coatings may be used in the pavement, as follows:

- *Tack coat*: Applied on top of stabilized base layers and between lifts in thick asphalt concrete surface layers to promote bonding of the layers.
- *Prime coat*: Applied on the untreated aggregate base layer to minimize flow of asphalt cement from the asphalt concrete to the aggregate base and to promote a good interface bond. Prime coats are often used to stabilize the surface of the base to

support the paving construction activities above. Cutback asphalt (asphalt cement blended with a petroleum solvent) is typically used because of its greater depth penetration.

Proper compaction of the asphalt concrete during construction is critical for satisfactory pavement performance. Improper compaction can lead to excessive rutting (permanent deformations) in the asphalt concrete layer due to densification under traffic; cracking or raveling of the asphalt concrete due to embrittlement of the bituminous binder from exposure to air and water; and failure of the underlying unbound layers due to excessive infiltration of surface water. Typical construction specifications require field compaction levels of 92% or more of the theoretical maximum density for the mixture. Layers of unbound material below the asphalt concrete layers must be constructed properly in order to achieve the overall objectives of pavement performance.

1.3.4 Rigid Pavements

Rigid pavements in general consist of Portland cement pavement slabs constructed on a granular base layer over the subgrade soil. The base layer serves to increase the effective stiffness of the slab foundation. The base layer also provides the additional functions listed in Section 1.3.1, plus the base must also prevent pumping of fine-grained soils at joints, cracks, and edges of the slab. Gradation characteristics of the base and/or subbase are critical here. The base may also be stabilized with asphalt or cement to improve its ability to perform this function. A subbase layer is occasionally included between the base layer and the subgrade. The effective foundation stiffness will be a weighted average of the subbase and subgrade stiffnesses. For high quality coarse subgrades (*e.g.,* stiffness equal to that of the base) or low traffic volumes (less than 1 million 80-kN (18-kip) ESALs), the base and subbase layer may be omitted.

Because of the low stresses induced by traffic and environmental effects (*e.g.,* thermal expansion and contraction) relative to the tensile strength of Portland cement concrete, most rigid pavement slabs are unreinforced or only slightly reinforced. When used, the function of reinforcement is to eliminate or lengthen spacing of joints, which fault and infiltrate water. Reinforcement in the concrete does not influence subgrade support requirements. The subbase layer may be omitted if there is low truck traffic volume or good subgrade conditions. For high traffic volumes and/or poor subgrade conditions, the subbase may be stabilized using cementitious or bituminous admixtures. Drainage layers can and should be included to remove water quickly from the pavement structure, similar to flexible pavements.

A geotextile layer may be used to control migration of fines into the open graded base layer. Some common variations of rigid pavement systems are shown in Figure 1-4.

Rigid pavement systems are customarily divided into four major categories:

- *Jointed Plain Concrete Pavements (JPCP)*. These unreinforced slabs require a moderately close spacing of longitudinal and transverse joints to maintain thermal stresses within acceptable limits. Longitudinal joint spacing typically conforms to the lane width (around 3.7 m (12 ft)), and transverse joint spacing typically ranges between 4.5 – 9 m (15 – 30 ft). Aggregate interlock, often supplemented by steel dowels or other load transfer devices, provides load transfer across the joints.
- *Jointed Reinforced Concrete Pavements (JRCP)*. The light wire mesh or rebar reinforcement in these slabs is not designed to increase the load capacity of the pavement, but rather to resist cracking under thermal stresses and, thereby, permit longer spacings between the transverse joints between slabs. Transverse spacing typically ranges between 9 – 30 m (30 - 100 ft) in JRCP pavements. Dowel bars or other similar devices are required to ensure adequate load transfer across the joints.

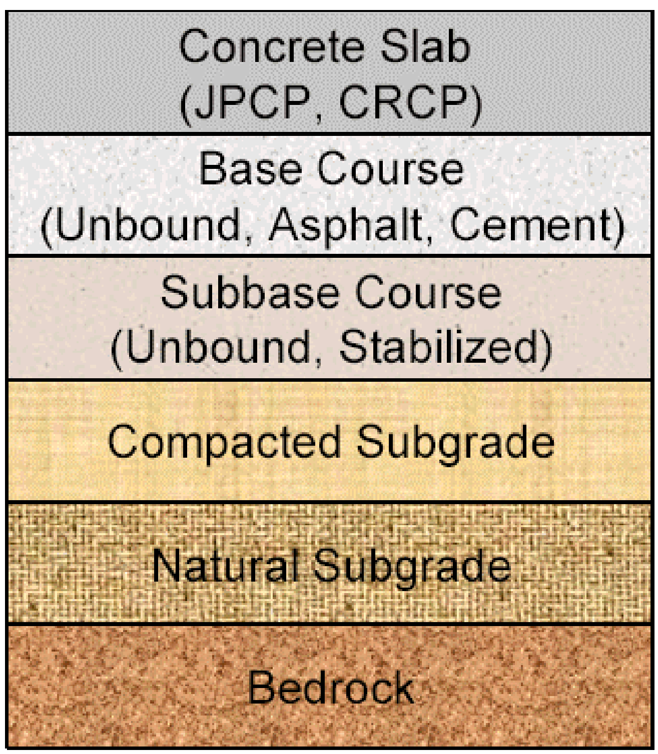


Figure 1-4. Variations for rigid pavement section (NCHRP 1-37A, 2002).

- *Continuously Reinforced Concrete Pavements (CRCP)*. Transverse joints are not required in CRCP pavements. Instead, the pavement is designed so that transverse thermal cracks develop at very close spacings, with typical spacings on the order of a meter (a few feet). The continuous conventional reinforcement bars are designed to hold these transverse thermal cracks tightly together and to supplement the aggregate interlock at the cracks to provide excellent load transfer across the cracks. In addition to the benefit of no transverse joints, CRCP pavement designs are typically 25 – 50 mm (1 – 2 in.) thinner than conventional JPCP or JRCP systems.
- *Prestressed Concrete Pavements (PCP)*. PCP designs are similar to CRCP, except that the longitudinal reinforcement now consists of continuous steel strands that are prestressed prior to placing the concrete (or post-tensioned after the concrete has hardened). The initial tensile stress in the reinforcement counteracts the load- and thermal-induced tensile stresses in the concrete and, therefore, permits thinner slabs. Prestressed concrete pavements are more commonly used for airfield pavements than for highway pavements because of the greater benefit from the reinforcement in the comparatively thicker airfield slabs. Precast, prestressed concrete sections are also being used for pavement rehabilitation.

As suggested above, the basic components in rigid pavement slabs are Portland cement concrete, reinforcing steel, joint load transfer devices, and joint sealing materials. The AASHTO *Guide Specifications for Highway Construction* and the *Standard Specifications for Transportation Materials* provide guidance on mix design and material specifications for rigid highway pavements. These specifications can be modified based on local conditions and experience. Pavement concrete tends to have a very low slump, particularly for use in slip-formed paving. Air-entrainment is used to provide resistance to deterioration from freezing and thawing and to improve the workability of the concrete mix. Joint sealing materials must be sufficiently pliable to seal the transverse and longitudinal joints in JPCP and JRCP pavements against water intrusion under conditions of thermal expansion and contraction of the slabs. Commonly used joint sealing materials include liquid sealants (asphalt, rubber, elastomeric compounds, and polymers), preformed elastomeric seals, and cork expansion filler.

Load transfer devices in JPCP and JRCP pavements are designed to spread the traffic load across transverse joints to adjacent slabs and correspondingly reduce or eliminate joint faulting. The most commonly used load transfer device is a plain, round steel dowel; these are typically 450 mm (18 in.) long, 25 mm (1 in.) in diameter, and spaced at approximately every 300 mm (12 in.) along transverse joints. Tie bars, typically deformed steel rebars, are often used to hold the faces of abutting slabs in firm contact, but tie bars are not designed to act as load transfer devices.

1.3.5 Composite Pavements

Composite pavements consist of asphaltic concrete surface course over PCC or treated bases as shown in Figure 1-5. Composite pavements with PCC over asphalt are also being used. The treated bases may be either asphalt-treated base (ATB) or cement-treated base (CTB). The base layers are treated to improve stiffness and, in the case of permeable base, stability for construction. The composite pavement type shown in Figure 1-5 of an AC overlay on top of a PCC rigid pavement system is a very common rehabilitation scenario.

1.3.6 Unpaved Roads (Naturally Surfaced)

Why use a paved surface? Over one-half of the roads in the United States are unpaved. In some cases, these roads are simply constructed with compacted or modified subgrade. In most cases, a gravel (base) layer is used as the wearing surface. Because of sparse population and low volumes of traffic, these roads will remain unpaved long into the future. Consideration for the subgrade are, again, the same as with flexible pavement, albeit the load levels are generally much higher. The subgrade should also be crowned to a greater extent than paved sections to promote drainage of greater quantities of infiltration surface water. The function of the gravel surfaced is now to carry the load and to provide adequate service. The problems with this approach include roughness, lateral displacement of surface gravel, traction, and dust. Maintenance of ditch lines is also problematic, due to continuous infilling, but open ditches are critical to long-term performance.

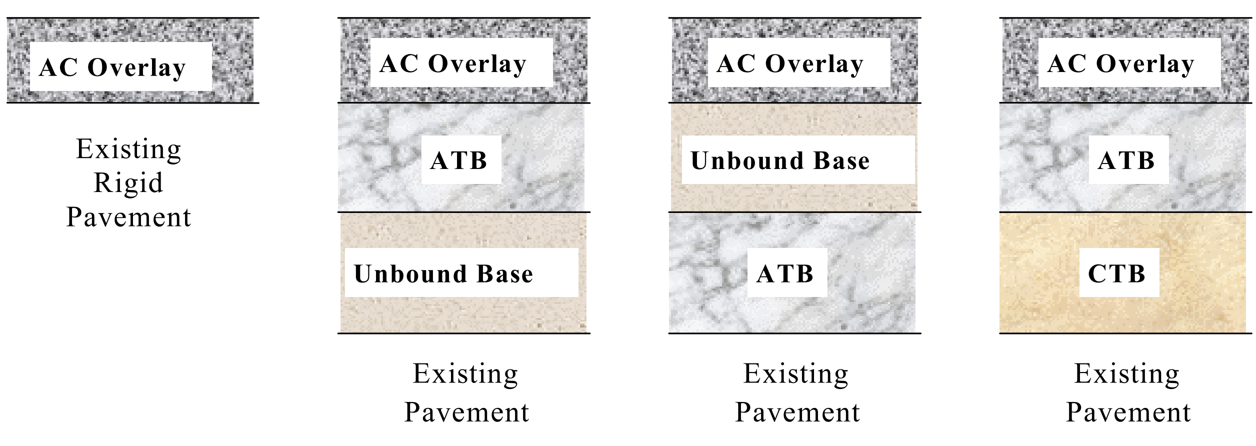


Figure 1-5. Typical composite pavement sections.

1.4 PAVEMENT PERFORMANCE WITH TIES TO GEOTECHNICAL ISSUES

Regardless of which pavement type is used, all of the components make up the pavement system, and failure to properly design or construct any of these components often leads to reduced serviceability or premature failure of the system.

Distress refers to conditions that reduce serviceability or indicate structural deterioration. Failure is a relative term. In the context of this manual, failure denotes a pavement section that experiences excessive rutting or cracking that is greater than anticipated during the performance period, or that a portion of the pavement is structurally impaired at any time during the performance period with incipient failure anticipated from the local distress. There are a number of ways that a pavement section can fail, and there are many reasons for pavement distress and failure.

Yoder and Witczak (1975) define two types of pavement distress, or failure. The first is a structural failure, in which a collapse of the entire structure or a breakdown of one or more of the pavement components renders the pavement incapable of sustaining the loads imposed on its surface. The second type of failure is a functional failure; it occurs when the pavement, due to its roughness, is unable to carry out its intended function without causing discomfort to drivers or passengers or imposing high stresses on vehicles. The cause of these failure conditions may be due to inadequate maintenance, excessive loads, climatic and environmental conditions, poor drainage leading to poor subgrade conditions, and disintegration of the component materials. Excessive loads, excessive repetition of loads, and high tire pressures can cause either structural or functional failures.

Pavement failures may occur due to the intrusion of subgrade soils into the granular base, which results in inadequate drainage and reduced stability. Distress may also occur due to excessive loads that cause a shear failure in the subgrade, base course, or the surface. Other causes of failures are surface fatigue and excessive settlement, especially differential of the subgrade. Volume change of subgrade soils due to wetting and drying, freezing and thawing, or improper drainage may also cause pavement distress. Inadequate drainage of water from the base and subgrade is a major cause of pavement problems (Cedergren, 1987). If the subgrade is saturated, excess pore pressures will develop under traffic loads, resulting in subsequent softening of the subgrade. Under dynamic loading, fines can be literally pumped up into the subbase and/or base.

Improper construction practices may also cause pavement distress. Wetting of the subgrade during construction may permit water accumulation and subsequent softening of the subgrade in the rutted areas after construction is completed. Use of dirty aggregates or contamination of the base aggregates during construction may produce inadequate drainage, instability, and frost susceptibility. Reduction in design thickness during construction due to insufficient subgrade preparation, may result in undulating subgrade surfaces, failure to place proper layer thicknesses, and unanticipated loss of base materials due to subgrade intrusion. Yoder and Witczak (1975) state that a major cause of pavement deterioration is inadequate observation and field control by qualified engineers and technicians during construction.

After construction is complete, improper or inadequate maintenance may also result in pavement distress. Sealing of cracks and joints at proper intervals must be performed to prevent surface water infiltration. Maintenance of shoulders will also affect pavement performance.

Nearly all measures of pavement performance are based upon observations at the surface of the pavement – *e.g.,* surface rutting, cracking of the asphalt or PCC, ride quality, and others. In some cases, these surface distresses are due directly to deficiencies in the asphalt or PCC surface layers, but in many other cases these distresses are caused at least in part by deficiencies from the underlying unbound layers. Since pavement design is ultimately an attempt to minimize pavement distresses and, thereby, maximize pavement performance, it is important to understand how geotechnical factors impact these distresses.

Table 1-1, Table 1-2, and Table 1-3 summarize the geotechnical influences on the major distresses for flexible, rigid, and composite pavements, respectively. The composite pavement type considered in Table 1-3 is an AC overlay on top of a PCC rigid pavement system and a very common rehabilitation scenario.

The dominant geotechnical factor(s) for many pavement distresses is/are the stiffness and/or strength of the unbound materials. In reality, the stresses that develop in any well-designed in-service pavement are well below the failure strength of the unbound materials. As a consequence, the true strength parameters (*i.e.,* the cohesion and friction angle from triaxial tests) are not typically needed or measured for unbound pavement materials. Strength indices like the California Bearing Ratio[1] (CBR) have been commonly measured in the past as an overall indication of the material quality in terms of stiffness and resistance to permanent deformation. More recent trends have been to replace these quality indices with direct stiffness testing via the resilient modulus[2] (M_R). Fortunately, strength and stiffness are

[1] California Bearing Ratio is described in more detail in Chapters 3 and 5.

[2] Resilient Modulus is described in more detail in Chapters 3, 4, and 5.

usually closely correlated in most geomaterials (see, for example, the correlations between M_R and CBR described in Chapter 5).

Table 1-1. Geotechnical influences on major distresses in flexible pavements.

	Insufficient Base Stiffness/Strength	Insufficient Subgrade Stiffness/Strength	Moisture/Drainage Problems	Freeze/Thaw	Swelling	Contamination	Erosion	Spatial Variability
Fatigue Cracking	X	X	X	X		X		
Rutting	X	X	X	X		X		
Corrugations	X							
Bumps				X	X			X
Depressions	X		X	X		X		X
Potholes			X	X				X
Roughness	X	X	X	X	X	X		X

Table 1-2. Geotechnical influences on major distresses in rigid pavements.

	Insufficient Base Stiffness/Strength	Insufficient Subgrade Stiffness/Strength	Moisture/Drainage Problems	Freeze/Thaw	Swelling	Contamination	Erosion	Spatial Variability
Fatigue Cracking	X	X	X	X		X	X	
Punchouts (CRCP)	X	X	X	X		X	X	
Pumping			X				X	
Faulting	X		X	X	X	X	X	
Roughness	X		X	X	X	X	X	X

Table 1-3. Geotechnical influences on major distresses in rehabilitated pavements (AC overlay over PCC).

	Insufficient Base Stiffness/Strength	Insufficient Subgrade Stiffness/Strength	Moisture/Drainage Problems	Freeze/Thaw	Swelling	Contamination	Erosion	Spatial Variability
Reflection Cracking	X		X				X	
Roughness	X		X	X	X		X	X

A major effect of the moisture/drainage, freeze/thaw, and contamination (material from one layer intermixing with another) factors listed in Table 1-1 through 1-3 is to degrade the stiffness and strength of the affected unbound materials. Moisture and drainage are combined here because excessive moisture in the pavement system is often the result of inadequate or malfunctioning drainage systems. Freeze/thaw and swelling can cause heaving of the pavement surface. Erosion can produce voids beneath the surface layers, causing a complete loss of foundation support. The spatial variability factor represents the nonuniformity of the geotechnical factors along the pavement and will, in general, apply to all of the other geotechnical factors.

Note that there are many other important pavement distresses, like thermal cracking, low skid resistance, and others, that are not included in Tables 1-1 through 1-3. The influence of geotechnical factors on these other distresses is generally quite small.

Some further comments on the major distress types are given in the following paragraphs:

Permanent Deformations (Rutting, Bumps, Corrugations, and Depressions). Surface rutting is often the controlling stress mode for flexible pavements. It is sometimes caused by an unstable asphalt concrete mixture that deforms plastically within the first few inches beneath the wheel paths. For a well-designed mixture, however, any rutting observed at the surface will be only partly due to permanent deformations in the asphalt layer, with the remainder due to accumulated permanent deformations in the underlying unbound layers and the subgrade. For example, at the AASHO road test, the percent of final total surface rutting attributable to the asphalt layer averaged 32%, versus 18% for the granular base layer, 39% for the granular subbase, and 11% for the subgrade. In other words, two-thirds of the rutting observed at the surface was due to accumulated permanent deformations in the geomaterials in the pavement structure. Potential causes for excessive permanent deformations in the pavement geomaterials include

- inadequate inherent strength and stiffness of the material.
- degradation of strength and stiffness due to moisture effects (including freeze/thaw); inadequate or clogged drainage systems will contribute to this degradation.
- contamination of base and subbase materials by subgrade fines (*i.e.,* inadequate separation of layer materials).

The shape of the rut trough is usually a good indicator of the source of the permanent deformations. Permanent deformations concentrated in the surface asphalt layers tend to give a narrow rut trough (individual wheel tracks may even be evident), while deep seated permanent deformations from the underlying unbound layers and subgrade typically give a much broader rut trough at the surface.

Nonuniform geotechnical conditions along the pavement can contribute to local permanent deformations in the form of bumps, corrugations, and depressions.

Fatigue Cracking. This form of distress is the cracking of the pavement surface as a result of repetitive loading. It may be manifested as longitudinal or alligator cracking (interconnected or interlaced cracks forming a pattern that resembles an alligator's hide) in the wheel paths for flexible pavement and transverse cracking (and sometimes longitudinal cracking) for jointed concrete pavement. Fatigue cracking in both flexible and rigid pavements is governed by two factors: the inherent fatigue resistance of the surface layer material, and the magnitude of the cyclic tensile strains at the bottom of the layer. The inherent fatigue resistance is clearly dependent only on the properties of the asphalt or PCC. The magnitude of the cyclic tensile strain, on the other hand, is a function of the composite response of the entire pavement structure. Low stiffness in the base, subbase, or subgrade materials – whether due to deficient material quality and/or thickness, moisture influences, or freeze/thaw effects – will all raise the magnitude of the tensile strains in the bound surface layer and increase the potential for fatigue cracking. Localized fatigue cracking may also be caused by nonuniformities in the geomaterials along the pavement alignment – *e.g.,* voids, local zones of low stiffness material, etc.

Reflective Cracking. Reflective cracking in asphalt or concrete surfaces of pavements occurs over joints or cracks in the underlying layers. Like fatigue cracking, reflection cracking of asphalt overlays on top of rigid pavements is governed by the inherent fatigue resistance of the asphalt concrete and the magnitude of the tensile and shear strains in the overlay above the joint in the underlying rigid pavement. Inadequate foundation support (*e.g.,* voids) at the joint will allow differential movement between slabs under a passing vehicle, producing large strains in the overlay above. Intrusion of water, inadequate drainage, and erosion of the unbound base material beneath a joint are all major geotechnical factors influencing reflection cracking.

Potholes. Potholes are formed due to a localized loss of support for the surface course though either a failure in the subgrade or base/subbase layers. Potholes are often associated with frost heave, which pushes the pavement up due to ice lenses forming in the subgrade during the freeze. During the thaw, voids (often filled with water) are created in the soil beneath the pavement surface due to the melting ice and/or gaps beneath the surface pavement resulting from heave. When vehicles drive over this gap, high hydraulic pressure is created in the void, which further weakens the surrounding soil. The road surface cracks and falls into the void, leading to the birth of another pothole. Potholes can also occur as a result of pumping problems.

Punchouts. Punchouts are identified as a broken area of a CRCP bounded by closely spaced cracks usually spaced less than 1 m (3 ft).

Pumping. Pumping is the ejection of foundation material, either wet or dry, through joints or cracks, or along edges of rigid slabs, resulting from vertical movements of the slab under traffic, or from cracks in semi-rigid pavements.

Faulting. Faulting appears as an elevation or depression of a PCC slab in relation to an adjoining slab, usually at transverse joints and cracks.

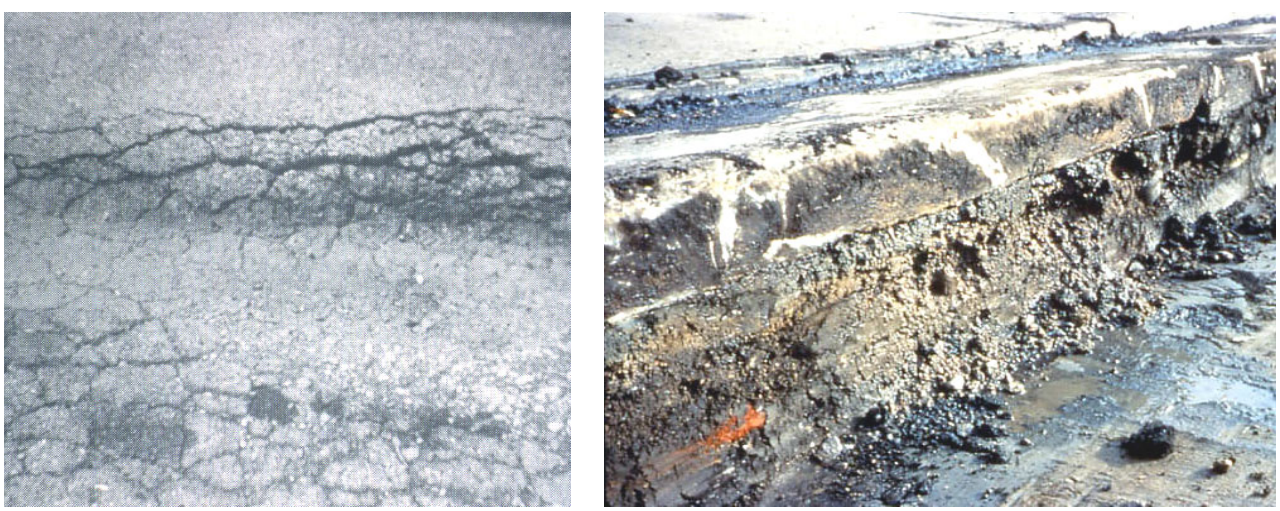
Roughness. Surface roughness is due in large measure to nonuniform permanent deformations and cracking along the wheel path. Consequently, all of the geotechnical factors influencing permanent deformations and cracking will also impact roughness. Nonuniformity of the stiffness/strength of the geomaterials along the pavement, in particular, can be a major contributor to surface roughness. Nonuniform swelling of subgrade soils along the pavement alignment provides a classic example of extreme pavement roughness in some areas of the country.

Liquefaction. The process of transforming any soil from a solid state to a liquid state, usually as a result of increased pore pressure and reduced shearing resistance (ASTM, 2001) is called liquefaction. Spontaneous liquefaction may be caused by a collapse of the structure by shock or other type of strain, and is associated with a sudden, but temporary, increase of the prefluid pressure.

Thermal Cracking. Thermal cracks appear in an asphalt pavement surface, usually full width transverse, as a result of seasonal or diurnal volume changes of the pavement restrained by friction with an underlying layer.

By now it should be apparent that there are a number of ways that a pavement may become impaired to the extent that it is no longer serviceable. In designing a pavement section, the pavement is anticipated to deform over its service life so that at a period in time it will need to be repaired or replaced. Normal failure is defined by rutting of the pavement section, as shown in Figure 1-6, and usually consists of no more than 20 – 25 mm (¾ – 1 in.) within the anticipated performance period. However, as previously reviewed in this section, there are a number of factors that may result in premature failure, long before the performance period, most of which are related to geotechnical issues. Specifically, geotechnical failures, as shown in Figure 1-7, are generally related to excessive subgrade rutting, aggregate contamination or degeneration, subgrade pumping, poor drainage, frost action, and swelling soils. There are other ancillary geotechnical issues, which will impact pavement performance, but are usually addressed in roadway design (*i.e.,* not by the pavement group). These include differential embankment settlement, embankment and cut slope stability, liquefaction, collapsing soils, and karstic (sinkhole) formations. Design methods to evaluate these specific issues, along with procedures to mitigate potential problems, can be found in reference manuals for NHI 132012 Soils and Foundations Workshop (FHWA NHI-00-045 (Cheney & Chassie, 2000)) and NHI 132034 on Ground Improvement Methods (FHWA NHI-04-001 (Elias et al., 2004)).

Figure 1-6. Normal rutting.



a) Excessive rutting | b) Aggregate contamination or degeneration

Figure 1-7. Examples of geotechnical related pavement failures.

c) Subgrade pumping

d) Drainage problems

e) Frost action

f) Swelling soils

g) Differential settlement

h) Collapsing soils, karst conditions, or liquefaction

Figure 1-7. Examples of geotechnical related pavement failures (continued).

1.5 CASE HISTORIES OF PAVEMENT GEOTECHNICS (Failure Examples)

Geotechnical failures are often the result of not recognizing or adequately evaluating conditions prior to construction of the road. The following section provides several case histories of pavement failures that occurred due to inadequate geotechnical information.

1.5.1 Drainage Failure

The existing pavement along a 3 km (2 mile) portion of U.S. Route 1A in a northern state had been plagued by cracking, rutting, and potholes. The highway is a major transport route for tanker trucks that transport oil from the port to a major city. This particular roadway section required frequent maintenance to maintain a trafficable pavement surface, and recently had received a 100-mm (4-in.) overlay. However, within two years of construction, the overlay was badly cracked and rutted, again needing repair. These conditions prompted the reconstruction project. A subsurface investigation encountered moist clay soils (locally known as the Presumpscot Formation) along the entire length of the project. These soils are plastic and moisture sensitive, with water contents greater than 20%. Borings indicated up to 300 mm (12 in.) of asphalt in some sections, and an extensively contaminated base. During the investigation, water was observed seeping out of pavement sections, even though this had been the second driest summer on record in the state. Water in the pavement section was obviously one of the existing pavement section failure mechanisms. Based on soil conditions and past roadway construction experiences, designers initially recommended that the subgrade soils be undercut by 150 mm (6 in.) – with a greater depth of undercut anticipated in some areas – and replaced with granular soil to create a stable working surface prior to placing the overlying subbase course. However, this approach would not solve the drainage problem. Roadway drainage was not conventionally used in this state due to concerns that outlet freezing would prevent effectiveness.

In order to evaluate the most effective repair methods, test sections were established along the alignment consisting of alternate stabilization methods and drainage sections. The test sections were fully instrumented. Monitoring included FWD testing performed prior to reconstruction, after construction and periodically (*e.g.,* before and after the spring thaw) since the project was completed in 1997. An indication of the poor subgrade condition on this project was encountered during construction, when a control section (no stabilization lift) failed and required a 600 mm (24 in.) undercut and gravel replacement to allow construction over the section. A 820 mm (32 in.) pavement section was then constructed over the undercut.

The roadway is performing well in all sections, and at this time (five years after construction) it is too early to determine which stabilization method proved most effective. Minimal frost heave has been observed thus far in all of the test sections, and it may take several additional seasons to provide discernible results. In the drainage section, water flows from the drains and corresponds strongly with precipitation events and water table levels. One surprising result occurs in the spring of each year. More water flows from the drains during the month of spring thaw than all of the other months combined. Over the long-term it is anticipated that this drainage will prove very beneficial to the performance of the pavement system.

1.5.2 Collapsible Soils

Sections of Interstate 15 within a 27 km (17-mi) length of roadway in a western state have been experiencing considerable distress since construction. Maintenance costs have been significant, and it appears that distress may not simply be due to an inadequate pavement section. The problems associated with bumps, cracks, and edge failures were likely associated with troubles in the subgrade soils along the alignment. Potential causes could have included collapsible soil, expansive soil, compressible soil, poorly compacted fill, and poor drainage. A study was performed with the objectives of determining the causes for the problems and developing potential solutions prior to design and reconstruction of the area in question. Based on surficial geology and borehole data, zones were identified where collapsible soils were likely the culprit. Because the zone of collapsible soil extends to depths of up to 6 m (20 ft) below the ground surface, deep dynamic compaction was recommended over excavation and replacement as a treatment method in these zones. Distress related to expansive soils exists throughout the study area, but significant damage concentrations are located in a cut section between mileposts 208 and 207 along I-15. This area is long enough to propose treatments for the area, in order to improve ride quality throughout the cut section. This study recommends a combination of methods to improve the odds of success. Because of the potential for differential settlement on the roadway, asphalt pavement should be used in reconstructing the roadway in the study area. A lack of adequate surface drainage is another critical factor leading to problems with both collapsible and expansive subgrade soils in this area. Deep dynamic compaction was found not to be feasible during construction, most likely due to an intervening fine-grained layer in the deposit.

1.6 CONCLUDING REMARKS

All pavement systems are constructed on earth and practically all components are constructed with earth materials. When these materials are bound with asphalt or cement to form surface layers, they take on a manufactured structural component that is relatively well understood by pavement designers. However, in their unbound state, the properties of these "geotechnical" materials are extremely variable and are the results of the natural processes that have formed them, and natural or man-made events following their formation. Often the earth provides inferior foundation materials in their natural state, but replacement is often impractical and uneconomical. As a result, the design engineer is often faced with the challenge of using the foundation and construction materials available on or near the project site. Therefore, designing and building pavement systems requires a thorough understanding of the properties of available soils and rocks that will constitute the foundation and other components of the pavement system.

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CHAPTER 2.0 STATE AGENCIES

2.1 HOST AGENCY AND OTHER IDENTIFIED GUESTS

Various state agencies take different approaches to pavement design. Local practices often vary in terms of 1) the methods for obtaining subsurface information and laboratory testing in relation to pavement design; 2) design guides followed by the agency (usually a variation of current or previous AASHTO guidelines); and 3) field construction monitoring for subgrade approval and pavement component approval, as well as contractors' quality control requirements for pavement component construction. This chapter presents issues related to each of these items, discussed by a representative of the agency conducting this course. A questionnaire for an agency to document and evaluate their own practice in relation to geotechnical aspects of pavement design is provided.

Issues that should be specifically addressed by each agency include:

- Current subsurface investigation practices and procedures for new construction, reconstruction, and rehabilitation pavement projects, including in-house facilities (field and laboratory) and outsourced capabilities, noting prequalification and QC/QA requirements for outsourced programs.
- Special or complex subsurface conditions (*e.g.,* soft soils, frost susceptible soils, swelling soils, collapsible soils, caliche, karst topography, etc.) encountered in this state.
- The standard pavement systems (including pavement type, base and subbase layers, drainage requirements, and subgrade treatments) currently used by the agency.
- Current design approach (*i.e.,* AASHTO referenced to year of provision, state agency procedure, or other) and implementation status of empirical-mechanistic design approach.
- Current pavement projects requiring special or complex procedures.
- Construction and design verification procedures, including determination of subgrade stabilization requirements during construction (*e.g.,* undercut, use of geosynthetics, lime, etc.).
- Agency organizational structure, as it relates to personnel involved with pavement material evaluation, design, construction, and maintenance.

There are also many impediments such as time, money, and personnel to performing an adequate subsurface investigation program for pavement design. Agencies should always be aware of these issues and the continual work to remove such impediments. The cost benefit of performing an adequate subsurface program will be discussed in Chapter 4. However, a cost-benefit analysis as evaluated in Pavement Management Programs could substantially assist individual states in assessing their own priorities.

2.2. QUESTIONAIRE ON GEOTECHNICAL PRACTICES IN PAVEMENT DESIGN

1. Which of the following pavement design methods (or modification thereof) is currently used by your agency? (Please circle appropriate method and provide details of any modifications.)

   - AASHTO 1972
   - AASHTO 1986
   - AASHTO 1993 with 1998 Supplement
   - Mechanistic-Empirical design (please identify reference method) _______________
   - Other (please identify or describe) _____________________

2. What are the design performance periods (a.k.a. design life) assigned to each of the following type of roads in your state?

Type	**Performance Period (# or years)**	
	Asphalt	**PCC**
I Secondary		
II Primary		
III Interstate and Freeway		

   Comments:

3. Does your current design achieve the performance period? (If no, what is the typical actual performance period, or range?)

4. What method(s) or test(s) do(es) your state use to evaluate subgrades for inputs values (*e.g.,* CBR, R-value, resilient modulus, etc.) to pavement design?

5. Which group within your agency (*i.e.,* pavement design, geotechnical, hydrology, or other) is responsible for design of pavement drainage?

6. Of the following, which method(s) do(es) your state perform for evaluating subgrade conditions in the field?

Method	Frequently	Sometimes	Never
Remote sensing* (*e.g.,* air photo, landsat photos, etc.)			
Geophysical Non-destructive Tests*			
Falling Weight Deflectometer, FWD			
Ground Penetrating Radar, GPR			
Surface Resistivity, SR			
Seismic Refraction			
In-situ Investigation*			
(Cone Penetration Test, CPT)			
(Dynamic Cone Penetration Test, DCP)			
(Standard Penetration Test, SPT)			
Disturbed sampling (usually with borings)			
Undisturbed sampling (usually with borings)			

* Please list the equipment that you have available and identify it as 1) in-house or 2) outsourced.

7. How is the frequency and spacing determined for borings along the alignment (*e.g.,* standard spacing – provide, available info, site recognizance, etc.) and where are the borings located (*e.g.,* centerline, wheel path, shoulder, other)?

8. What method(s) or test(s) do(es) your state use to evaluate/control subgrade construction?

9. Which of the following stabilization methods are used in your state?

Stabilization Method	**Yes**	**No**
Undercut and Backfill		
Thicker Aggregate		
Geotextiles and Aggregate		
Geogrids and Aggregate		
Cement		
Lime		
Lime-Flyash		
Lime-Cement		
Lime-Cement-Flyash		
Bitumen Modification		
Other – Please provide details		

CHAPTER 3.0 GEOTECHNICAL ISSUES IN PAVEMENT DESIGN AND PERFORMANCE

3.1 INTRODUCTION

Satisfactory pavement performance depends upon the proper design and functioning of all of the key components of the pavement system. These include:

1. A wearing surface that provides sufficient smoothness, friction resistance, and sealing or drainage of surface water (*i.e.,* to minimize hydroplaning).
2. Bound structural layers (*i.e.,* asphalt or Portland cement concrete) that provide sufficient load-carrying capacity, as well as barriers to water intrusion into the underlying unbound materials.
3. Unbound base and subbase layers that provide additional strength – especially for flexible pavement systems – and that are resistant to moisture-induced deterioration (including swelling and freeze/thaw) and other degradation (*e.g.,* erodibility, intrusion of fines).
4. A subgrade that provides a uniform and sufficiently stiff, strong, and stable foundation for the overlying layers.
5. Drainage systems that quickly remove water from the pavement system before the water degrades the properties of the unbound layers and subgrade.
6. Remedial measures, in some cases, such as soil improvement/stabilization or geosynthetics to increase strength, stiffness, and/or drainage characteristics of various layers or to provide separation between layers (*e.g.,* to prevent fines contamination).

Traditionally, these design issues are divided among many groups within an agency. The geotechnical group is typically responsible for characterizing the foundation characteristics of the subgrade. The materials group may be responsible for designing a suitable asphalt or Portland cement concrete mix and unbound aggregate blend for use as base course. The pavement group may be responsible for the structural ("thickness") design. The construction group may be responsible for ensuring that the pavement structure is constructed as designed. Nonetheless, the overall success of the design – *i.e.,* the satisfactory performance of the pavement over its design life – is the holistic consequence of the proper design of all of these key components.

Keeping this holistic view in mind, this chapter builds upon the introduction from Chapter 1 and expands upon the major geotechnical considerations in pavement design (*i.e.,* the factors influencing items 3-6 above). The emphasis is on the "big picture," on identifying the key geotechnical issues and describing their potential impact on the pavement design and performance. Most of the issues introduced here are elaborated in subsequent chapters, and forward references to these later sections are given as appropriate. A brief history of the AASHTO highway pavement design techniques is also included to illustrate how geotechnical design considerations have grown in importance and prominence over time.

3.2 BASIC CONCEPTS

Pavements are layered systems designed to meet the following objectives:

- to provide a strong structure to support the applied traffic loads (structural capacity).
- to provide a smooth wearing surface (ride quality).
- to provide a skid-resistant wearing surface (safety).

Additionally, the system must have sufficient durability so that it does not deteriorate prematurely due to environmental influences (water, oxidation, temperature effects).

The unbound soil layers in a pavement provide a substantial part of the overall structural capacity of the system, especially for flexible pavements (often more than 50 percent). As shown in Figure 3-1, the stresses induced in a pavement system by traffic loads are highest in the upper layers and diminish with depth. Consequently, higher quality – and generally more expensive – materials are used in the more highly stressed upper layers of all pavement systems, and lower quality and less expensive materials are used for the deeper layers of the pavement (Figure 3-2). This optimization of material usage minimizes construction costs and maximizes the ability to use locally available materials. However, this approach also requires greater attention to the lower quality layers in the design (*i.e.,* the subgrade) in order to reduce life-cycle pavement costs. Good long-term performance of lower layers means that upper layers can be maintained (rehabilitated) while avoiding the more costly total reconstruction typically associated with foundation failures.

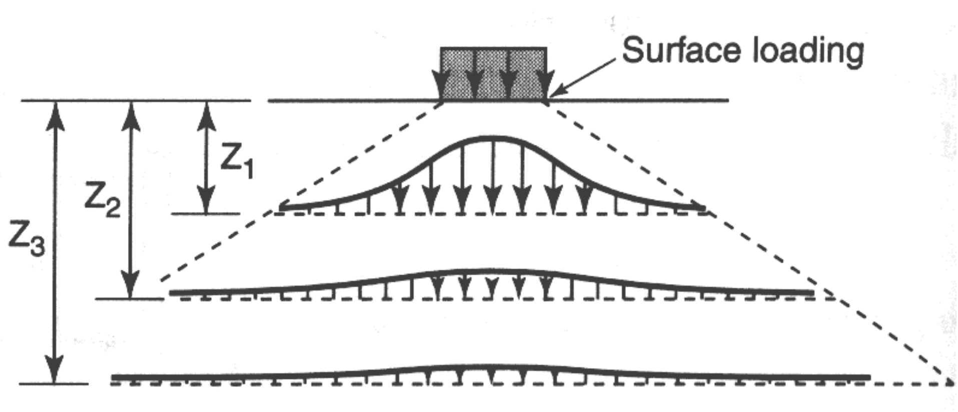


Figure 3-1. Attenuation of load-induced stresses with depth.

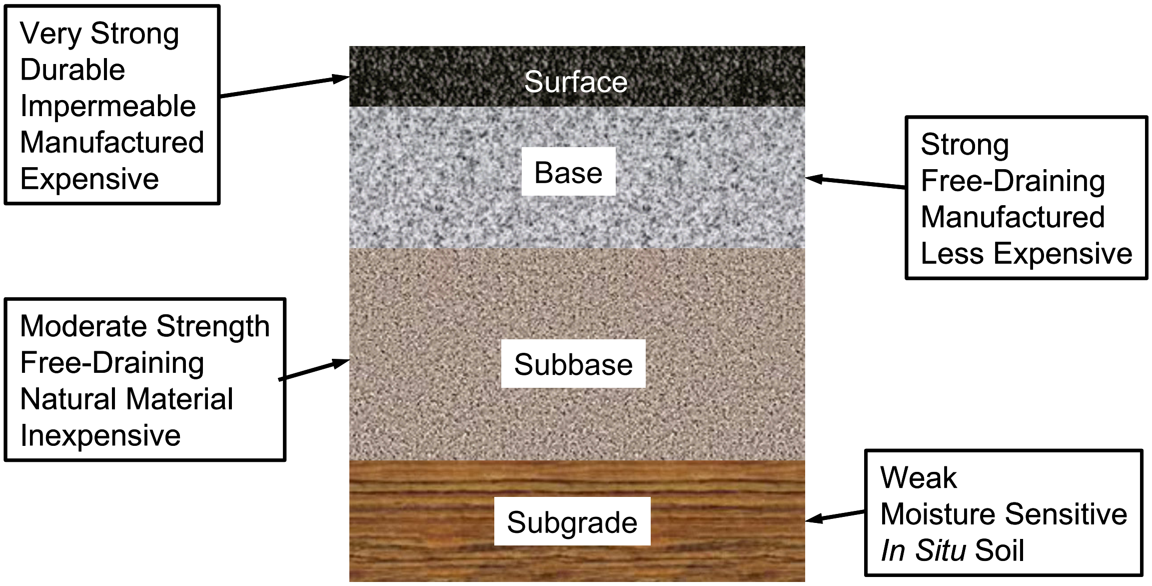


Figure 3-2. Variation of material quality with depth in a pavement system with ideal drainage characteristics.

As is the case for all geotechnical structures, pavements will be strongly influenced by moisture and other environmental factors. Water migrates into the pavement structure through combinations of surface infiltration (*e.g.,* through cracks in the surface layer), edge inflows (*e.g.,* from inadequately drained side ditches or inadequate shoulders), and from the underlying groundwater table (*e.g.,* via capillary potential in fine-grained foundation soils). In cold environments, the moisture may undergo seasonal freeze/thaw cycles. Moisture within the pavement system nearly always has detrimental effects on pavement performance. It reduces the strength and stiffness of the unbound pavement materials, promotes contamination of coarse granular material due to fines migration, and can cause swelling (*e.g.,* frost heave and/or soil expansion) and subsequent consolidation. Moisture can also introduce substantial spatial variability in the pavement properties and performance, which can be manifested either as local distresses, like potholes, or more globally as excessive roughness. The design of the geotechnical aspects of pavements must consequently focus on the selection of moisture-insensitive free-draining base and subbase materials, stabilization of moisture-sensitive subgrade soils, and adequate drainage of any water that does infiltrate into the pavement system. Material selection and characterization is described more fully later in Chapter 5, and pavement drainage design is covered in Chapter 7.

3.3 KEY GEOTECHNICAL ISSUES

The geotechnical issues in pavement design can be organized into two categories: (1) general issues that set the entire tone for the design – *e.g.,* new versus rehabilitation design; and (2) specific technical issues – *e.g.,* subgrade stiffness and strength determination. The geotechnical considerations in each of these categories are briefly introduced in the following subsections. Again, the intent here is to provide an overview of the broad range of geotechnical issues in pavement design. More detailed treatment of each of these issues will be provided in subsequent chapters.

3.3.1 General Issues

New Construction vs. Rehabilitation vs. Reconstruction

The first issue to be confronted in any pavement design is whether the project involves new construction, rehabilitation, or reconstruction. As defined in Chapter 1, new construction is the construction of a pavement system on a new alignment that has not been previously constructed. Rehabilitation is defined as the repair and upgrading of an existing in-service pavement. Typically, this involves repair/removal and construction of additional bound pavement layers (e.g., asphalt concrete overlays) and could include partial-depth or full-depth recycling or reclamation. Reconstruction is defined as the complete removal of an existing pavement system, typically down to and including the upper portions of the foundation soil, and the replacement with a new pavement structure. New construction has been the traditional focus of most pavement design procedures, although this focus has shifted to rehabilitation and reconstruction over recent years, as highway agencies have switched from system expansion to system maintenance and preservation.

New construction vs. rehabilitation vs. reconstruction has a significant impact on several key geotechnical aspects of pavement design. As described more fully in Chapter 4, new construction typically requires substantial "conventional" site characterization work – *e.g.,* examination of geological and soil maps, boring programs, laboratory testing of borehole samples, geophysical subsurface exploration, etc. Little will be known in advance of the soil profiles and properties along the new alignment, so a comparatively comprehensive subsurface exploration and material characterization program is required. Exploration also usually involves evaluation of both cut and fill conditions along the alignment. Access is often limited due to adverse terrain conditions.

For rehabilitation projects, on the other hand, original design documents and as-built construction records are often available to provide substantial background information about the subsurface conditions along the project alignment. The material properties (*e.g.,* subbase stiffness) determined during the initial design may no longer be relevant (*e.g.,* because of contamination from subgrade fines), so new tests may be required, either from laboratory tests on samples extracted from borings through the existing pavement or from in-situ tests like the Dynamic Cone Penetrometer (DCP—see Chapter 4), again via boreholes through the existing pavement structure. Nondestructive evaluation via falling weight deflectometers (FWD—see Chapter 4) is very commonly used to determine in-place material properties for rehabilitation design. Forensic evaluation of the distresses in the existing pavement can also help identify deficiencies in the underlying unbound layers. However, since the underlying unbound layers are not exposed or removed in typical rehabilitation projects, any deficiencies in these layers must be compensated by increased structural capacity, etc., in the added surface layers.

Original design documents and as-built construction records are also often available for reconstruction projects. Information on the original subsurface profile will generally remain relevant for the reconstruction design. However, detailed material characterization from the original design documents will generally not be useful, since the original pavement materials down to and often including the upper portion of the foundation are completely removed and replaced during reconstruction. Although direct testing of the newly exposed foundation soil is theoretically possible in reconstruction projects, this will occur only once construction has begun and, thus, will be too late for design purposes. Consequently, foundation soil properties for reconstruction projects must typically be determined from original design records, borehole sampling and testing, and FWD testing, similar to rehabilitation design. The characterization of the new or recycled unbound subbase and base materials in reconstruction projects will typically be performed via laboratory tests, similar to new construction design.

The influence of new construction vs. rehabilitation vs. reconstruction on site characterization and subsurface exploration is described in detail in Chapter 4. The different methods for characterizing the geotechnical materials in these different types of projects are detailed in Chapter 5.

Natural Subgrade vs. Cut vs. Fill

Pavement construction on a natural subgrade is the classic "textbook" case for pavement design. The subsurface profile (including depth to bedrock and groundwater table) are determined directly from the subsurface exploration program, and subgrade properties needed for the design can be taken from tests on the natural foundation soil in its in-situ condition and in its compacted state, if the upper foundation layer is to be processed and recompacted or removed and replaced during construction. This issue is discussed in greater detail in Chapter 4.

However, the alignment for most highway projects does not always follow the site topography, and consequently a variety of cuts and fills will be required. The geotechnical design of the pavement will involve additional special considerations in cut and fill areas. Attention must also be given to transition zones – *e.g.,* between a cut and an at-grade section—because of the potential for nonuniform pavement support and subsurface water flow.

The main additional concern for cut sections is drainage, as the surrounding site will be sloping toward the pavement structure and the groundwater table will generally be closer to the bottom of the pavement section in cuts. Stabilization of moisture-sensitive natural foundation soils may also be required. Stability of the cut slopes adjacent to the pavement will also be an important design issue, but one that is typically treated separately from the pavement design itself.

The embankments for fill sections are constructed from well compacted material, and, in many cases, this results in a subgrade that is of higher quality than the natural foundation soil. Drainage and groundwater issues will, in general, be less critical for pavements on embankments, although erosion of side slopes from pavement runoff may be a problem, along with long-term infiltration of water. The principal additional concerns for pavements in fill sections will be the stability of the embankment slopes and settlements, either due to compression of the embankment itself or due to consolidation of soft foundation soils beneath the embankment (again, usually evaluated by the geotechnical unit as part of the roadway embankment design).

Information on soil slope and embankment design can be found in the reference manual for FHWA NHI 132033 (FHWA NHI-01-028). Reinforced slope design (often an alternative where steep embankment slopes are required) is addressed in the reference manual for FHWA NHI 132042 (FHWA NHI-00-043). Rock slope design is covered in the FHWA NHI 132035 reference manual (FHWA NHI-99-007).

Environmental Effects

Environmental conditions have a significant effect on the performance of both flexible and rigid pavements. Specifically, moisture and temperature are the two environmentally driven variables that can significantly affect the pavement layer and subgrade properties and, thus, the performance of the pavement. Some of the effects of environment on pavement materials include the following:

- Asphalt bound materials exhibit varying modulus values depending on temperature. Modulus values can vary from 2 to 3 million psi (14,000 to 20,000 MPa) or more during cold winter months to about 100,000 psi (700 MPa) or less during hot summer months.

- Although cementitious material properties like flexural strength and modulus are not significantly affected by normal temperature changes, temperature and moisture gradients can induce significant stresses and deflections—and consequently pavement damage and distresses—in rigid pavement slabs.
- At freezing temperatures, water in soil freezes and the resilient modulus of unbound pavement materials can rise to values 20 to 120 times higher than the values before freezing.
- The freezing process may be accompanied by the formation and subsequent thawing of ice lenses. This creates zones of greatly reduced strength in the pavement structure.
- The top down thawing in spring traps water above the still frozen zone; this can greatly reduce strength of geomaterials.
- All other conditions being equal, the stiffness of unbound materials decreases as moisture content increases. Moisture has two separate effects:
  - First, it can affect the state of stress through suction or pore water pressure. Coarse grained and fine-grained materials can exhibit more than a fivefold increase in modulus as they dry. The moduli of cohesive soils are affected by complex clay-water-electrolyte interactions.
  - Second, it can affect the structure of the soil through destruction of the cementation between soil particles.
- Bound materials are not directly affected by the presence of moisture. However, excessive moisture can lead to stripping in asphalt mixtures or can have long-term effects on the structural integrity of cement bound materials.
- Cement bound materials may also be damaged during freeze-thaw and wet-dry cycles, which causes reduced modulus and increased deflections.

All pavement distresses are affected by environmental factors to some degree. However, it is often very difficult to include these effects in pavement design procedures.

3.3.2 Specific Issues

Material Types and Properties

The major material types encountered in pavement systems are listed in Table 3-1. The geotechnical materials that are the focus of this manual include non-stabilized granular base/subbase materials (including recycled materials), nonstabilized subgrade soils, mechanically and chemically stabilized subgrade soils, and bedrock groups.

Table 3-1. Major material types in pavement systems (NCHRP 1-37A).

Asphalt Materials Hot Mix AC—Dense Graded Central Plant Produced In-Place Recycled Hot Mix AC—Open Graded Asphalt Hot Mix AC—Sand Asphalt Mixtures Cold Mix AC Central Plant Processed In-Place Recycled	**Non-Stabilized Granular Base/Subbase** Granular Base/Subbase Sandy Subbase Cold Recycled Pavement (used as aggregate) RAP (includes millings) Pulverized In-Place
PCC Materials Intact Slabs Fractured Slabs Crack/Seat Break/Seat Rubblized	**Subgrade Soils** Gravelly Soils (A-1; A-2; GW; GP; GM; GC) Sandy Soils Loose Sands (A-3; SW; SP) Dense Sands (A-3; SW; SP) Silty Sands (A-2-4;A-2-5; SM) Clayey Sands (A-2-6; A-2-7; SC) Silty Soils (A-4;A-5; ML; MH) Clayey Soils Low Plasticity Clays (A-6; CL) Dry-Hard Moist Stiff Wet/Sat-Soft High Plasticity Clays (A-7; CH) Dry-Hard Moist Stiff Wet/Sat-Soft
Cementitiously Stabilized Materials Cement Stabilized Materials Soil Cement Lime Cement Flyash Lime Flyash Lime Stabilized/Modified Soils Open Graded Cement Stabilized Materials	**Bedrock** Solid, Massive and Continuous Highly Fractured, Weathered

The material properties of interest in pavement design can be organized into the following categories:

- Physical properties (*e.g.,* soil classification, density, water content)
- Stiffness and/or strength (*e.g.,* resilient modulus, modulus of subgrade reaction, CBR)
- Thermo-hydraulic properties (*e.g.,* drainage coefficients, permeability, coefficient of thermal expansion)
- Performance-related properties (*e.g.,* repeated load permanent deformation characteristics)

Details of the procedures for determining the geotechnical properties required for pavement design are given in Chapter 5. Note that not all material properties will be equally important in terms of their impact on pavement design and performance, and not all properties are required in all pavement design procedures. Stiffness, usually quantified in terms of the resilient modulus (see Chapter 5), is the most important geotechnical property in pavement design and is incorporated explicitly in most current pavement design procedures (*e.g.,* the 1993 AASHTO Pavement Design Guide). Newer mechanistic-empirical design procedures, such as developed in the recently-completed NCHRP Project 1-37A, require more information regarding material properties, particularly in relation to thermo-hydraulic behavior and performance.

Bedrock is worth a brief special mention here because its presence at shallow depths beneath the pavement structure may have a significant impact on pavement construction (Chapter 8), design (Chapters 5 and 6), and performance (Chapter 6). While the precise measurement of bedrock properties like stiffness is seldom if ever warranted, the effect of shallow (less than 3 m (10 ft) depth) bedrock on pavement analyses must be considered. This is especially true for FWD backcalculation procedures used to estimate in-situ material stiffnesses in rehabilitation design (see Chapter 4).

Drainage

As early as 1820, John McAdam noted that, regardless of the thickness of the structure, many roads in Great Britain deteriorated rapidly when the subgrade was saturated:

> "The roads can never be rendered thus perfectly secure until the following principles be fully understood, admitted and acted upon: namely, that it is the native soil which really supports the weight of traffic: that while it is preserved in a dry state, it will carry any weight without sinking, and that it does in fact carry the road and the carriages also; that this native soil must previously be made quite dry, and a covering impenetrable to rain must then be placed over it, to preserve it in that dry state; that the thickness of a road should only be regulated by the quantity of material necessary to form such impervious covering, and never by any reference to its *own* power of carrying weight.
>
> The erroneous opinion so long acted upon and so tenaciously adhered to, that by placing a large quantity of stone under the roads, a remedy will be found for the sinking into wet clay, or other soft soils, or in other words, that a road may be made sufficiently strong *artificially*, to carry heavy carriages, though the subsoil be in a wet state, and by such means to avert the inconveniences of the natural soil receiving water from rain or other causes, has produced most of the defects of the roads of Great Britain." (McAdam, 1820)

It is widely recognized today that excess moisture in pavement layers, when combined with heavy traffic and moisture-susceptible materials, can reduce service life. Freezing of this moisture often causes additional performance deterioration.

Moisture in the subgrade and pavement structure can come from many different sources (Figure 3-3). Water may seep upward from a high groundwater table, or it may flow laterally from the pavement edges and shoulder ditches. However, the most significant source of excess water in pavements is typically infiltration through the surface. Joints, cracks, shoulder edges, and various other defects in the surface provide easy access paths for water.

A major objective in pavement design is to prevent the base, subbase, subgrade, and other susceptible paving materials from becoming saturated or even exposed to constant high moisture levels in order to minimize moisture-related problems. The three main approaches for controlling or reducing moisture problems follow below:

- *Prevent moisture from entering the pavement system*. Techniques for preventing moisture from entering the pavement include providing adequate cross slopes and longitudinal slopes for rapid surface water runoff and sealing all cracks, joints, and other discontinuities to minimize surface water infiltration.
- *Use materials and design features that are insensitive to the effects of moisture.* Materials that are relatively insensitive to moisture effects include granular materials with few fines, cement-stabilized and lean concrete bases, and asphalt stabilized base materials.[1] Appropriate design features for rigid pavements include dowel bars and widened slabs to reduce faulting and inclusion of a subbase between the base and subgrade to reduce erosion and promote bottom drainage. Design features for flexible pavements include full width paving to eliminate longitudinal joints, asphalt stabilized base layers, and use of a subbase to reduce erosion and promote drainage.
- *Quickly remove moisture that enters the pavement system*. A variety of different drainage features are available for removing excess moisture. Features such as underdrains and ditches are designed to permanently lower the water table under the pavement, whereas other features, such as permeable bases and edge drains, are designed to remove surface infiltration water.

Pavement drainage design is described in more detail in Chapter 7. Additional detail can be found in Christopher and McGuffey (1997) and in the reference manual for FHWA NHI Course 131026 *Pavement Subsurface Drainage Design*.

[1] Moisture-induced stripping of asphalt stabilized materials may be a problem for some aggregates and some asphalt cements.

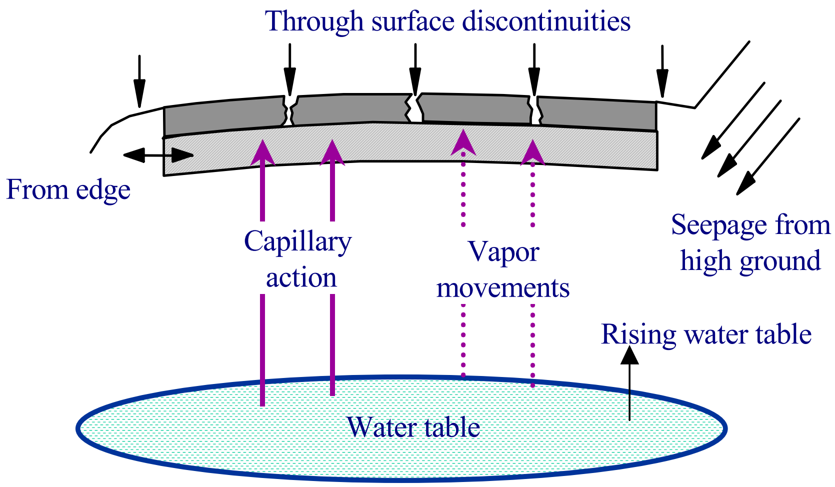


Figure 3-3. Sources of moisture in pavement systems (NHI 13126).

Special Conditions

Special problem soil conditions include frost heave, swelling or expansive soils, and collapsible soils.

Freeze/thaw: The major effect is the weakening that occurs during the spring thaw period. Frost heave during the winter can also cause a severe reduction in pavement serviceability (increased roughness). The requirements for freeze/thaw conditions are (a) a frost-susceptible soil; (b) freezing temperatures; and (c) availability of water.

Swelling or expansive soils: Swelling refers to the localized volume changes in expansive roadbed soils as they absorb moisture. It is estimated that the damage to pavements caused by expansive soils is well over $1 billion each year.

Collapsible soils: Collapsible soils have metastable structures that exhibit large volume decreases when saturated. Silty loess deposits are the most common type of collapsible soil. Native subgrades of collapsible soils must be soaked with water prior to construction and rolled with heavy compaction equipment. If highway embankments are to be constructed over collapsible soils, special remedial measures may be required to prevent large-scale cracking and differential settlement.

Identification of potential problem soils is a primary objective of the pavement geotechnical design. Design approaches and mitigation measures for these special conditions are detailed in Chapter 7.

Soil Improvement

The natural soils at a project site are often unsuitable for use in the pavement structure. They may have inappropriate gradation, inadequate strength and/or stiffness, or insufficient stability against swelling. Some of these deficiencies can be addressed by blending two or more soils and/or providing adequate mechanical stabilization (compaction). Other deficiencies, particularly for subgrades, may require the mixing of stabilizing admixtures such as bituminous binders or lime, Portland cement, or other pozzolanic materials with the natural soil. Although the primary purpose of these admixtures is usually to improve the strength and stiffness of the soil, they can also be used to improve workability, reduce swelling, and provide a suitable construction platform. Geosynthetic products can also be used as soil reinforcement and as filter and drainage layers.

In extreme soft soil conditions, special ground improvement techniques may be required, such as wick drains, piled embankments, surcharge, lightweight fill (*e.g.,* geofoam), etc. These techniques are typically evaluated by the geotechnical unit as part of the roadway design. The methods are discussed briefly in Chapter 7.

A summary of the stabilization methods most commonly used in pavements, the types of soils for which they are most appropriate, and their intended effects on soil properties is provided in Table 3-2. Design inputs for improved soils will be covered in Chapter 5 and details for selection and implementation of treatment techniques for specific problems will be covered in Chapter 7. Compaction, one of the key geotechnical issues in pavement design and construction, is covered in Chapter 5 (determination of design inputs) and Chapter 8 (construction issues).

3.4 SENSITIVITY OF PAVEMENT DESIGN TO GEOTECHNICAL FACTORS

While the most significant layer for pavement performance is the surface course, the geotechnical layers are intimately intertwined in the pavement design. For example, the stiffness or strength of the subgrade soil is a direct input to most pavement design procedures, and its impact on the structural design can thus be evaluated quantitatively. Figure 3-4 shows the influence of the subgrade California Bearing Ratio (CBR—see Chapter 5) on the required thickness and structural capacity contribution for the unbound granular base layer in a flexible pavement designed according to the 1993 AASHTO procedures (see Section 3.5.2). The contribution of the granular base to the overall structural capacity varies from 50% for a low subgrade CBR value of 2 to essentially zero at a high CBR value of 50. The influence of base layer quality on the pavement structural design is similarly shown in Figure 3-5. Additional examples of the sensitivity of pavement design to various geotechnical factors are provided in Chapter 5.

A good indicator of the overall sensitivity of pavement design to geotechnical inputs. is the impact of subgrade support on the cost of the pavement, as shown in Figure 3-6. For example, at a traffic loading of 10 million ESALs and a subgrade CBR of 8, the cost per 1000 square yards (850 m^2) of surface area is approximately $9,800 for the asphalt layer and $3,000 dollars for the underlying base and granular borrow, for a total pavement cost of $12,800 per 1000 square yards of surface area. If the subgrade CBR value were only 4, the same area of pavement section would cost $15,600, or more than 20% more.

It is also important to recognize at the outset that while many of the geotechnical factors influencing pavement performance can be incorporated explicitly in the design process, other important considerations can not. For example, the potential for a slope failure beneath a pavement constructed on a side hill cut is not generally considered as part of "pavement design," even though such a failure can be much more devastating to the pavement than inadequate subgrade stiffness (see Figure 3-7).

Table 3-2. Stabilization methods for pavements (from Rollings and Rollings, 1996).

Method	Soil	Effect	Remarks
Blending	Moderately plastic	None	Too difficult to mix
	Others	Improve gradation Reduce plasticity Reduce breakage	
Lime	Plastic	Drying Immediate strength gain Reduce plasticity Coarsen texture Long-term pozzolanic cementing	Rapid Rapid Rapid Rapid Slow
	Coarse with fines	Same as with plastic soils	Dependent on quantity of plastic fines
	Nonplastic	None	
Cement	Plastic	Similar to lime Cementing of grains	Less pronounced Hydration of cement
	Coarse	Cementing of grains	Hydration of cement
Bituminous	Coarse	Strengthen/bind, waterproof	Asphalt cement or liquid asphalt
	Some fines	Same as coarse	Liquid asphalt
	Fine	None	Can't mix
Pozzolanic and slags	Silts and coarse	Acts as a filler Cementing of grains	Denser and stronger Slower than cement
Misc. methods	Variable	Variable	Depends on mechanism

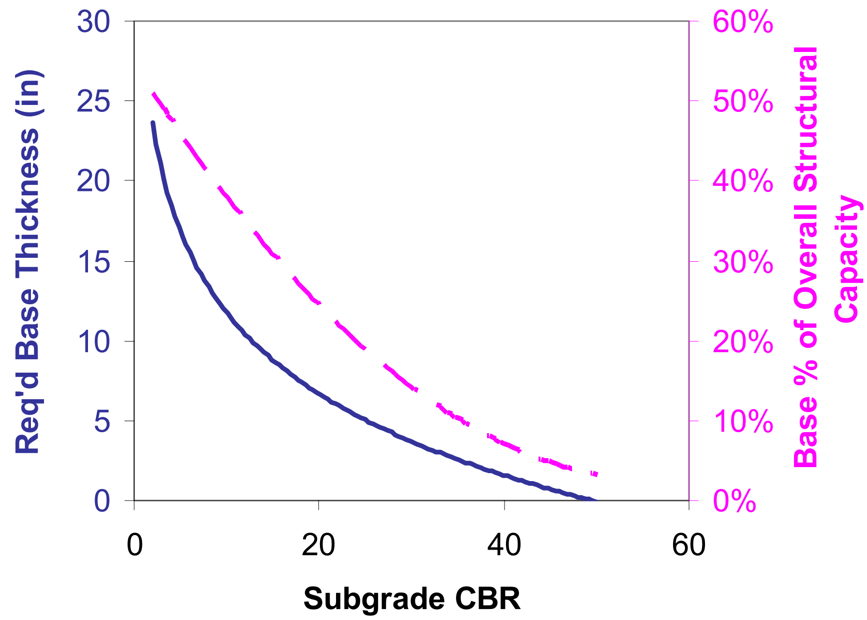


Figure 3-4. Impact of subgrade strength on pavement structural design (AASHTO 93 Design Guide: W_{18}=10M, 85% reliability, S_o=0.4, ΔPSI=1.7, a_1=0.44, a_2=0.14, m_2=1).

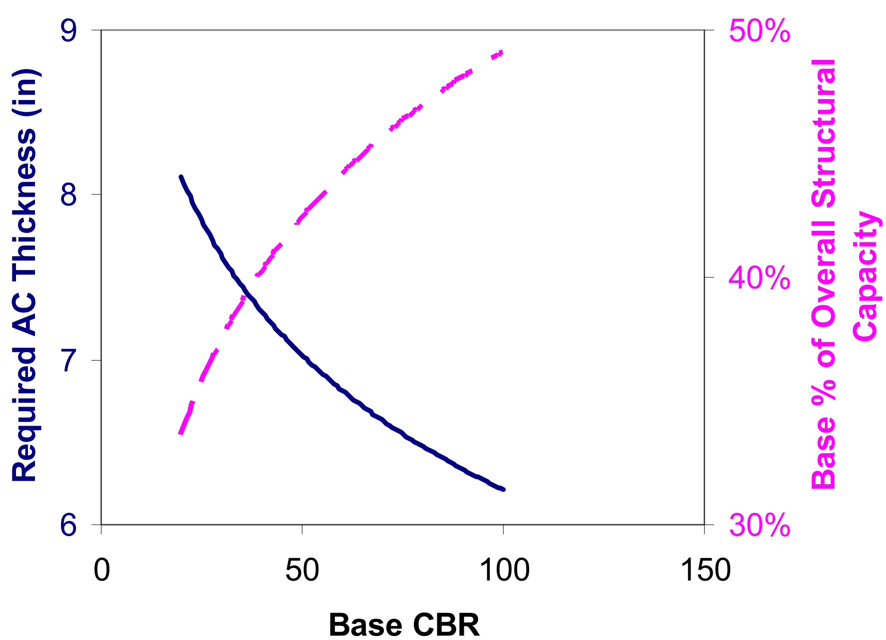
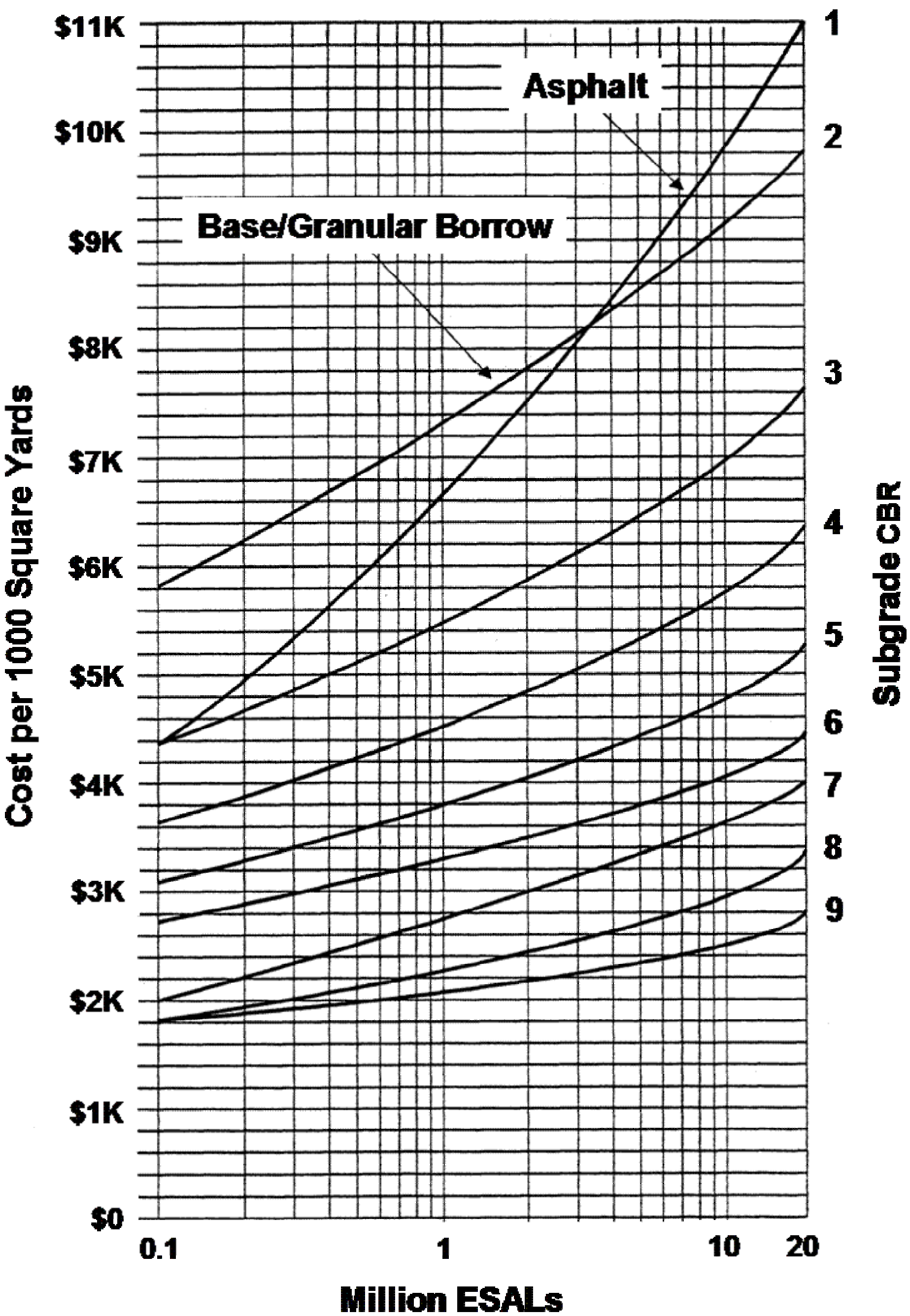


Figure 3-5. Impact of base strength on pavement structural design design (AASHTO 93 Design Guide: W_{18}=10M, 85% reliability, S_o=0.4, ΔPSI=1.7, a_1=0.44, m_2=1, subgrade CBR=4).



Notes:

- Assumed unit costs are: asphalt - \$1.25/inch thickness; untreated base - \$0.30/inch thickness; granular borrow - \$0.20/inch thickness.
- Thicknesses used in cost estimating are based on 90% reliability.
- Minimum granular borrow or base thickness is 6 in.
- Thickness/cost of asphalt only varies with ESALs because base support value is constant.
- Units: 1 inch = 25 mm; 1 yd^2 = 0.85 m^2.

Figure 3-6. Approximate pavement cost for varying subgrade support conditions (B.Vandre, personal communication).

Figure 3-7. Slope failure beneath road pavement (www.geoengineer.com).

3.5 INCORPORATION OF GEOTECHNICAL FACTORS IN PAVEMENT DESIGN

3.5.1 Pavement Design Methodologies

The terms *empirical design*, *mechanistic design*, and *mechanistic-empirical design* are frequently used to identify general approaches toward pavement design. The key features of these design methodologies are described in the following subsections.

Empirical Design

An empirical design approach is one that is based solely on the results of experiments or experience. Observations are used to establish correlations between the inputs and the outcomes of a process – *e.g.,* pavement design and performance. These relationships generally do not have a firm scientific basis, although they must meet the tests of engineering reasonableness (*e.g.,* trends in the correct directions, correct behavior for limiting cases, etc.). Empirical approaches are often used as an expedient when it is too difficult to define theoretically the precise cause-and-effect relationships of a phenomenon.

The principal advantages of empirical design approaches are that they are usually simple to apply and are based on actual real-world data. Their principal disadvantage is that the validity of the empirical relationships is limited to the conditions in the underlying data from which they were inferred. New materials, construction procedures, and changed traffic characteristics cannot be readily incorporated into empirical design procedures.

Mechanistic Design

The mechanistic design approach represents the other end of the spectrum from the empirical methods. The mechanistic design approach is based on the theories of mechanics to relate pavement structural behavior and performance to traffic loading and environmental influences. The mechanistic approach for rigid pavements has its origins in Westergaard's development during the 1920s of the slab on subgrade and thermal curling theories to compute critical stresses and deflections in a PCC slab. The mechanistic approach for flexible pavements has its roots in Burmister’s development during the 1940s of multilayer elastic theory to compute stresses, strains, and deflections in pavement structures.

A key element of the mechanistic design approach is the accurate prediction of the response of the pavement materials – and, thus, of the pavement itself. The elasticity-based solutions by Boussinesq, Burmister, and Westergaard were an important first step toward a theoretical description of the pavement response under load. However, the linearly elastic material behavior assumption underlying these solutions means that they will be unable to predict the nonlinear and inelastic cracking, permanent deformation, and other distresses of interest in pavement systems. This requires far more sophisticated material models and analytical tools. Much progress has been made in recent years on isolated pieces of the mechanistic performance prediction problem. The Strategic Highway Research Program during the early 1990s made an ambitious but, ultimately, unsuccessful attempt at a fully mechanistic performance system for flexible pavements. To be fair, the problem is extremely complex; nonetheless, the reality is that a fully mechanistic design approach for pavement design does not yet exist. Some empirical information and relationships are still required to relate theory to the real world of pavement performance.

Mechanistic-Empirical Design Approach

As its name suggests, a mechanistic-empirical approach to pavement design combines features from both the mechanistic and empirical approaches. The mechanistic component is a mechanics-based determination of pavement responses, such as stresses, strains, and deflections due to loading and environmental influences. These responses are then related to the performance of the pavement via empirical distress models. For example, a linearly elastic mechanics model can be used to compute the tensile strains at the bottom of the asphalt layer due to an applied load; this strain is then related empirically to the accumulation of fatigue cracking distress. In other words, an empirical relationship links the mechanistic response of the pavement to its expected or observed performance.

The development of mechanistic-empirical design approaches dates back at least four decades. Huang (1993) notes that Kerkhoven and Dormon (1953) were the first to use the vertical compressive strain on top of the subgrade as a failure criterion for permanent deformation in flexible pavement systems, while Saal and Pell (1960) recommended the use of horizontal tensile strain at the bottom of the AC layer to minimize fatigue cracking. Likewise, Barenberg and Thompson (1990) note that mechanistic-based design procedures for concrete pavements have also been pursued for many years. Several design methodologies based on mechanistic-empirical concepts have been proposed over the years, including the Asphalt Institute procedure (Shook et al., 1982) for flexible pavements, the PCA procedure for rigid pavements (PCA, 1984), the AASHTO 1998 Supplemental Guide (AASHTO, 1998) for rigid pavements, and the NCHRP 1-26 procedures (Barenberg and Thompson, 1990, 1992) for both flexible and rigid pavements. Some mechanistic-empirical design procedures have also been implemented at the state level (*e.g.,* Illinois, Kentucky, Washington, and Minnesota; see also Newcomb and Birgisson, 1999).

3.5.2 The AASHTO Pavement Design Guides

The *AASHTO Guide for Design of Pavement Structures* is the primary document used to design new and rehabilitated highway pavements. The Federal Highway Administration's 1995-1997 National Pavement Design Review found that some 80 percent of states use the 1972, 1986, or 1993 AASHTO Guides[2] (AASHTO, 1972; 1986; 1993). Of the 35 states that responded to a 1999 survey by Newcomb and Birgisson (1999), 65% reported using the 1993 AASHTO Guide for both flexible and rigid pavement designs.

All versions of the AASHTO Design Guide are empirical methods based on field performance data measured at the AASHO Road Test in 1958-60, with some theoretical support for layer coefficients and drainage factors. The overall serviceability of a pavement during the original AASHO Road Test was quantified by the Present Serviceability Rating (PSR; range = 0 to 5), as determined by a panel of highway raters. This qualitative PSR was subsequently correlated with more objective measures of pavement condition (*e.g.,* cracking, patching, and rut depth statistics for flexible pavements) and called the Pavement Serviceability Index (PSI). Pavement performance was represented by the serviceability history of a given pavement – *i.e.,* by the deterioration of PSI over the life of the pavement (Figure 3-8). Roughness is the dominant factor in PSI and is, therefore, the principal component of performance under this measure.

[2] A 1998 supplement to the 1993 AASHTO Guide (AASHTO, 1998) provides optional alternative methods for rigid pavement and rigid pavement joint design procedures based on recommendations from NCHRP Project 1-30 and verification studies conducted using the LTPP database.

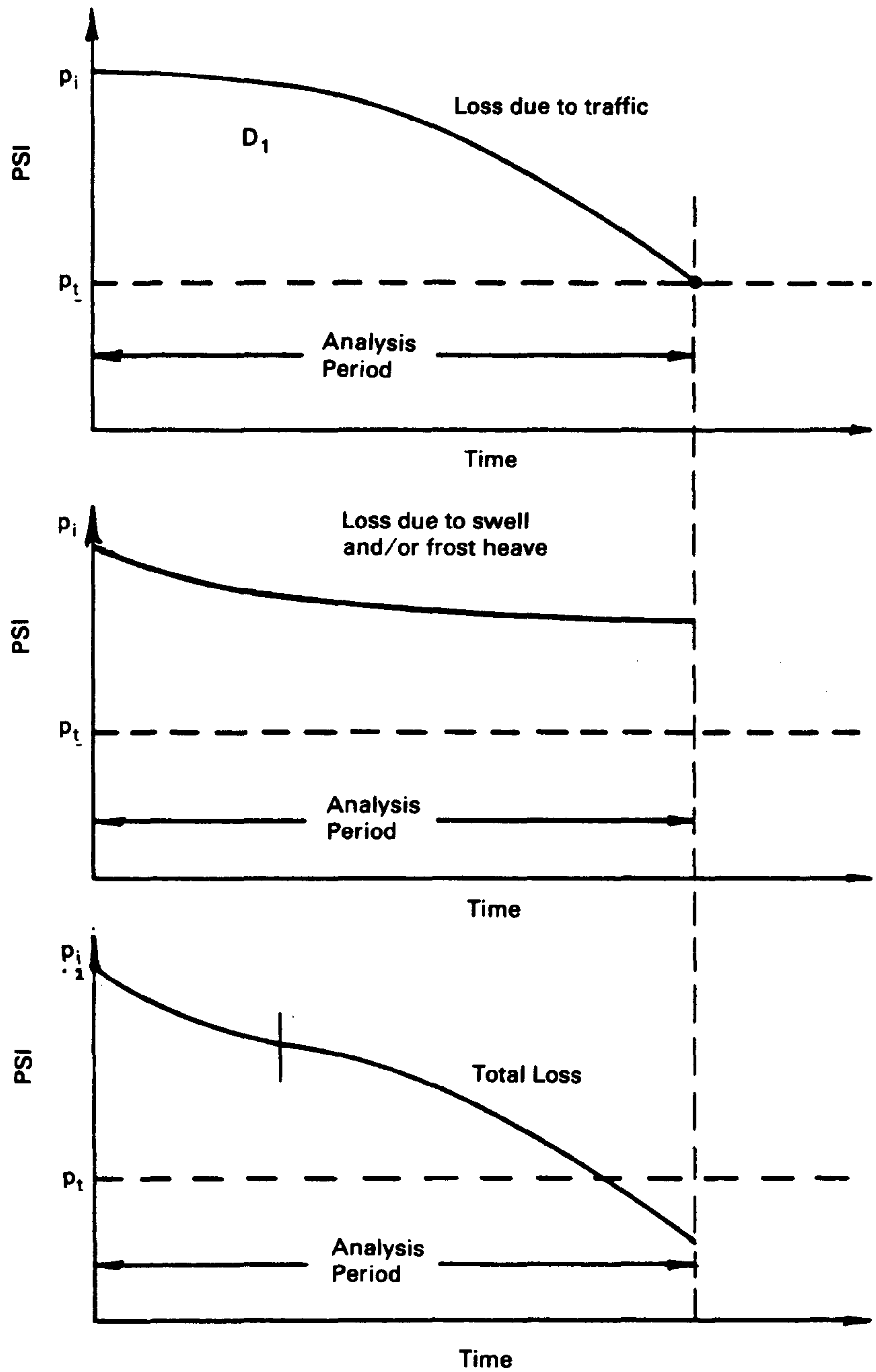


Figure 3-8. Pavement serviceability in the AASHTO Design Guides (AASHTO, 1993).

Each successive version of the AASHTO Design Guide has introduced more and more sophisticated geotechnical concepts into the pavement design process. The 1986 Guide in particular introduced important refinements for materials input parameters, design reliability, and drainage factors, as well as empirical procedures for rehabilitation design. Enhancements were made to both the flexible and rigid design methodologies, although the impact is perhaps more significant for flexible pavements because of the greater contribution of the unbound layers to the structural capacity of these systems. The evolution of geotechnical considerations in the various versions of the AASHTO Design Guides is highlighted in the following sections.

1961 Interim Guide

The 1961 Interim AASHO Pavement Design Guide contained the original empirical equations relating traffic, pavement performance, and structure, as derived from the data measured at the AASHO Road Test (HRB, 1962). These equations were specific to the particular foundation soils, pavement materials, and environmental conditions at the test site in Ottawa, Illinois. The empirical equation for the flexible pavements at the AASHO Road Test is

$$\log W_{18} = 9.36 \log(SN+1) - 0.20 + \frac{\log\left[\left(4.2 - p_t\right)/\left(4.2 - 1.5\right)\right]}{0.4 + 1094/\left(SN+1\right)^{5.19}} \tag{3.1}$$

in which W_{18} = number of 18 kip equivalent single axle loads (ESALs)
p_t = terminal serviceability at end of design life
SN = structural number

Equation (3.1) must be solved implicitly for the structural number *SN* as a function of the other input parameters. The structural number *SN* is defined as

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3 \tag{3.2}$$

in which D_1, D_2, and D_3 are the thicknesses (inches) of the surface, base, and subbase layers, respectively, and a_1, a_2, and a_3 are the corresponding layer coefficients. For the materials used in the majority of the flexible pavement sections at the AASHO Road Test, the values for the layer coefficients were determined as a_1=0.44, a_2=0.14, and a_3=0.11. Note that there may be many combinations of layer thicknesses that can provide satisfactory *SN* values; cost and other issues must be considered as well to determine the final design layer structure.

The corresponding empirical design equation relating traffic, performance, and structure for the rigid pavements at the AASHO Road Test is

$$\log W_{18} = 7.35\log(D+1) - 0.06 + \frac{\log\left[\left(4.5 - p_t\right)/(4.5-1.5)\right]}{1+1.624\times10^{7}/\left(D+1\right)^{8.46}} \tag{3.3}$$

in which D is the pavement slab thickness (inches) and the other terms are as defined previously. Equation (3.3) must be solved implicitly for the slab thickness D as a function of the other input parameters.

Since Eqs. (3.1) through (3.3) are for the specific foundation soils, materials, and environmental conditions at the AASHO Road Test site, there are no geotechnical or environmental inputs to determine. This clearly limited the applicability of these design equations to other sites and other conditions and was the primary motivation behind the development of the 1972 Interim Guide.

1972 Interim Guide

The 1972 Interim Design Guide (AASHTO, 1972) was the first attempt to extend the findings from the AASHO Road Test to foundation, material, and environmental conditions different from those at the test site. This was done through the introduction of several new features for the flexible and rigid pavement design. A rudimentary overlay design procedure was also included in the 1972 Interim Guide.

<u>Flexible Pavements</u>

The major new features added to the 1972 Interim Guide to extend its flexible pavement design methodology to conditions other than those at the AASHO Road Test were:

- An empirical soil support scale to reflect the influence of local foundation soil conditions in Equation (3.1). This soil support scale ranged from 1 to 10, with a soil support value S_i of 3 corresponding to the silty clay foundation soils at the AASHO Road Test site and the upper value of 10 corresponding to crushed rock base materials. All other points on the scale were assumed from experience, with some limited checking through theoretical computations. It is important to note that "the units of soil support, represented by the soil support scale, have no direct relationship to any procedure for testing soils" (AASHTO, 1972) and that it was left up to each agency to determine correlations between soil support and material testing procedures.

- An empirical regional factor R to provide an adjustment to the structural number SN in Equation (3.2) for local environmental and other considerations. Values for the regional factor were estimated from serviceability reduction rates in the AASHO Road Test. These estimates varied between 0.1 and 4.8, with an annual average value of about 1.0. Recommended values for the regional factor based on the AASHO Road Test results are summarized in Table 3-3. However, the Guide cautions that "the regional factor may not adjust for special conditions, such as serious frost conditions, or other local problems" and that "considerable judgment must still be exercised in evaluating [environmental] effects and in selecting an appropriate regional factor for design" (AASHTO, 1972).

- Guidelines for estimating structural layer coefficients a_1, a_2, and a_3 in Equation (3.2) for materials other than those at the AASHO Road Test. These guidelines were based primarily on a survey of state highway agencies regarding the values for the layer coefficients that they were currently using in design for various materials. Ranges of layer coefficient values reported in this survey are summarized in Table 3-4. The Guide recommends that "Because of widely varying environments, traffic, and construction practices, it is suggested that each design agency establish layer coefficients applicable to its own experience. Careful consideration should be given before adoption of values developed by others" (AASHTO, 1972).

Table 3-3. Recommended values for Regional Factor R (AASHTO, 1972).

Roadbed Material Condition	R
Frozen to depth of 5" (130 mm) or more (winter)	0.2 to 1.0
Dry (summer and fall)	0.3 to 1.5
Wet (spring thaw)	4.0 to 5.0

Table 3-4. Ranges of structural layer coefficients from agency survey (AASHTO, 1972).

Coefficient	Low Value	High Value
a_1 (surface)	0.17	0.45
a_2 (untreated base)	0.05	0.18
a_3 (subbase)	0.05	0.14

The modified version of Equation (3.1) for flexible pavements implemented in the 1972 Interim Guide is as follows:

$$\log W_{18} = 9.36\log(SN+1) - 0.20 + \frac{\log\left[\left(4.2-p_t\right)/\left(4.2-1.5\right)\right]}{0.40+1094/(SN+1)^{5.19}} + \log\frac{1}{R} + 0.372\left(S_i - 3.0\right) \tag{3.4}$$

in which R is the regional factor, S_i is the soil support value, and the other terms are as defined previously. As in the 1961 Interim Guide, the thicknesses for each pavement layer are determined as functions of the structural layer coefficients using Equation (3.2) and the required SN determined from Equation (3.4). The principal geotechnical inputs in the design procedure are thus the soil support value S_i for the subgrade and the structural layer coefficients a_2, a_3 and thicknesses D_2, D_3 for the base and subbase layers, respectively.

Rigid Pavements

Only one major new feature was added to the 1972 Interim Guide to extend its rigid pavement design methodology to conditions other than those at the AASHO Road Test. This was the use of the Spangler/Westergaard theory for stress distributions in rigid slabs to incorporate the effects of local foundation soil conditions. The foundation soil conditions are characterized by the overall modulus of subgrade reaction k, which is a measure of the stiffness of the foundation soil.[3]

Interestingly, the modifications made to the rigid pavement design procedure in the 1972 Interim Guide do not include a regional factor for local environmental conditions similar to that implemented in the flexible design procedure. The explanation offered for this was that "it was not possible to measure the effect of variations in climate conditions over the two-year life of the pavement at the Road Test site" (AASHTO, 1972).

The modified version of Equation (3.3) for rigid pavements implemented in the 1972 Interim Guide is as follows:

[3] Although the 1972 Guide does not state this explicitly, it is presumed that the k value for design includes the influence of the subbase layer, if present, as well as the subgrade soil.

$$\log W_{18} = 7.3\log(D+1) - 0.06 + \frac{\log\left[\left(4.5-p_t\right)/\left(4.5-1.5\right)\right]}{1+1.624\times10^7/\left(D+1\right)^{8.46}}$$
$$+\left(4.22-0.32p_t\right)\left[\log\left(\frac{S_c}{215.63J}\right)\left(\frac{D^{0.75}-1.132}{D^{0.75}-18.42/\left(E_c/k\right)^{0.25}}\right)\right] \quad (3.5)$$

in which S_c is the modulus of rupture and E_c is the modulus of elasticity for the concrete (psi), J is an empirical joint load transfer coefficient, k is the modulus of subgrade reaction (pci), and all other terms are as defined previously. Note that k, the principle geotechnical input in the 1972 rigid pavement design procedure, is a "gross" k defined as load (stress) divided by deflection, and as such it includes both elastic and inelastic response of the foundation soil.

For the design of reinforcement in jointed reinforced concrete pavements (JRCP), one additional geotechnical design input is required: the friction coefficient between the slab and the subbase/subgrade.

Sensitivity to Geotechnical Inputs

The sensitivity of the pavement design to the new geotechnical properties in the 1972 AASHTO Guide can be illustrated via some simple examples. Figure 3-9 shows the variation of the required structural number *SN* with the soil support factor S_i for a three-layer (asphalt, base, subgrade) flexible pavement system with design traffic W_{18} = 10 million, regional factor R = 1 (*i.e.,* the environmental conditions at the AASHO Road Test), and terminal serviceability p_t = 2.5. Also shown in the figure is the pavement cost index as a function of soil support, assuming that asphalt is twice as expensive per inch of thickness than crushed stone base and that the cost index equals 1 at S_i = 3 (i.e., the foundation conditions in the AASHO Road Test). Figure 3-10 shows similar variations of SN and cost index with the regional factor R for the same three-layer flexible pavement and S_i = 3. The results for this example suggest that the pavement design and cost is quite sensitive to soil support (cost index varying between 0.3 and 1.3 over the range of valid S_i values), but only moderately sensitive to the regional factor (cost index varying by about $\pm$20% over the range of valid R values).

The sensitivity of rigid pavement slab thickness to the modulus of subgrade reaction k is summarized in Figure 3-11 for three different concrete compressive strength values. The results confirm the conventional wisdom that rigid pavement designs are relatively insensitive to foundation stiffness.

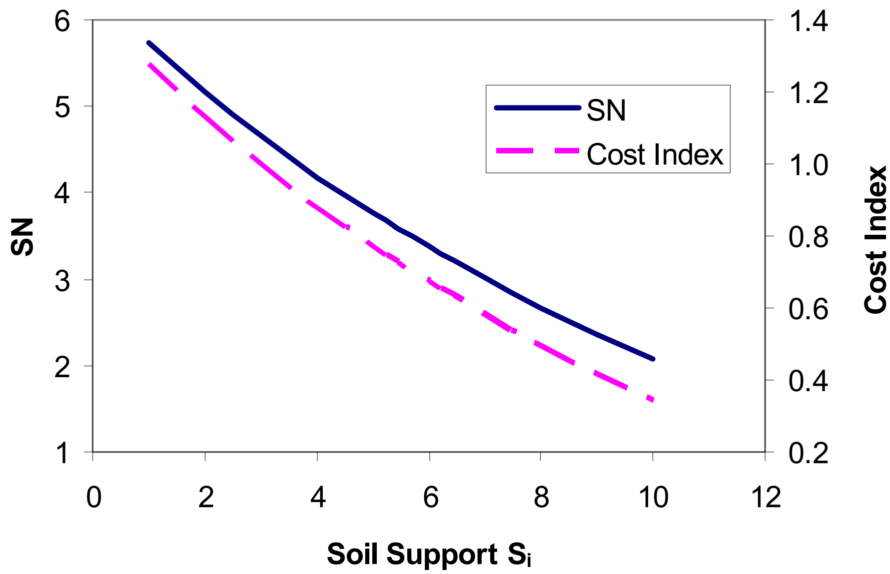


Figure 3-9. Sensitivity of 1972 AASHTO flexible pavement design to foundation support quality.

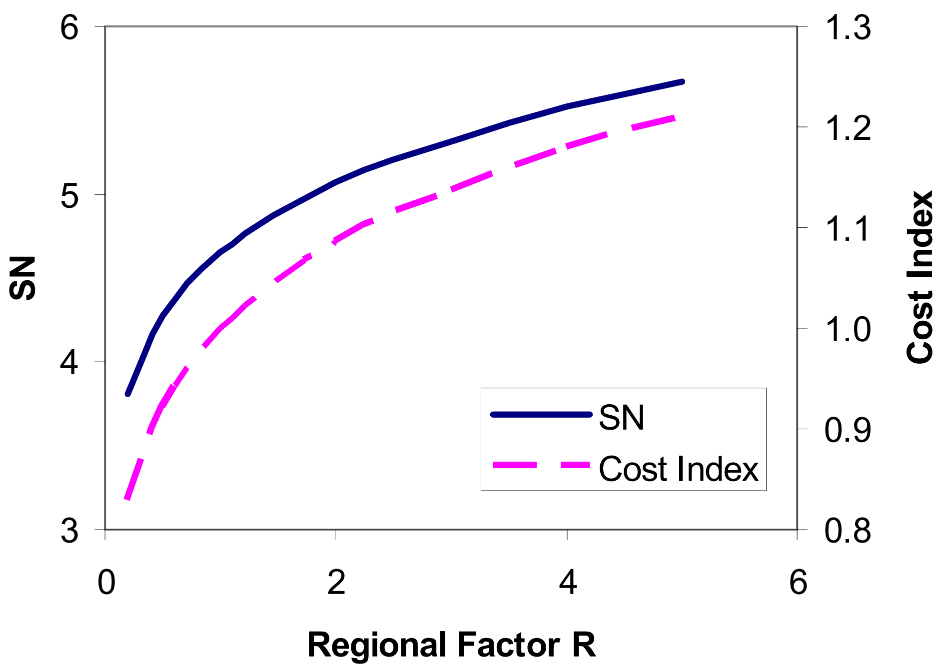


Figure 3-10. Sensitivity of 1972 AASHTO flexible pavement design to environmental conditions.

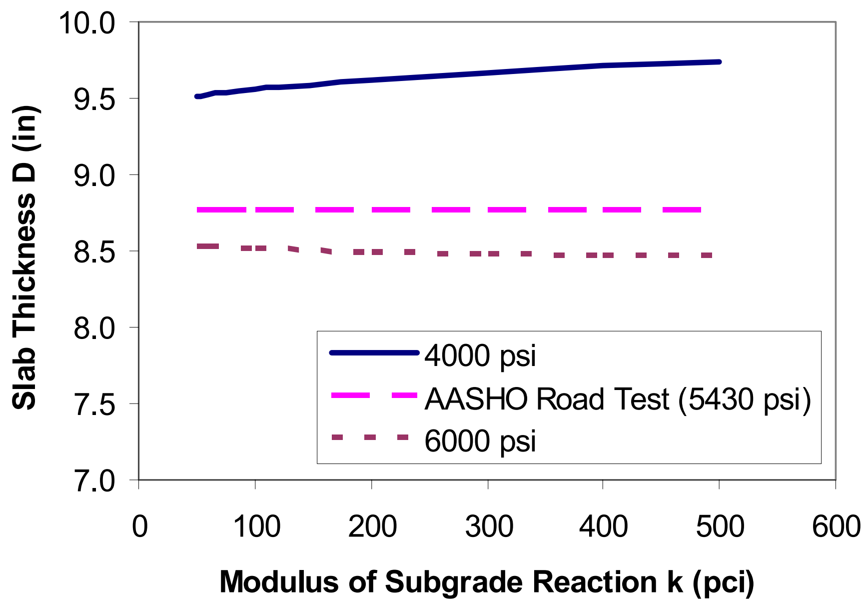


Figure 3-11. Sensitivity of 1972 AASHTO rigid pavement design to foundation stiffness (1 in = 25 mm; 1 pci = 284 MN/m^3).

1986 Guide

The 1986 AASHTO Design Guide (AASHTO, 1986) retained the basic approach from the 1972 Interim Guide but added several new features. Key among these are a more rational characterization of subgrade and unbound materials in terms of the resilient modulus, the explicit consideration of the benefits of pavement drainage (and conversely the consequences of poor drainage), and better treatment of environmental influences on pavement performance. Additional significant enhancements in the 1986 Guide include the incorporation of a reliability factor into the design, expanded treatment of rehabilitation (both with and without overlays), and life-cycle cost analysis.

The geotechnical-related enhancements in the 1986 Guide include the following:

<u>Flexible and Rigid Pavements</u>

- Use of the resilient modulus M_R (AASHTO T272) as a stiffness parameter for characterizing the soil support provided by the subgrade. The resilient modulus M_R is a measure of the elastic stiffness of the soil recognizing certain nonlinear characteristics. It is a basic material property that can be measured directly using established laboratory test protocols, evaluated in-situ from nondestructive tests, or estimated using various empirical relations as detailed later in Chapter 5.

- Improvements in incorporating the effects of environment on pavement performance. Specific emphasis is given to frost heave, thaw-weakening, and swelling of subgrade soils. The enhancements in the 1986 Guide for environmental effects include

    - The explicit separation of total serviceability loss ΔPSI into load- and environment-related components:

    $$\Delta PSI = \Delta PSI_{TR} + \Delta PSI_{SW} + \Delta PSI_{FH} \tag{3.6}$$

    in which ΔPSI_{TR}, ΔPSI_{SW} and ΔPSI_{FH} are the components of serviceability loss attributable to traffic, swelling, and frost heave, respectively.

    - Estimation of an effective resilient modulus for the roadbed that reflects the seasonal variations in subgrade stiffness.

- Incorporation of reliability considerations to reflect the inevitable uncertainty and variability in the design inputs and the importance of the project. Reliability is incorporated in the design through factors that increase the design traffic level.

<u>Flexible Pavements</u>

The geotechnical-related enhancements to the flexible pavement design procedures in the 1986 AASHTO Guide included the following:

- Use of the resilient modulus for determining the structural layer coefficients for both stabilized and unstabilized unbound materials in flexible pavements. The structural layer coefficients a_2 and a_3 for base and subbase materials are estimated via correlations with resilient modulus; these regressions are detailed later in Chapter 5, Section 5.4.5. Nomographs that relate layer coefficients for unstabilized and stabilized base and subbase materials to other strength and stiffness properties are also provided in the 1993 Guide. It is important to remember, however, that these relations for the structural layer coefficients are largely empirical and are based primarily on engineering judgment with only limited amounts of data.

- Guidance for the design of subsurface drainage systems and modifications to the flexible pavement design equations to take advantage of improvements in performance due to good drainage. The benefits of drainage are incorporated into the structural number via empirical drainage coefficients:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \tag{3.7}$$

in which m_2 and m_3 are the drainage coefficients for the base and subbase layers, respectively, and all other terms are as defined previously. The empirical values for m_i, which are specified in terms of quality of drainage and the estimated percentage of time the layer will be near saturation, range from 0.4 to 1.4. Section 5.5.1 in Chapter 5 provides the details for estimating the m_i input values for design. The development of these values can be found in Appendix DD of the 1986 AASHTO Guide.

The modified version of Equation (3.4) for flexible pavements implemented in the 1986 Guide is as follows:

$$\begin{aligned}\log_{10}\left(W_{18}\right) &= Z_R S_0 + 9.36\log_{10}\left(SN+1\right) - 0.20 \\ &+ \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{\left(SN+1\right)^{5.19}}} + 2.32\log_{10}\left(M_R\right) - 8.07\end{aligned} \tag{3.8}$$

in which Z_R is a function of the design reliability level, S_0 is a measure of the overall uncertainty or variability of the design inputs and performance prediction, M_R is the subgrade resilient modulus, and the other terms are as defined previously. Equation (3.7) is used to determine the layer thicknesses required to achieve the total SN value required by Equation (3.8).

In summary, the explicit geotechnical inputs in the 1986 flexible design procedure are the

- seasonally adjusted subgrade resilient modulus M_R,
- base and subbase resilient moduli E_{BS} and E_{SB} (used to determine the a_2 and a_3 structural layer coefficients),
- base and subbase drainage coefficients m_2 and m_3, and
- base and subbase layer thicknesses D_2 and D_3.

Rigid Pavements

The geotechnical-related enhancements to the rigid pavement design procedures in the 1986 AASHTO Guide included the following:

- Guidance for the design of subsurface drainage systems and modifications to the rigid pavement design procedure to take advantage of improvements in performance due to good drainage. The benefits of drainage are incorporated in the rigid pavement design equation via an empirical drainage coefficient C_d. The empirical values for C_d, which are specified in terms of quality of drainage and the estimated percentage of time the pavement will be near saturation, range from 0.7 to 1.25. Section 5.5.1 in Chapter 5 provides the details for estimating the C_d input values for design.

- Enhancements to the procedures for estimating a composite modulus of subgrade reaction that explicitly incorporate the influence of subbase type and thickness, the presence of shallow bedrock, and seasonal variations in subgrade and subbase resilient moduli.

- Adjustment of the design equations to account for the potential loss of support arising from subbase erosion and/or differential vertical soil movements. A loss of support factor *LS* is used to determine the effective *k* value for the foundation soil. Section 5.4.6 in Chapter 5 summarizes the recommended values for *LS* in the 1986 AASHTO Guide for various subbase material types.

The modified version of Equation (3.5) for rigid pavements implemented in the 1986 Guide is as follows:

$$\log_{10}\left(W_{18}\right) = Z_R S_o + 7.35\log_{10}\left(D+1\right) - 0.06$$

$$+\frac{\log_{10}\left[\dfrac{\Delta PSI}{4.5-1.5}\right]}{1+\dfrac{1.64\text{x}10^7}{\left(D+1\right)^{8.46}}} + \left(4.22-0.32p_t\right)\log_{10}\left[\frac{S_c C_d\left(D^{0.75}-1.132\right)}{215.63J\left[D^{0.75}-\dfrac{18.42}{\left(E_c/k\right)^{0.25}}\right]}\right] \tag{3.9}$$

in which C_d is the drainage coefficient and the other terms are as defined previously.

In summary, the explicit geotechnical inputs in the 1986 rigid pavement design procedure are:

- The seasonally adjusted effective modulus of subgrade reaction *k*. This in turn is a function of the seasonally adjusted values for the subgrade and subbase resilient moduli M_R and E_{SB}, the thickness of the subbase D_{SB}, the subgrade depth to rigid foundation D_{SG}, and the loss of support factor *LS*.
- The drainage coefficient C_d.
- A friction factor related to the frictional resistance between the slab and subbase/subgrade for reinforcement design in JRCP pavements.

Sensitivity to Geotechnical Inputs

The key geotechnical inputs in the 1986 AASHTO design procedure for flexible pavements are

- foundation stiffness, as characterized by the subgrade resilient modulus (M_R), and
- moisture and drainage, as characterized by the layer drainage coefficients (m_i).

For rigid pavements, the key geotechnical inputs are

- foundation stiffness, as characterized by the resilient moduli of the subgrade (M_R) and granular subbase (E_{SB}) and the thickness of the subbase (D_{SB}).
- erodibility of the granular subbase, as characterized by the Loss of Support factor (*LS*).
- moisture and drainage, as characterized by the drainage coefficient (C_d).

The sensitivity of the pavement design to the geotechnical inputs in the 1986 AASHTO Guide can be illustrated via some simple examples. Table 3-5 summarizes assumed baseline design inputs for a typical flexible pavement section. These values (except for traffic) generally conform to those at the AASHO Road Test. The variation of required pavement structure with subgrade stiffness and drainage for these conditions are summarized in Figure 3-12 and Figure 3-13, respectively. Also shown in these figures is a pavement cost index, which is based on the assumption that asphalt concrete is twice as expensive as crushed stone base per inch of thickness; the cost index is normalized to 1.0 at baseline conditions (*i.e.,* values in Table 3-5). The vertical cost axes in Figure 3-12 and Figure 3-13 have been kept constant in order to highlight the relative sensitivities of cost to subgrade stiffness and drainage conditions. The horizontal axes in the figures span the full range of stiffness and drainage conditions for flexible pavements.

Both the structural number and pavement cost are highly sensitive to foundation stiffness. As shown in Figure 3-12, reducing M_R from 20,000 psi (138 MPa, corresponding to a CBR of about 30) to 2000 psi (13.8 MPa, corresponding to a CBR value of about 2) results in a 115% increase in required total structural number. This translates to a corresponding 170% increase in cost.

From Equation (3.8), it is clear that changing the drainage coefficient m_2 for the base layer will not affect the total required structural number *SN* (nor will it directly affect the required structural number for each of the layers). However, changes in drainage do directly affect the structural effectiveness of the granular material in the base layer and, thus, its thickness and cost. As shown in Figure 3-13, reducing m_2 from its maximum value of 1.4 to its minimum value of 0.4 requires more than a 3-fold increase in required base thickness. This translates to a 150% increase in overall pavement structural cost for these example conditions.

A similar sensitivity analysis can be performed for the rigid pavement design procedure in the 1986 AASHTO Guide. Table 3-6 summarizes assumed design inputs for a typical rigid pavement section. Again, these values (except for traffic) generally conform to those at the AASHO Road Test. The variations of required slab thickness with foundation stiffness, base erodibility, and drainage conditions are summarized in Figure 3-14, Figure 3-15, and Figure 3-16, respectively. The vertical axes in Figure 3-14 through Figure 3-16 have been kept constant in order to highlight the relative sensitivities of slab thickness to the respective geotechnical inputs. Since rigid pavement cost essentially varies directly with slab thickness, a cost index is not included in the figures. The horizontal axes in the figures span the full range of stiffness, erodibility, and drainage conditions for rigid pavements.

Table 3-5. Flexible pavement baseline conditions for 1986 AASHTO sensitivity study.

Input Parameter	Design Value
Traffic (W_{18})	10×10^6 ESALs
Reliability	90%
Reliability factor (Z_R)	-1.282
Overall standard error (S_o)	0.45
Allowable serviceability deterioration (ΔPSI)	1.7
Subgrade resilient modulus (M_R)	3,000 psi (20.7 MPa)
Granular base resilient modulus (E_{BS})	30,000 psi (207 MPa)
Granular base layer coefficient (a_2)	0.14
Granular base drainage coefficient (m_2)	1.0
Asphalt concrete layer coefficient (a_1)	0.44

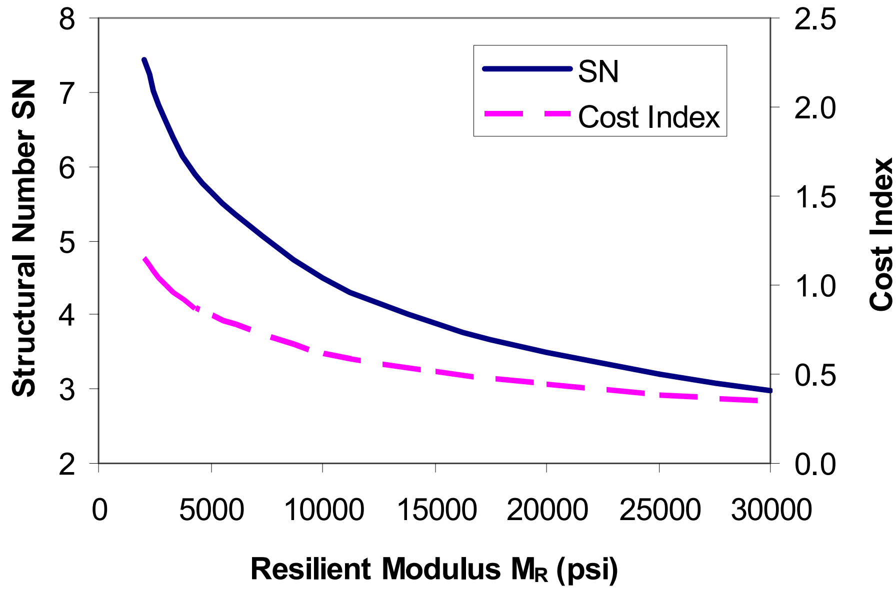


Figure 3-12. Sensitivity of 1986 AASHTO flexible pavement design to subgrade stiffness (1 psi = 6.9 kPa).

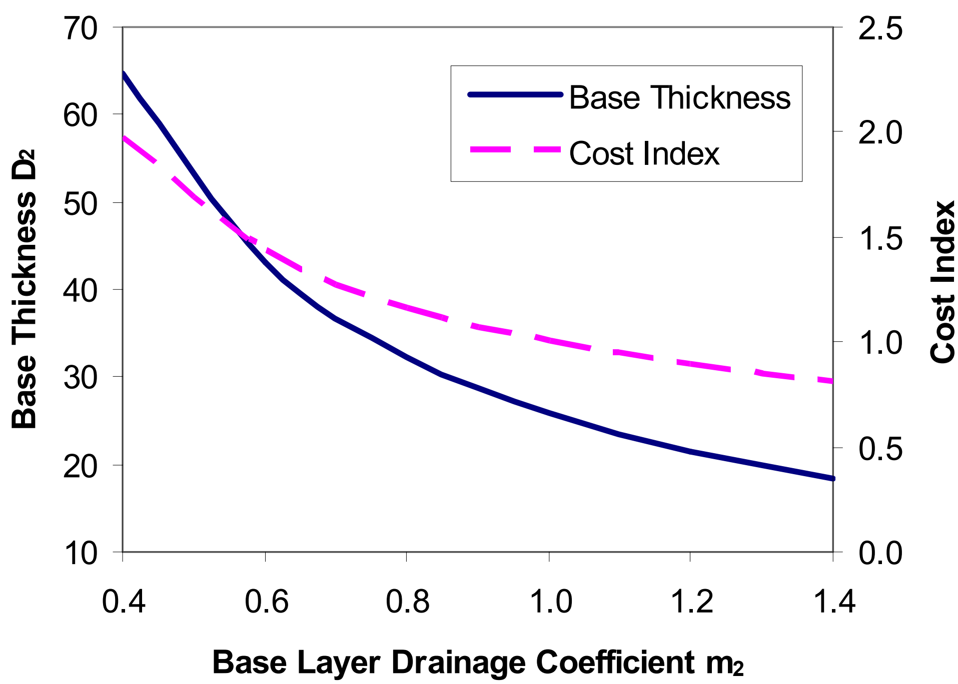


Figure 3-13. Sensitivity of 1986 AASHTO flexible pavement design to drainage conditions (1 inch = 25 mm).

Figure 3-14 clearly shows that slab thickness is quite insensitive to foundation stiffness. This conforms to conventional wisdom, and in fact is one of the reasons that rigid pavements are often considered when foundation soils are very poor. Erodibility of the granular subbase is somewhat more important. As shown in Figure 3-15, increasing *LS* from 0 (least erodible) to 3 (most erodible) results in an additional 1.0 inch (25 mm) of required slab thickness. By far the most important rigid pavement geotechnical input is the moisture/drainage condition. As shown in Figure 3-16, decreasing the drainage coefficient C_d from its maximum value of 1.25 to its minimum value of 0.7 results in a 3.5 inch (87.5 mm) or 35% increase in required slab thickness for these example conditions.

Table 3-6. Rigid pavement baseline conditions for 1986 AASHTO sensitivity study.

Input Parameter	Design Value
Traffic (W_{18})	10x10^6 ESALs
Reliability	90%
Reliability factor (Z_R)	-1.282
Overall standard error (S_o)	0.35
Allowable serviceability deterioration (ΔPSI)	1.9
Terminal serviceability level (p_t)	2.5
Subgrade resilient modulus (M_R)	3,000 psi (20.7 MPa)
Granular subbase resilient modulus (E_{SB})	30,000 psi (207 MPa)
Drainage coefficient (C_d)	1.0
Loss of Support (*LS*)	1.0
PCC modulus of rupture (S_c')	690 psi (4.8 MPa)
PCC modulus of elasticity (E_c)	4.2x10^6 psi (29 GPa)
Joint load transfer coefficient (*J*)	4.1

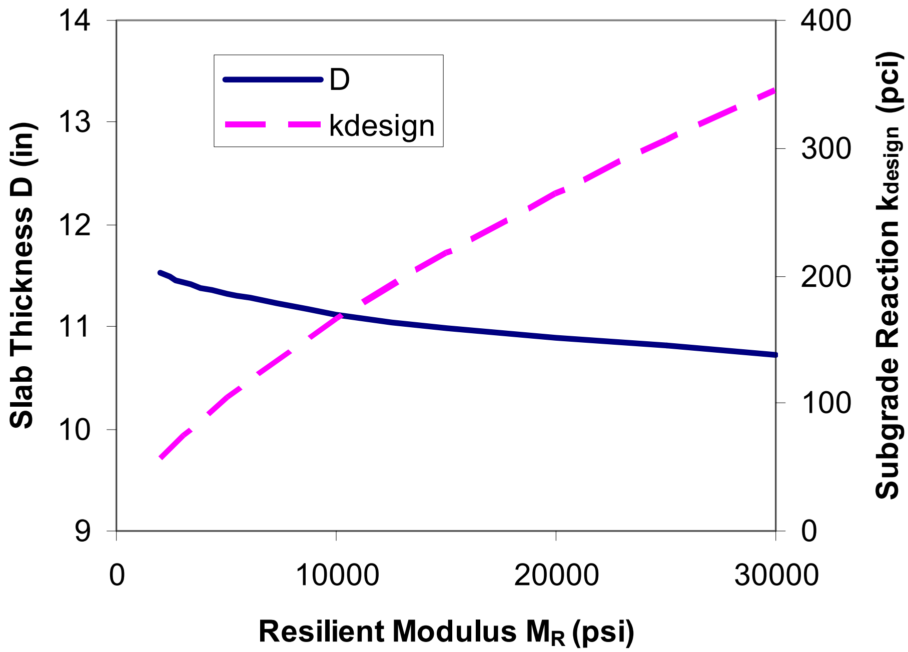


Figure 3-14. Sensitivity of 1986 AASHTO rigid pavement design to subgrade stiffness (1 inch = 25 mm; 1 psi = 6.9 kPa; 1 pci = 284 MN/m^3).

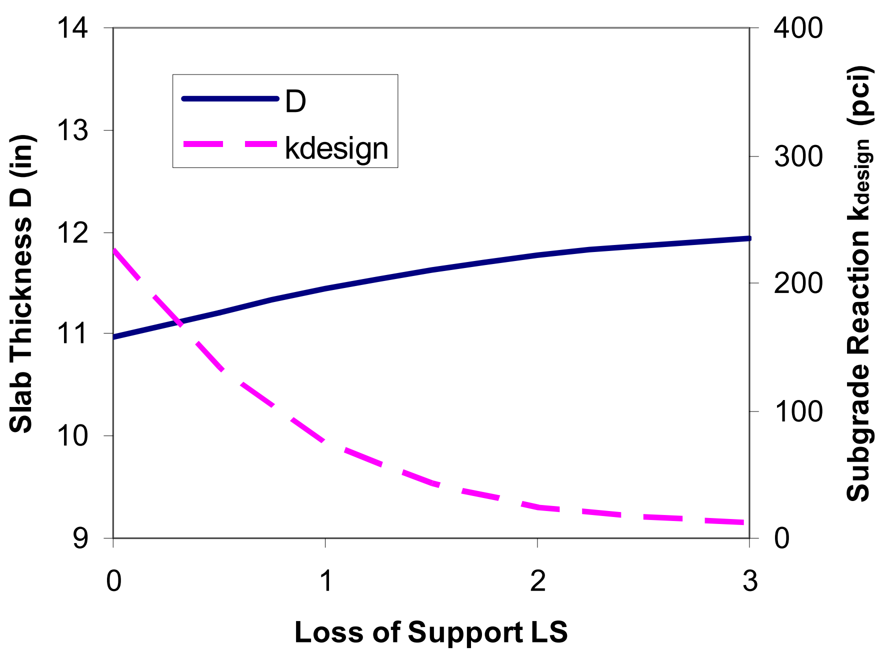


Figure 3-15. Sensitivity of 1986 AASHTO rigid pavement design to subbase erodibility (1 inch = 25 mm; 1 pci = 284 MN/m^3).

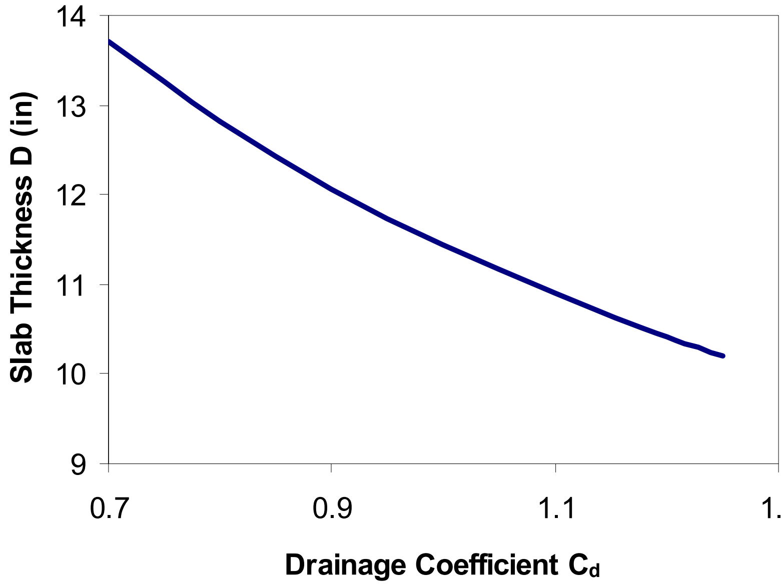


Figure 3-16. Sensitivity of 1986 AASHTO rigid pavement design to drainage conditions (1 inch = 25 mm).

Another of the new parameters introduced in the 1986 Design Guide is design reliability. The target reliability level is set by agency policy; Table 3-7 summarizes common recommendations for design reliability for different road categories. Although reliability is not strictly a geotechnical parameter, it is useful to examine the sensitivity of pavement designs to the target reliability level. Figure 3-17 and Figure 3-18 summarize the sensitivity of the example flexible and rigid pavement designs (design inputs in Tables 3-5 and 3-6) to the design reliability level. It is clear from these figures that the required pavement structure is quite sensitive to the design reliability level, especially for the higher reliability levels. Increasing the design reliability level from 50% to 99.9% increases both the required *SN* and cost for flexible pavements by approximately 50% for these example conditions. The increase in required slab thickness for rigid pavements is of a similar magnitude. These increases in design structure in essence correspond to a safety factor based on agency policy for the design reliability level.

Table 3-7. Suggested levels of reliability for various functional classifications (AASHTO 1986).

Functional classification	Recommended level of reliability (%)	
	Urban	Rural
Interstate and other freeways	85-99.9	80-99.9
Principal arterials	80-99	75-95
Collectors	80-95	75-95
Local	50-80	50-80

Note: Results based on a survey of AASHTO Pavement Design Task Force.

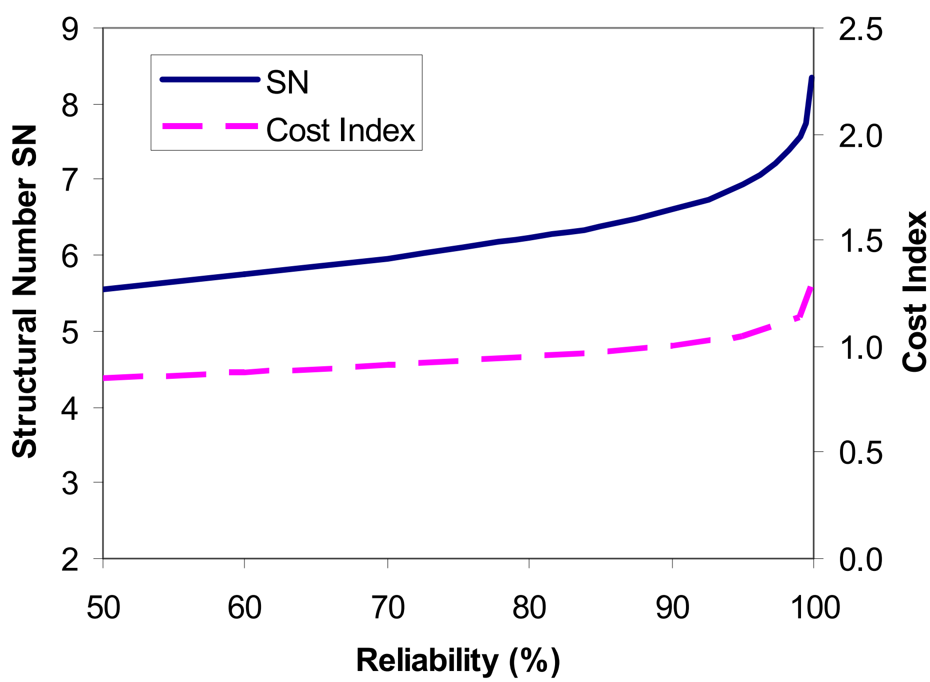


Figure 3-17. Sensitivity of 1986 AASHTO flexible pavement design to reliability level.

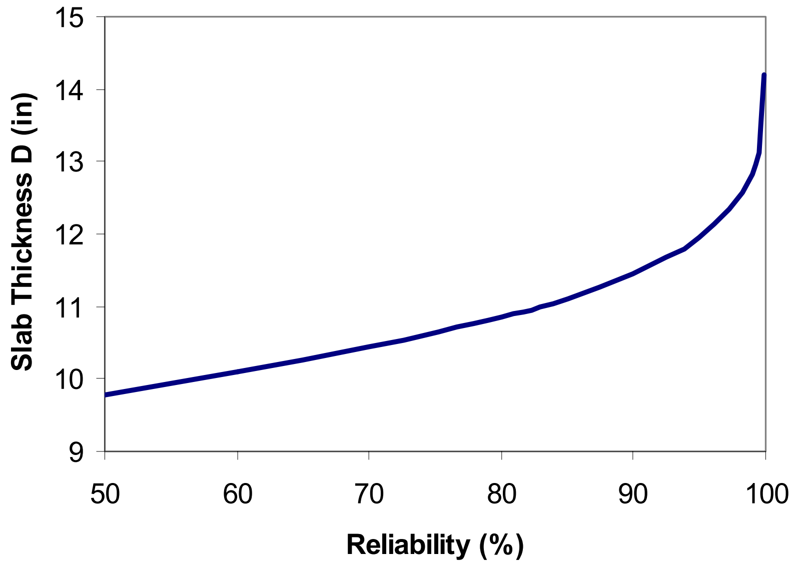


Figure 3-18. Sensitivity of 1986 AASHTO rigid pavement design to reliability level (1 inch = 25 mm).

1993 Guide

The major additions to the 1993 version of the AASHTO Pavement Design Guide (AASHTO, 1993) were in the areas of rehabilitation designs for flexible and rigid pavement systems using overlays. The only significant change to the geotechnical aspects of pavement design was the increased emphasis on nondestructive deflection testing for evaluation of the existing pavement and backcalculation of layer moduli. All other geotechnical aspects are identical to those in the 1986 Guide.

A summary of the design procedures for flexible and rigid pavements in the 1993 AASHTO Guide is provided in Appendix C. A detailed discussion of the key geotechnical inputs in the 1993 AASHTO Guide is presented in Chapter 5. Examples of the sensitivity of the pavement structural design to the various geotechnical factors included in the 1993 AASHTO Guide are the focus of Chapter 6.

1998 Guide Supplement

The 1998 supplement to the 1993 AASHTO Pavement Design Guide (AASHTO, 1998) provided an alternate method for rigid pavement design. The main changes from the procedures in the 1993 Guide included the following:

- The modulus of subgrade reaction *k* is now defined as the elastic value on the top of the subgrade (or embankment, if present). When measured in a plate loading test, only the elastic (*i.e.,* recoverable) deformation is now used to compute k, and all permanent deformation is neglected. This is in contrast to previous versions of the Guide which defined k as a gross value that included both the elastic and permanent deformations from plate loading tests. Recommended procedures in the 1998 Guide Supplement for determining k are (a) correlations with soil type and other soil properties or tests; (b) deflection testing and backcalculation (most highly recommended); and (c) plate bearing tests.

- The design k value is still modified for the influence of shallow bedrock, as in the 1993 Guide. A new modification is also included for the effects of embankments.

- The effective k value for design is no longer modified for the stiffness and thickness of the base[4] layer, as in the 1993 Guide. Instead, the base layer thickness and resilient modulus are included explicitly in the revised rigid pavement design equations.

- The drainage factor C_d is no longer included in the design equations.

- The loss of support factor LS is no longer included in the design procedure.

- Both load and temperature stresses are included in the design calculations.

A set of revised design equations for the alternate rigid pavement design method are provided in the 1998 supplement. The principal geotechnical parameters in these equations are: effective elastic modulus of subgrade support (k); modulus of elasticity of the base (E_b); and thickness of the base layer (H_b). The coefficient of friction between the slab and the base/subgrade is also required for reinforcement design in JRCP systems.

[4] The granular layer between the slab and the subgrade is termed the base layer in the 1998 supplement. In earlier versions of the AASHTO Design Guides, this layer was termed the subbase.

3.5.3 The NCHRP 1-37A Pavement Design Guide[5]

The various editions of the *AASHTO Guide for Design of Pavement Structures* have served well for several decades. These procedures are all based on performance data from the original AASHO Road Test (HRB, 1962). However, the range of conditions considered in the AASHO Road Test were quite limited, and these increasingly serious deficiencies limit the continued use of the AASHTO Design Guide as the nation's primary pavement design procedure:

- *Traffic loading*: Heavy truck traffic levels have increased tremendously. The original Interstate pavements were designed in the 1960s for 5 – 10 million equivalent single-axle loads, whereas today these same pavements must be designed for 50 – 200 million axle loads, and sometimes more. It is unrealistic to expect that the existing AASHTO Guide based on the data from the original AASHO Road Test can be used reliably to design for this level of traffic. The pavements in the AASHO Road Test sustained slightly over 1 million axle load applications—less than the traffic carried by many modern pavements within the first few years of their use. When applying these procedures to modern traffic streams, the designer must extrapolate the design methodology far beyond the original field data (Figure 3-19). Such highly-trafficked projects are likely either under-designed or over-designed to an unknown degree, with significant economic inefficiency in either case.

- *Rehabilitation limitations*: Pavement rehabilitation design procedures were not considered at the AASHO Road Test. The rehabilitation design recommendations in the 1993 Guide are completely empirical and very limited, especially under heavy traffic conditions. Improved capabilities for rehabilitation design are vital to today's highway designs, as most projects today involve rehabilitation rather than new construction.

- *Climatic conditions*: Because the AASHO Road Test was conducted at one geographic location, the effects of different climatic conditions can only be included in a very approximate manner in the AASHTO Design Guides. A significant amount of distress at the original AASHO Road Test occurred in the pavements during the spring thaw, a condition that does not exist in a large portion of the country. Direct consideration of site-specific climatic effects will lead to improved pavement performance and reliability.

[5] The official name for the NCHRP 1-37A project is the "2002 Guide for the Design of New and Rehabilitated Pavement Structures." However, since official AASHTO approval of this guide is still in process, it will be referred to in this report simply as the "NCHRP 1-37A Pavement Design Guide."

- *Subgrade types*: One type of subgrade–and a poor one at that (AASHTO A-6/A-7-6)–existed at the Road Test, but many other types exist nationally. The significant influence of subgrade support on the performance of highway pavements can only be included very approximately in the current AASHTO design procedures.

- *Surfacing materials*: Only a single asphalt concrete and Portland cement concrete mixture were used at the Road Test. The HMAC and PCC mixtures in common use today (*e.g.,* Superpave, stone-mastic asphalt, high-strength PCC) are significantly different and better than those at the Road Test, but the benefits from these improved materials cannot be fully considered in the existing AASHTO Guide procedures.

- *Base materials*: Only two unbound dense granular base/subbase materials were included in the main flexible and rigid pavement sections of the AASHO Road Test (limited testing of stabilized bases was included for flexible pavements). These exhibited significant loss of modulus due to frost and erosion. Today, various stabilized types are used routinely, especially for heavier traffic loadings.

- *Traffic*: Truck suspension, axle configurations, and tire types and pressures were representative of the types used in the late 1950s. Many of these are outmoded (tire pressures of 80 psi versus 115 psi today), and pavement design procedures based on the older, lower tire pressures may be deficient for today's higher values.

- *Construction and drainage*: Pavement designs, materials, and construction were representative of those used at the time of the Road Test. No subdrainage was included in the Road Test sections, but positive subdrainage has become common in today's highways.

- *Design life*: Because of the short duration of the Road Test, the long-term effects of climate and aging of materials were not addressed. The AASHO Road Test was conducted over 2 years, while the design lives for many of today's pavements are 20 to 50 years. Direct consideration of the cyclic effect on materials response and aging are necessary to improve design life reliability.

- *Performance deficiencies*: Earlier AASHTO procedures relate the thickness of the pavement surface layers (asphalt layers or concrete slab) to serviceability. However, research and observations have shown that many pavements need rehabilitation for reasons that are not related directly to pavement thickness (*e.g.,* rutting, thermal cracking, faulting). These failure modes are not considered directly in the current AASHTO Guide.

- *Reliability*: The 1986 AASHTO Guide included a procedure for considering design reliability that has never been fully validated. The reliability multiplier for design traffic increases rapidly with reliability level and may result in excessive layer thicknesses for heavily trafficked pavements that may not be warranted.

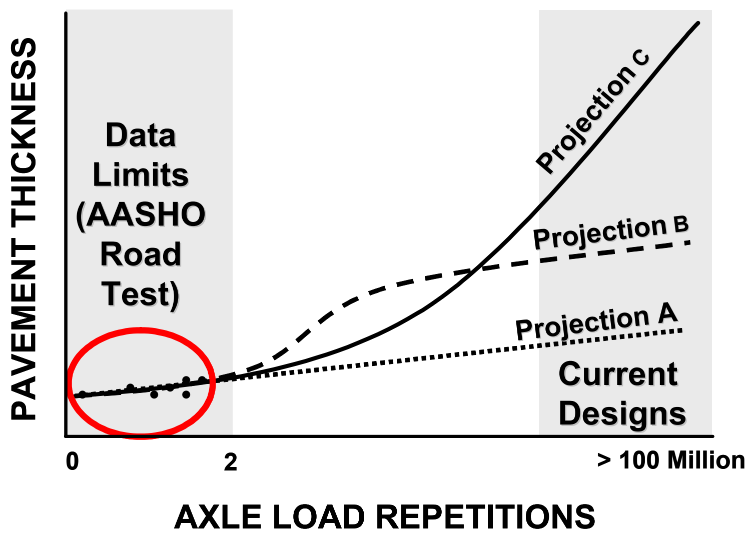


Figure 3-19. Extrapolation of traffic levels in current AASHTO pavement design procedures (NHI Course 131064).

The latest step forward in mechanistic-empirical design is the recently-completed NCHRP Project 1-37A *Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures* (NCHRP, 2004). NCHRP Project 1-37A was a multi-year effort to develop a new national pavement design guide based on mechanistic-empirical principles. A key distinction of the models developed under NCHRP Project 1-37A is their calibration and validation using data from the FHWA Long Term Pavement Performance Program national database in a well-balanced experiment design representing all regions of the country. The NCHRP 1-37A models also include flexibility for re-calibration and validation using local or regional databases, if desired, by individual agencies. The mechanistic-empirical design approach as implemented in the NCHRP 1-37A Pavement Design Guide will allow pavement designers to:

- evaluate the impact of new load levels and conditions,
- better utilize current and new materials,
- incorporate daily, seasonal, and yearly changes in materials, climate, and traffic,
- better characterize seasonal/drainage effects,
- improve rehabilitation design,

- predict/minimize specific failure modes,
- understand/minimize premature failures (forensics),
- extrapolate from limited field and laboratory data,
- reduce life cycle costs,
- rationalize cost allocation, and
- create more efficient, reliable, and cost-effective designs.

Of course, benefits do not come without a cost. There are some drawbacks to mechanistic-empirical design methodologies like those in the NCHRP 1-37A procedure:

- Substantially more input data are required for design. Detailed information is required for traffic data, project environmental conditions, and material properties.
- Most of the required material properties are fundamental engineering properties that should be measured via laboratory and field testing, as opposed to empirical properties that can be estimated qualitatively.
- The design calculations are no longer amenable to hand computation. Sophisticated software is generally required. The execution time for this software is generally longer than that required for the DarWIN software commonly used for the current AASHTO design procedures.
- Many agencies will need to upgrade their technical capabilities. This may include laboratory upgrades, new and faster computers, training for personnel, and changes in operational procedures.

An extended summary of the NCHRP 1-37A methodology is provided in Appendix D. A detailed discussion of the key geotechnical inputs in the NCHRP 1-37A Pavement Design Guide is presented in Chapter 5. Examples using the NCHRP 1-37A Design Guide, including comparisons with the current AASTHO Design Guide, are the focus of Chapter 6.

3.5.4 Low-Volume Roads

Pavement structural design for low-volume roads is divided into four categories:

1. Flexible pavements
2. Rigid pavements
3. Aggregate surfaced roads
4. Natural surface roads

The traffic levels on low-volume roads are significantly lower than those for which pavement structural design methods like the empirical 1993 AASHTO Guide and the mechanistic-empirical NCHRP 1-37A procedure are intended. Consequently, these methods are generally not applied directly to the design of low-volume roads. Instead, both the 1993 AASHTO and

NCHRP 1-37A Design Guides provide catalogs of typical flexible pavement, rigid pavement, and aggregate surfaced designs for low-volume roads as functions of traffic category, subgrade quality, and climate zone. The 1993 AASHTO Guide also provides a simple separate design procedure for aggregate surfaced roads. Refer to the 1993 AASHTO Design Guide for additional details.

Rutting is the primary distress for aggregate or natural surfaced roads. Vehicles traveling over aggregate or natural surfaced roads generate significant compressive and shear stresses that can cause failure of the soil. An acceptable rutting depth for aggregate surfaced roads can be estimated considering aggregate thickness and vehicle travel speed. A 2-inch (50 mm) rut depth in a 4-inch-thick (100 mm) aggregate layer probably will result in mixing of the soil subgrade with the aggregate, which will destroy the paving function of the aggregate. Rutting depths greater than 2 to 3 inches (50 to 75 mm) in either aggregate or natural surface roads can be expected to significantly reduce vehicle speeds.

Note that rutting may not be the only design consideration. Poor traction or dust conditions may dictate a hard surface. Traction characteristics may be indicated by the soil plasticity index, and dust potential may be indicated by the percent fines.

The depth of rutting in aggregate or natural surfaced roads will depend upon the soil support characteristics and magnitude and number of repetitions of vehicle loads. The most common measure of rutting susceptibility is the California Bearing Ratio (CBR – see Section 5.4.1). Both the CBR test and rutting involve penetration of the soil surface due to a vertical loading. Although the CBR test does not measure compressive or shear strength values, it has been empirically correlated to rut depth for a range of vehicle load magnitudes and repetitions. The U.S. Forest Service (USDA, 1996) uses the following relationship for designing aggregate thickness in aggregate surfaced roads:

$$\text{Rut Depth (inches)} = 5.833 \frac{\left(F_r R\right)^{0.2476}}{(\log t)^{.002} C_1^{0.9335} C_2^{0.2848}} \tag{3.10}$$

in which

- R = number of Equivalent Single Axle Loads (ESALs) at a tire pressure of 80 psi
- t = thickness of top layer (inches)
- C_1 = CBR of top layer
- C_2 = CBR of subgrade
- F_r = reliability factor applied to R—see Table 3-8

Equation (3.10) is based upon an algorithm developed by the U.S. Army Corps of Engineers (Barber *et al.*, 1978). Consult the U.S. Forest Service *Earth and Aggregate Surfacing Design Guide* (USDA, 1996) for more details on the design procedure.

The allowable ESALs R in Equation (3.10) will vary depending upon the pavement materials and tire pressure. ESAL equivalency factors are defined in terms of pavement damage or reduced serviceability. The Forest Service Design Guide suggests that the ESAL equivalency factor for a 34-kip tandem axle be between 0.09 and 2.15 for tire pressures varying between 25 – 100 psi (172 – 690 kPa). According to the AASHTO Design Guide, this same axle has equivalency factors of between 1.05 and 1.1 for flexible pavements (SN between 1 and 6) and between 1.8 and 2 for rigid pavements (slab thickness D between 6 and 14 inches). Rut depth can be managed by limiting tire pressures. Rut depth can decrease by more than 50% for aggregate surfaced roads if the tire pressure for a 34-kip tandem axle is reduced from 100 to 25 psi (690 to 172 kPa). The Forest Service has partnered with industry to develop equipment that will centrally adjust tire pressures of log-hauling vehicles.

Equation (3.10) can also be used to estimate rut depth for naturally surfaced roads. The upper layer of soil is expected to be compacted by traffic. Values must therefore be assigned to the compacted surface CBR (C_1), the underlying soil CBR (C_2), and the compacted thickness (t). Values of C_1 at 90% relative compaction, C_2 at 85% relative compaction, and t = 6 inches (150 mm) are reasonable values for typical conditions.

The South Dakota Gravel Roads Maintenance and Design Manual (Skorseth and Selim, 2000) discusses two additional design approaches for aggregate surfaced roads. One approach consists of design catalogs based on traffic categories, soil support classes, and climatic region. The more analytical approach considers ESALs, subgrade resilient modulus, seasonal variations of subgrade stiffness, the elastic moduli of the other pavement materials, allowable serviceability loss, allowable rutting depth, and allowable aggregate loss. The loss of pavement thickness due to traffic is unique to aggregate surfacing and must be considered by all thickness design methods for these types of roads. The hardness and durability of the aggregate may also require evaluation.

For low-volume road surface layers that are stiffer than aggregate – *e.g.,* hot mix asphalt and concrete – the recoverable strain within the subgrade can be used to calculate deflections in the soil that can cause fatigue damage in the material above. The use of unconfined compressive strength or unconsolidated-undrained shear strength is a reasonable approach for identifying pavement sections that have a potential for subgrade rutting. Intuitively, if the computed stresses within the pavement section are substantially less that the measured strength, rutting is less likely. It has been proposed that the unconfined compressive strength (psi) is equal to approximately 4.5 times the CBR value (IDOT, 1995).

Table 3-8. Reliability factors for use in Equation (3.10).

Reliability Level (%)	Reliability Factor F_r
50	1.00
70	1.44
90	2.32

3.6 EXERCISE

The Main Highway project is described in Appendix B. Working in groups, participants should read through this description and summarize in order of importance the key geotechnical issues that will influence the pavement design for this project. Each group will list its key geotechnical issues on the blackboard/flip chart, and all groups will then discuss the commonalities and discrepancies between the individual groups' assessments.

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CHAPTER 4.0 GEOTECHNICAL EXPLORATION AND TESTING

4.1 INTRODUCTION

The purpose of the geotechnical subsurface investigation program for pavement design and construction is to obtain a thorough understanding of the subgrade conditions along the alignment that will constitute the foundation for support of the pavement structure. The specific emphasis of the subsurface investigation is to identify the impact of the subgrade conditions on the construction and performance of the pavement, characterize material from cut sections that may be used as subgrade fill, and to obtain design input parameters. The investigation may be accomplished through a variety of techniques, which may vary with geology, design methodology and associated design requirements, type of project and local experience. To assist agencies in achieving the stated purpose of subsurface investigation, this chapter presents the latest methodologies in the planning and execution of the various exploratory investigation methods for pavement projects. It is understood that the procedures discussed in this chapter are subject to local variations. Users are also referred to AASHTO R 13 and ASTM D 420, *Conducting Geotechnical Subsurface Investigations* and FHWA NHI-01-031 *Subsurface Investigations*, for additional guidance.

In Chapter 1, a simplistic subsurface exploration program consisting of uniformly spaced soil borings (*i.e.,* systematic sampling) with SPT testing was mentioned as an antiquated method for determining the subsurface characteristics for pavement design. "Adequate for design and low cost" are often used in defense of this procedure. The cost-benefit of additional subsurface exploration is a subject that is often debated. This subject is now addressed in the new NCHRP 1-37A Design Guide. The guide allows for use of default values in the absence of sufficient data for characterizing the foundation, thus minimizing agency design costs, but at the increased risk of over- or under-designing the pavement structure.

In evaluating the cost-benefit of the level of subsurface investigation, all designers must recognize that the reliability and quality of the design will be directly related to the subsurface information obtained. The subsurface exploration program indeed controls the quality of the roadway system. A recent FHWA study indicated that a majority of all construction claims were related to inadequate subsurface information. With great certainty, inadequate information will lead to long-term problems with the roadway design. The cost of a subsurface exploration program is a few thousand dollars, while the cost of over-conservative designs or costly failures in terms of construction delays, construction extras, shortened design life, increased maintenance, and public inconvenience is typically in the hundreds of thousands of dollars.

Engineers should also consider that the actual amount of subgrade soil sampled and tested is typically on the order of one-millionth to one-billionth of the soil being investigated. Compare this with sampling and testing of other civil engineering materials. Sampling and testing of concrete is on the order of 1 sample (3 test specimens, or about ¼ cubic meter) every 40 cubic meters, which leads to 1 test in 100,000. Sampling and testing of asphalt is on the same order as concrete. Now consider that the variability in properties of these well-controlled, manufactured materials is much less than the properties of the subgrade, which often have coefficients of variation of well over 100% along the alignment. Again cost, not quality is usually the deciding factor. The quality of sampling can be overcome with conservative designs (as is often the case; *e.g.,* AASHTO 1972). For example, laboratory tests are often run on soil samples in a weaker condition than in the ground, rather than running more tests on the full range of conditions that exist in the field. While this approach may provide a conservative value for design purposes, there are hidden costs in both conservatism and questionable reliability. Modern pavement design uses averages with reliability factors to account for uncertainty (AASHTO, 1993 and NCHRP 1-37A). However, sufficient sampling and testing are required to check the variability of design parameters to make sure that they are within the bounds of reliability factors; otherwise, on highly variable sites designs, they will not be conservative and on very uniform sites, they will still be over conservative.

The expense of conducting soil borings is certainly a detriment to obtaining subsurface information. However, exploration itself is not just doing borings. There is usually a significant amount of information available from alternate methods that can be performed prior to drilling to assist in optimizing boring and sampling locations (*i.e.,* representative sampling). This is especially the case for reconstruction and rehabilitation projects. Significant gains in reliability can be made by investigating subgrade spatial variability in a pavement project and often at a cost reduction due to decreased reliance on samples. This chapter provides guidelines for a well-planned exploration program for pavement design, with alternate methods used to overcome sampling and testing deficiencies. Geotechnical exploration requirements for borrow materials (base, subbase, and subgrades) are also reviewed.

Figure 4-1 provides a flow chart of the process for performing a geotechnical exploration and testing program. As shown in the flow chart, the steps for planning and performing a complete geotechnical and testing program include

Subsurface Exploration Steps	Relevance to Pavement Design
1) Establish the type of pavement construction.	Whether new construction, reconstruction, or rehabilitation.
2) Search available information.	To identify anticipated subsurface conditions at the vertical and horizontal location of the pavement section.
3) Perform site reconnaissance.	To identify site conditions requiring special consideration.
4) Plan the exploration program for evaluation of the subsurface conditions and identification of the groundwater table, including methods to be used with consideration for using • remote sensing, • geophysical investigations, • in-situ testing, • disturbed sampling, and • undisturbed sampling.	To identify and obtain - more information on site conditions, - spacial distribution of subsurface conditions, - rapid evaluation of subsurface condition, - subgrade soils & classification test samples, - samples for resilient modulus tests and calibration of in-situ results.
5) Evaluate conceptual designs, examine subsurface drainage and determine sources for other geotechnical components (*e.g.,* base and subbase materials).	Identify requirements for subsurface drainage and subgrade stabilization requirements, as well as construction material properties.
6) Examine the boring logs, classification tests, soil profiles and plan view, then select representative soil layers for laboratory testing.	Use the soil profile and plan view along the roadway alignment to determine resilient modulus or other design testing requirements for each influential soil strata encountered.

Each of these steps will be reviewed in the following sections of this chapter.

4.2 LEVELS OF GEOTECHNICAL EXPLORATION FOR DIFFERENT TYPES OF PAVEMENT PROJECTS

There are three primary types of pavement construction projects. They are

- new construction,
- reconstruction, and
- rehabilitation.

Each of these pavement project types requires different considerations and a corresponding level of effort in the geotechnical exploration program.

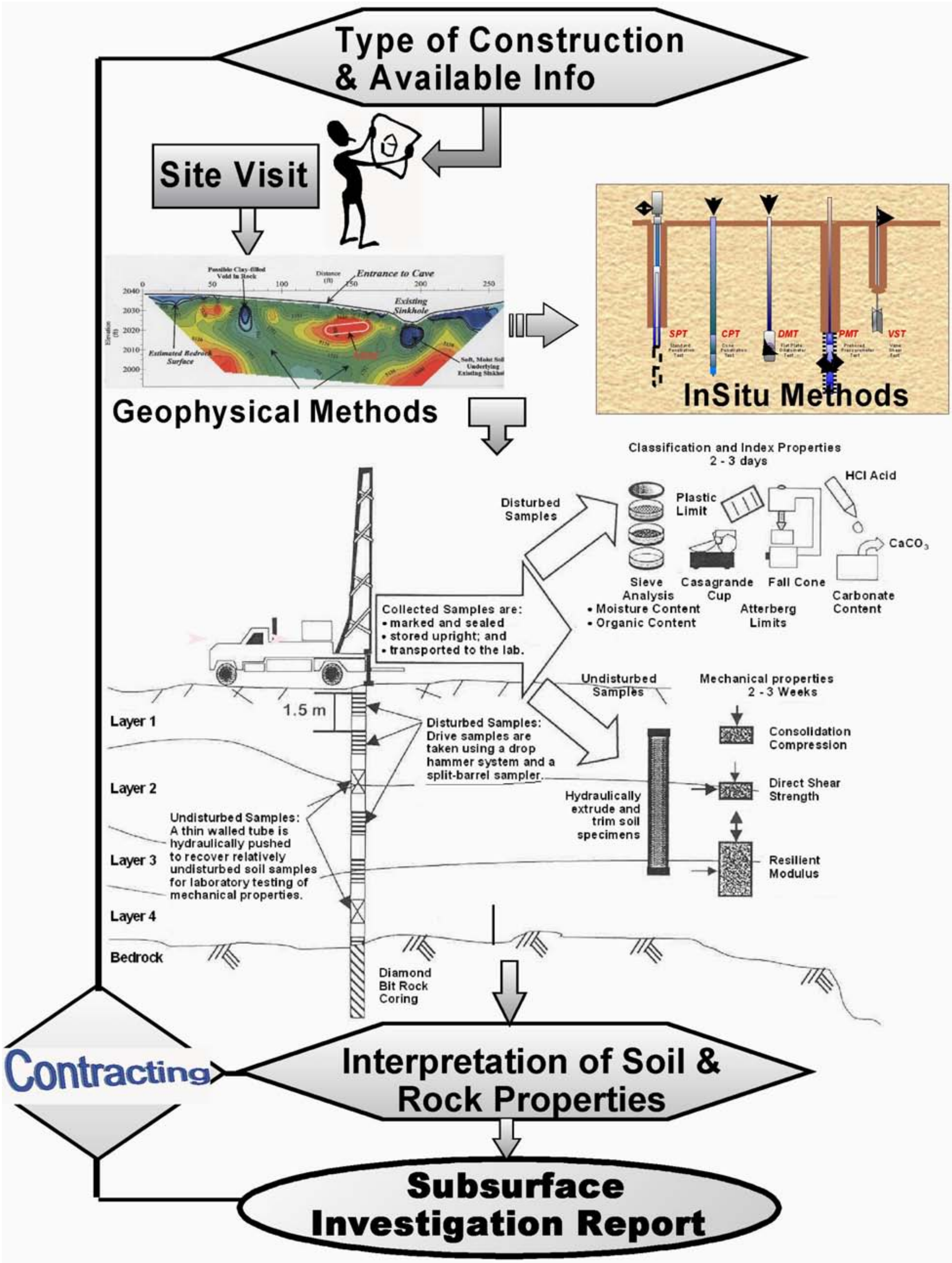


Figure 4-1. Geotechnical exploration and testing for pavement design.

4.2.1 New Pavement Construction

For new construction, the exploration program will require a complete evaluation of the subgrade, subbase, and base materials. Sources of materials will need to be identified and a complete subsurface exploration program will need to be performed to evaluate pavement support conditions. Prior to planning and initiating the investigation, the person responsible for planning the subsurface exploration program (*i.e.,* the geotechnical engineer or engineer with geotechnical training) needs to obtain from the designers the type, load, and performance criteria, location, geometry and elevations of the proposed pavement sections. The locations and dimensions of cuts and fills, embankments, retaining structures, and substructure elements (*e.g.,* utilities, culverts, storm water detention ponds, etc.) should be identified as accurately as practicable.

Also, for all new construction projects, samples from the subgrade soils immediately beneath the pavement section and from proposed cut soils to be used as subgrade fill will be required to obtain the design-input parameters for the specific design method used by the agency. Available site information (*e.g.,* geological maps and United States Department of Agriculture Natural Resources Conservation Service's soil survey reports) as discussed in Section 4.3, site reconnaissance (see Section 4.4), air photos (see Section 4.5.3) and geophysical tests (see Section 4.5.4) can all prove beneficial in identify representative and critical sampling locations.

For all designs using AASHTO 1993 or NCHRP 1-37A, particularly for critical projects, repeated load resilient modulus tests are needed to evaluate the support characteristics and the effects of moisture changes on the resilient modulus of each supporting layer. The procedures, sample preparation and interpretation of the resilient modulus test are discussed in Chapter 5. For designs based on subgrade strength, either lab tests (e.g., CBR) as discussed in Chapter 5 or in-situ tests (e.g., DCP) as discussed later in Section 4.5.5 of this chapter can be used to determine the support characteristics of the subgrade.

Another key part of subsurface exploration is the identification and classification (through laboratory tests) of the subgrade soils in order to evaluate the vertical and horizontal variability of the subgrade and select appropriate representative design tests. Field identification along with classification through laboratory testing also provides information to determine stabilization requirements to improve the subgrade should additional support be required, as discussed in Chapter 7.

Location of the groundwater table is also an important aspect of the subsurface exploration program for new construction to evaluate water control issues (*e.g.,* subgrade drainage requirements) with respect to both design and construction. Methods for locating the groundwater level are discussed in Section 4.5.6. Other construction issues include the identification of rock in the construction zone, rock rippability, and identification of soft or otherwise unsuitable materials to be removed from the subgrade. The location and rippability of rock can be determined by geophysical methods (*e.g.,* seismic refraction), as discussed in section 4.5.4 and/or borings and rock core samples.

4.2.2 Reconstruction

For pavement reconstruction projects, such as roadway replacement, full depth reclamation, or road widening, information may already exist on the subgrade support conditions from historical subsurface investigations. Existing borings should be carefully evaluated with respect to design elevation of the new facility. A survey of the type, severity, and amount of visible distress on the surface of the existing pavement (*i.e.,* a condition survey as described in the NHI, 1998, "Techniques for Pavement Rehabilitation" Participant's Manual) can also indicate local issues that need a more extensive evaluation. However, an additional limited subsurface investigation is usually advisable to validate the pavement design calculations and design for weak subgrade conditions, if present. It is also likely that resilient modulus, CBR or other design input values used by agencies would need to be obtained for the existing materials using current procedures. Test methods used by the agency often change over time (*e.g.,* lab CBR versus field CBR). Previous data may also not be valid for current conditions (*e.g.,* traffic). Water in old pavements can often result in poorer subgrade conditions than originally encountered. Drainage features, or lack thereof, in the existing pavement and their functionality should be examined. Again, subgrade soil identification and classification will be required to provide information on subgrade variability and assist in selection of soils to be tested.

It is possible to determine the value of reworking the subgrade (*i.e.,* scarifying, drying, and recompacting) if results indicate stiffness and/or subgrade strength values are below expected or typical values. This comparison can be made by examining the resilient modulus of undisturbed tube samples obtained to verify backcalculated moduli to that of a recompacted specimen remolded to some prescribed level of density and moisture content. For example, this comparison may ultimately lead to the need for underdrain installation in order to reduce and maintain lower moisture levels in the subgrade.

Subsurface investigation on reconstruction projects can usually be facilitated by using non-destructive tests (NDT) (a.k.a. geophysical methods) performed over the old pavement (or shoulder section for road widening) with one or more of the variety of methods presented in Section 4.5. For example, resilient modulus properties can best be obtained from non-destructive geophysical methods, such as falling weight deflectometer (FWD) tests and back calculating elastic moduli to characterize the existing structure and foundation soils needed for design. This approach is suggested because it provides data on the response characteristics of the in-situ soils and conditions. Back calculation of layer elastic moduli from deflection basin data is discussed later in Section 4.5.4 of this chapter. These results can be supported by laboratory tests on samples obtained from a minimal subsurface exploration program (described in Section 4.5). Old pavement layer thickness (i.e., asphalt or concrete, base and/or subbase) should also be obtained during sampling to provide information for back-calculation of the modulus values.

For designs based on subgrade strength (*e.g.,* CBR), in-situ tests (*e.g.,* Dynamic Cone Penetrometer (DCP), field CBR, and other methods as described in Section 4.7) can be performed to obtain a rapid assessment of the variability in subgrade strength and to determine design strength values via correlations. Some samples should still be taken to perform laboratory tests and confirm in-situ test correlation values. Geophysical test results (*e.g.,* FWD, Ground Penetrating Radar (GPR), and others described in Section 4.5.4) can also be used to assist in locating borings.

The potential sources of new base and subbase materials will need be identified and laboratory tests performed to obtain resilient modulus, CBR or other design values, unless catalogued values exist for these engineered materials. For pavement reclamation or recycling projects, composite samples should be obtained from the field and test specimens constituted following the procedures outlined in Chapter 5 to obtain design input values. The subgrade soils will also need to be evaluated for their ability to support construction activities, such as rubblize-and-roll type construction.

4.2.3 Rehabilitation

As discussed in Chapter 3, rehabilitation projects include a number of strategies, including overlays, rubbilization, and crack and seat. The details required for the subsurface investigation of pavement rehabilitation projects depends on a number of variables:

- The condition of the pavement to be rehabilitated (*e.g.,* pavement rutting, cracking, riding surface uniformity and roughness, surface distress, surface deflection under traffic, presence of water, etc., as described in the condition survey section of NHI, 1998, "Techniques for Pavement Rehabilitation" Participants Manual.)
- If the facility is distressed, the type, severity and extent of distress (pavement distress, pavement failures, crack-type pattern, deep-seated failures, settlement, drainage and water flow, and collapse condition) (see NHI, 1998, "Techniques for Pavement Rehabilitation" Participants Manual) should be quantified. Rutting and fatigue

cracking are often associated with subgrade issues and general require coring, drilling, and sampling to diagnose the cause of these conditions.

- Techniques to be considered for rehabilitation.
- Whether the facility will be returned to its original and as-built condition, or whether it will be upgraded, for example, by adding another lane to a pavement. If facilities will be upgraded, the proposed geometry, location, new loads and structure changes (*e.g.,* added culverts) must be considered in the investigation.
- The required performance period of the rehabilitated pavement section.

Selection of the rehabilitation alternative will partly depend on the condition assessment. NHI, 1998, "Techniques for Pavement Rehabilitation" Participants Manual covers condition surveys and selection of techniques for pavement rehabilitation. Information from the subsurface program performed for the original pavement design should also be reviewed. However, as with reconstruction projects, some additional corings and borings will need to be performed to evaluate the condition and properties of the of the pavement surface and subgrade support materials. Pavements are frequently cored at 150 – 300 m (500 – 1000 ft) intervals for rehabilitation projects. The core holes in the pavement also provide access to investigate the in-situ and disturbed properties of the base, subbase, and subgrade materials. Samples can be taken and/or in-situ tests (*e.g.,* DCP) can be used to indicate structural properties, as well as layer thickness.

Geophysical tests (*e.g.,* FWD, GPR, and others described in Section 4.5.4) can be used to assist in locating coring and boring locations, especially if the base is highly contaminated or there are indications of subgrade problems. Otherwise, the frequency of corings and borings should be increased. As with reconstruction projects, rehabilitation projects can use FWD methods and associated back-calculated elastic modulus to characterize the existing structure and foundation. Again, the FWD method is covered in Section 4.5.4 and back-calculation of layer elastic moduli from deflection basin data is discussed in Chapter 5. FWD results can also be correlated with strength design values (*e.g.,* CBR). A limited subsurface drilling and sampling program can then be used to confirm the back-calculated resilient modulus values and/or correlation with other strength design parameters. The layer thickness of each pavement component (*i.e.,* surface layer, base, and or subbase layer) is critical for back-calculation of modulus values.

4.2.4 Subsurface Exploration Program Objectives

As stated in the NCHRP 1-37A Design Guide, the objective of subsurface investigation 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 design and construction.

As outlined in the FHWA Soils and Foundation Workshop manual (FHWA NHI-00-045), the subsurface exploration program should obtain sufficient subsurface information and samples necessary to define soil and rock subsurface conditions as follows:

1) Statigraphy (for evaluating the areal extent of subgrade features)
   a) Physical description and extent of each stratum
   b) Thickness and elevation of various locations of top and bottom of each stratum
2) For cohesive soils (identify soils in each stratum, as described in Section 4.7, to assess the relative value for pavement support and anticipated construction issues, *e.g.,* stabilization requirements)
   a) Natural moisture contents
   b) Atterberg limits
   c) Presence of organic materials
   d) Evidence of desiccation or previous soil disturbance, shearing, or slickensides
   e) Swelling characteristics
   f) Shear strength
   g) Compressibility
3) For granular soils (identify soils in each stratum, as described in Section 4.7, to assess the relative value for pavement support and use in the pavement structure)
   a) In-situ density (average and range)
   b) Grain-size distribution (gradations)
   c) Presence of organic materials
4) Groundwater (for each aquifer within zone of influence on construction and pavement support, especially in cut sections as detailed in Section 4.5)
   a) Piezometric surface over site area, existing, past, and probable range in future
   b) Perched water table
5) Bedrock (and presence of boulders) (within the zone of influence on construction and pavement support as detailed in Section 4.5)
   a) Depth over entire site
   b) Type of rock
   c) Extent and character of weathering
   d) Joints, including distribution, spacing, whether open or closed, and joint infilling
   e) Faults
   f) Solution effects in limestone or other soluble rocks
   g) Core recovery and soundness (RQD)
   h) Ripability

4.3 SEARCH AVAILABLE INFORMATION

The next step in the investigation process is to collect and analyze all existing data. A complete and thorough investigation of the topographic and subsurface conditions must be made prior to planning the field exploration program so that it is clear where the pavement subgrade will begin and to identify the type of soils anticipated within the zone of influence of the pavement. The extent of the site investigation and the type of exploration required will depend on this information. ("If you do not know what you should be looking for in a site investigation, you are not likely to find much of value." Quote from noted speaker at the 8th Rankine Lecture). Simply locating borings without this information is like sticking a needle in your arm blindfolded and hoping to hit the vein. A little sleuthing can greatly assist in gaining an understanding of the site and planning the appropriate exploration program.

An extensive amount of information can be obtained from a review of literature about the site. There are a number of very helpful sources of data that can and should be used in planning subsurface investigations. Review of this information can often minimize surprises in the field, assist in determining boring locations and depths, and provide very valuable geologic and historical information, which may have to be included in the exploration report.

The first information to obtain is prior agency subsurface investigations (historical data) at or near the project site, especially for rehabilitation and reconstruction projects. To determine its value, this data should be carefully evaluated with respect to location, elevation, and site variability. Also, in review of data, be aware that test methods change over time. For example, SPT values 20 to 30 years ago were much less efficient than today, as discussed in Section 4.5.5. Prior construction and records of structural performance problems at the site (*e.g.,* excessive seepage, unpredicted settlement, and other information) should also be reviewed. Some of this information may only be available in anecdotal forms. For rehabilitation and reconstruction projects, contact agency maintenance personnel and discuss their observations and work along the project alignment. The more serious construction and/or maintenance problems should be investigated, documented if possible, and evaluated by the engineer.

In this initial stage of site exploration, for new pavement projects, the major geologic processes that have affected the project site should also be identified. Geology will be a key factor to allow the organization and interpretation of findings. For example, if the pavement alignment is through an ancient lakebed, only a few representative borings will be required to evaluate the pavement subgrade. However, in highly variable geologic conditions, additional borings (*i.e.,* in excess of the normal minimum) should be anticipated. Geological information is especially beneficial in pavement design and construction to identify the presence and types of shallow rock, rock outcrops, and rock excavation requirements. Geological information can readily be obtained from U.S. Geological Survey (USGS) maps, reports, publications and websites (www.usgs.gov), and State Geological Survey maps and publications.

Soils deposited by a particular process assume characteristic topographic features, called landforms, which can be readily identified by a geotechnical specialist or geologist. A landform contains soils with generally similar engineering properties and typically extends irregularly over wide areas of a project alignment. The soil may be further described as a residual or transported soil. A residual soil has been formed at a location by the in-place decomposition of the parent material (sedimentary, igneous, or metamorphic rock). Residual soils often contain a structure and lose strength when disturbed. A transported soil was formed at one location and has been transported by exterior forces (*e.g.,* water, wind, or glaciers). Alluvium soils are transported by water, loess type soils are transported by wind, and tills are transported by ice. Transported soils (especially alluvium and loess) are often fine grained and are usually characterized as poorly draining, compressible when saturated, and frost susceptible (*i.e.,* not the most desirable soils for supporting pavement systems). Sources of information for determining landform boundaries and their functional uses are given in Table 4-1.

One of the more valuable sources of landform information for pavement design and construction are soil survey maps produced by the U.S. Department of Agriculture, Natural Resources Conservation Service, in cooperation with state agricultural experiment stations and other Federal and State agencies. The county soil maps provide an overview of the spatial variability of the soil series within a county. These are well-researched maps and provide detailed information for shallow surficial deposits, especially valuable for pavements at or near original surface grade. They may also show frost penetration depths, drainage characteristics, and USCS soil types. Knowledge of the regional geomorphology (*i.e.,* the origin of landforms and types of soils in the region and the pedologic soil series definitions) is required to take full advantage of these maps. Such information will be of help in planning soil exploration activities. Plotting the pavement alignment on a USDA map and/or a USGS map can be extremely helpful. Figure 4-2 shows an example for a section of the Main Highway project.

The majority of the above information can be obtained from commercial sources (*i.e.,* duplicating services) or U.S. and state government, local or regional offices. Specific sources (toll-free phone numbers, addresses, etc.) for flood and geologic maps, aerial photographs, USDA soil surveys, can be very quickly identified through the Internet (*e.g.,* at the websites listed in Table 4-1).

Table 4-1. Sources of topographic & geologic data for identifying landform boundaries.

Source	Functional Use
Topographic maps prepared by the United States Coast and Geodetic Survey (USCGS).	Determine depth of borings required to evaluate pavement subgrade; determine access for exploration equipment; identify physical features, and find landform boundaries.
County agricultural soil maps and reports prepared by the U.S. Department of Agriculture's Natural Resources Conservation Service (a list of published soil surveys is issued annually, some of which are available on the web at http://soils.usda.gov/survey/online_surveys/).	Provide an overview of the spatial variability of the soil series within a county.
U.S. Geological Survey (USGS) maps, reports, publications and websites (www.usgs.gov), and State Geological Survey maps and publications.	Type, depth, and orientation of rock formations that may influence pavement design and construction.
State flood zone maps prepared by state or U.S. Geological Survey or the Federal Emergency Management Agency (FEMA: www.fema.gov) can be obtained from local or regional offices of these agencies.	Indicate deposition and extent of alluvial soils, natural flow of groundwater, and potential high groundwater levels (as well as danger to crews in rain events).
Groundwater resource or water supply bulletins (USGS or State agency).	Estimate general soils data shown, and indicate anticipated location of groundwater with respect to pavement grade elevation.
Air photos prepared by the United States Geologic Survey (USGS) and others (*e.g.,* state agencies).	Detailed physical relief shown; flag major problems. By studying older maps, reworked landforms from development activities can be identified along the alignment, *e.g.,* buried streambed or old landfill.
Construction plans for nearby structures (public agency).	Foundation type and old borings shown.

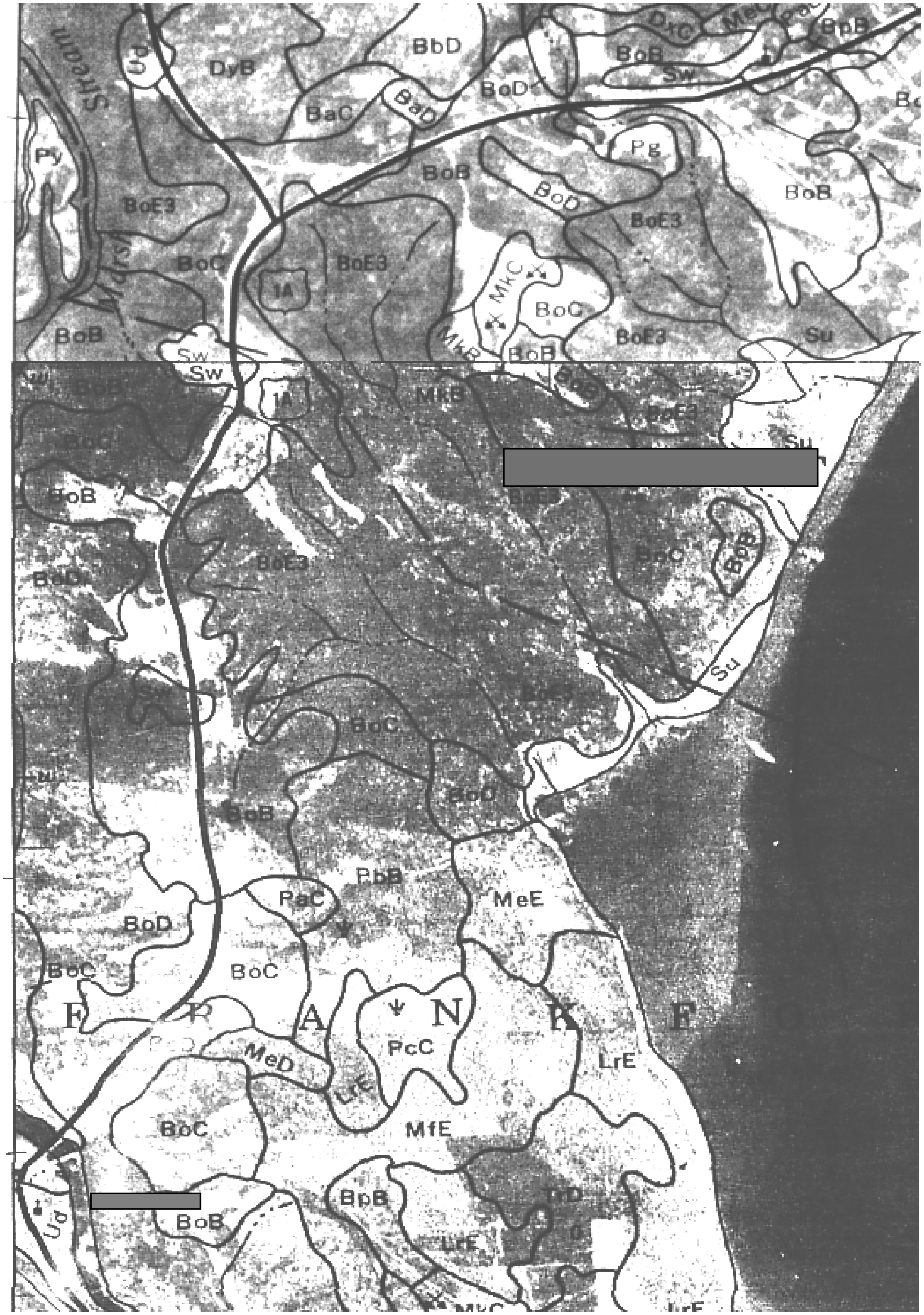


Figure 4-2. Soil Survey information along the Main Highway pavement alignment.

4.4 PERFORM SITE RECONNAISSANCE

A very important step in planning the subsurface exploration program is to visit the site with the project plans (*i.e.,* a plan-in-hand site visit). It is imperative that the engineer responsible for exploration, and, if possible, the project design engineer, conduct a reconnaissance visit to the project site to develop an appreciation of the geotechnical, topographic, and geological features of the site and become knowledgeable of access and working conditions. A plan-in-hand site visit is a good opportunity to learn about

- design and construction plans.
- general site conditions including special issues and local features, such as lakes and streams, exploration and construction equipment accessibility.
- surficial geologic and geomorphologic reconnaissance for mapping stratigraphic exposures and outcrops and identifying problematic surficial features, such as organic deposits.
- type and condition of existing pavements at or in the vicinity of the project.
- traffic control requirements during field investigations (a key factor in the type of exploration, especially for reconstruction and rehabilitation projects).
- location of underground and overhead utilities for locating in-situ tests and borings. (For pavement rehab projects, the presence of underground utilities may also support the use of non-destructive geophysical methods to assist in identifying old utility locations.
- adjacent land use (schools, churches, research facilities, etc.).
- restrictions on working hours (*e.g.,* noise issues), which may affect the type of exploration, as well as the type of construction.
- right-of-way constraints, which may limit boring locations.
- environmental issues (*e.g.,* old service stations for road widening projects).
- escarpments, outcrops, erosion features, and surface settlement.
- flood levels (as they relate to the elevation of the pavement and potential drainage issues.
- benchmarks and other reference points to aid in the location of borehole.
- subsurface soil and rock conditions from exposed cuts in adjacent works.

For reconstruction or rehabilitation projects, the site reconnaissance should include a condition survey of the existing pavement as detailed in NHI (1998) "Techniques for Pavement Rehabilitation." During this initial inspection of the project, the design engineer, preferably accompanied by the maintenance engineer, should determine the scope of the primary field survey, begin to assess the potential distress mechanisms, and identify the candidate rehabilitation alternatives. As part of this activity, subjective information on distress, road roughness, surface friction, and moisture/drainage problems should be gathered. Unless traffic volume is a hazard, this type of data can be collected without any traffic control, through both "windshield" and road shoulder observations. In addition, an initial assessment of traffic control options (both during the primary field survey and during rehabilitation construction), obstructions, and safety aspects should be made during this visit.

4.5 PLAN AND PERFORM THE SUBSURFACE EXPLORATION PROGRAM

Following the collection and evaluation of available information from the above sources, the geotechnical engineer (or engineer or geologist with geotechnical training) is ready to plan the field exploration program. The field exploration methods, sampling requirements, and types and frequency of field tests to be performed will be determined based on the existing subsurface information obtained from the available literature and site reconnaissance, project design requirements, the availability of equipment, and local practice. A geologist can often provide valuable input regarding the type, age, and depositional environment of the geologic formations present at the site for use in planning and interpreting the site conditions.

The subsurface investigation for any pavement project should be sufficiently detailed to define the depth, thickness, and area of all major soil and rock strata that will affect construction and long-term performance of the pavement structure. The extent of the exploration program depends on the nature of both the project and the site-specific subsurface conditions. 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.

4.5.1 Depth of Influence

Planning the subsurface exploration program requires a basic understanding of the depth to which subsurface conditions will influence the design, construction, and performance of the pavement system. For pavement design, the depth of influence is usually assumed to relate only to the magnitude and distribution of the traffic loads imposed on the pavement structure under consideration. Current AASHTO (1993) describes this depth at 1.5 m (5 ft) below the proposed subgrade elevation with this depth increased for special circumstances (*e.g.,* deep deposits of very soft soils). In this section, support for the recommended depth is provided, and special circumstances where this depth should be extended are reviewed.

The zone of influence under the completed pavement varies with the pavement section, but typically 80 – 90 percent of the applied stress is dissipated within 1 m (3 ft) below the asphalt section as shown in Figure 4-3. However, consideration must also be given to the roadway section (*i.e.,* height and width of the roadway embankment for fill application), the nature of the subsurface conditions, and consideration for construction (*e.g.,* depth of soils that may require stabilization to allow for construction). A common rule of thumb in geotechnical engineering is that the depth of influence is on the order of two times the width of the load. This adage is also true for pavement sections during construction and for unpaved roads. Considering a dual wheel is about 1 m (3 ft) in width, subsurface investigations for shallow cut and fill with no special problems should generally extend to 1.5 – 2 m (5 – 7 ft) below the proposed subgrade level to account for construction conditions. Special problems requiring deeper exploration may include deep highly compressible deposits (*e.g.,* peat or marsh areas) or deep deposits of frost-susceptible soils in cold regions. Greater depths may also be required for embankment design.

From a pavement design perspective, the critical layers are in the upper meter of the subgrade. This understanding is especially critical for rehabilitation projects. Mechanistic design is based upon the critical horizontal tensile strain at the base of an asphalt layer or the critical vertical compressive strain at the surface of the subgrade (and within the other pavement layers) under repetitions of a specific wheel or axle load (Huang, 1993). Subgrade strain often controls the pavement design except for very thick asphalt layers or overlays. For rehabilitation projects and in consideration of sampling for roadway design, the depth of influence should be evaluated based on the type of pavement and the reconstruction layering. The subsurface investigations should focus on these depths (typically the upper 1 to 2 meters). However, as discussed in Chapter 3, groundwater and bedrock at depths of less than 3 m (10 ft) beneath the pavement can have an influence on pavement design. In addition, location of the groundwater level within 3 m (10 ft) of the pavement will influence decisions of frost susceptibility, as discussed later in Chapter 7, and the presence of bedrock within 6 – 9 m (20 – 30 ft) can influence deflections of pavement layers and FWD results. Therefore, in order to confirm that there are no adverse deeper deposits, to identify groundwater conditions, and to locate bedrock within the influence zone, a limited amount of exploration should always be performed to identify conditions in the subgrade to depths of 6 m (20 ft). However, as discussed in the next section, this does not necessarily mean borings to that depth.

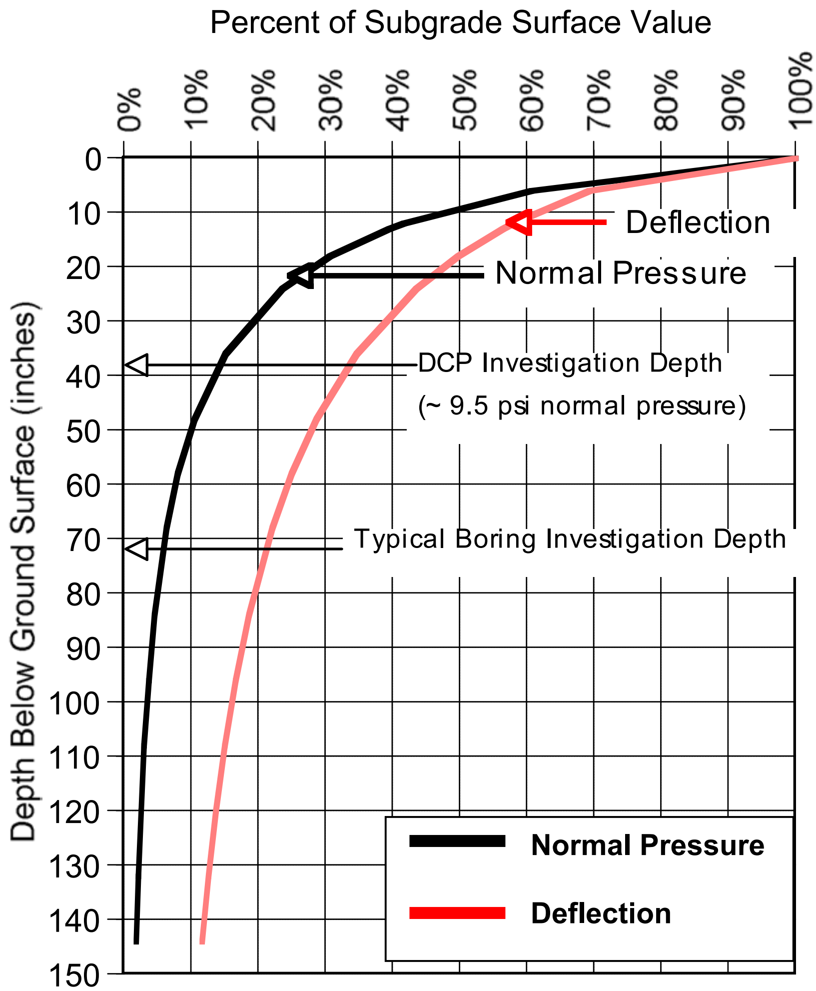


Figure 4-3. Typical zone of influence for an asphalt pavement section (Vandre et al., 1998).

4.5.2 Subsurface Exploration Techniques

Generally, there are four types of field subsurface investigation methods, best conducted in this order:

1. Remote sensing
2. Geophysical investigations
3. In-situ investigation
4. Borings and sampling

All of these methods are applicable for pavement design. For example, in new pavement construction projects, the location of old streambeds, usually containing soft, organic deposits that will require removal or stabilization, can usually be identified by remote sensing. Once identified, the vertical and horizontal extent of the streambed deposit can be explored by using geophysics to determine the horizontal extent, followed by in-situ tests to quantify the vertical extent and qualitatively evaluate soil properties, and borings with samples to quantify soil properties. The extent of use for a specific exploration method will be dependent upon the type of pavement project (*i.e.,* new construction, reconstruction, or rehabilitation), as discussed in the following subsections.

4.5.3 Remote Sensing

Remote sensing data from satellite and aircraft imagery can effectively be used to identify terrain conditions, geologic formations, escarpments and surface reflection of faults, buried stream beds, site access conditions, and general soil and rock formations that may impact new pavement design and construction. Infrared imagery can also be used to identify locally wet areas.

While remote sensing methods are most valuable for new construction, this information may also be used to explain poor performance of existing pavements in rehabilitation and reconstruction projects.

Remote sensing data from satellites (*e.g.,* LANDSAT images from NASA), aerial photographs from the USGS or state geologists, U.S. Corps of Engineers, commercial aerial mapping service organizations) can be easily obtained. State DOTs also use aerial photographs for right-of-way surveys and road and bridge alignments, and they can make them available for use by the engineer responsible for exploration. Especially valuable are old air photos compared to new ones in developed areas, which often identify buried features, such as old streambeds. Some ground control (*e.g.,* borings) is generally required to verify the information derived from remote sensing data.

4.5.4 Geophysical Investigations

Geophysical survey methods can be used to selectively identify boring locations, supplement borehole data, and interpolate between borings. There are several kinds of geophysical tests that can be used for stratigraphic profiling and delineation of subsurface geometries. These include the measurement of mechanical waves (deflection response, seismic refraction surveys, crosshole, downhole, and spectral analysis of surface wave tests), as well as electromagnetic techniques (resistivity, EM, magnetometer, and radar). Mechanical waves are additionally useful for the determination of elastic properties of subsurface media, primarily the small-strain shear modulus. Electromagnetic methods can help locate anomalous regions, such as underground cavities, buried objects, and utility lines. The geophysical tests do not alter the soil conditions and, therefore, classify as *non-destructive*. Several are performed at the surface level (termed *non-invasive*). The advantages of performing geophysical methods include

- nondestructive and/or non-invasive,
- fast and economical testing,
- provide theoretical basis for interpretation, and are
- applicable to soils and rocks.

The primary disadvantage is that no samples or direct physical penetration tests are taken. Models are also assumed for interpretation, which sometimes appears to be an art. The results are also affected by cemented layers or inclusions, and are influenced by water, clay, and depth.

The most common geophysical methods used for pavement evaluation is deflection response testing, with a majority of the agencies using the falling weight deflectometer (FWD) impulse type method (as previously mentioned in Sections 4.2.2 and 4.2.3 for rehabilitation and reconstruction projects). In rehabilitation and reconstruction projects, this method provides a direct evaluation of the stiffness of the existing pavement layers under simulated traffic loading. FWD, especially the newer lightweight deflectometers (LWD), can also be used during the construction of new pavements to confirm subgrade stiffness characteristics, either for verifying design assumptions or providing a quality control (QC) tool. LWD, along with other methods used for evaluating the stiffness of natural or compacted subgrades for construction control, are discussed in Chapter 8.

Other deflection methods include steady-state dynamic methods, which produce a sinusoidal vibration in the pavement, and quasi-static devices, which measure pavement deflections from a slow, rolling load (*e.g.,* the Benkelman beam). The most promising development in deflection methods is the high-speed deflectometers, which measure deflections while continuously moving. While these methods increase the complexity of measurement, they offer significant advantages in terms of safety, through reduced traffic control requirements, productivity (typically 3 – 20 km/hr {2 – 12 mph}), and increased volume of information. A detailed review of each of these deflection methods in provided in NCHRP Synthesis 278 (Newcomb and Birgisson, 1999), and guidelines for deflection measurements are provided in ASTM, one on general dynamic deflection equipment (ASTM D 4695) and one on falling-weight-type impulse load devices (ASTM D 4602).

Electrical-type geophysical tests may also be used in pavement design and construction, including surface resistivity (SR), ground penetrating radar (GPR), electromagnetic conductivity (EM), and magnetic survey (MS). These electrical methods are based on the resistivity or, conversely, the conductivity of pore water in soil and rock materials. Mineral grains comprised of soils and rocks are essentially nonconductive, except in some exotic materials, such as metallic ores, so the resistivity of soils and rocks is governed primarily by the amount of pore water, its resistivity, and the arrangement of the pores. These techniques allow for mapping of the entire surface area of the site, making them useful in imaging the generalized subsurface conditions and detecting utilities, hidden objects, boulders, and other anomalies. The mapping is conducted on a relative scale of measurements that reflect changes across the property. For rehabilitation projects and reconstruction projects, GPR is often used for mapping the thickness of existing pavement layers. Electrical-type methods may also aid in finding underground cavities, caves, sinkholes, and erosion features in limestone and dolomite terrain. In developed areas, they may be used to detect underground utility lines, buried tanks and drums, and objects of environmental concern. Additional details on SR, EM, GPR, and MS can be found in Greenhouse, et al. (1998), FHWA manual on *Application of Geophysical Methods to Highway Related Problems* (Wightman et al., 2003), and in the geophysical information portion of the Geoforum website at http://www.geoforum.com/info/geophysical/.

Mechanical wave geophysical methods are also used in pavement design and construction, including seismic refraction, seismic reflection, and, most recently, spectral analysis of surface waves (SASW). Both methods can be used to locate the depth to bedrock. Seismic refraction is also a key method for estimating rippability of rock. The use of the SASW mechanical wave method for determining subgrade modulus values for pavement design has recently been demonstrated in field trails. An automated device has been developed and is being tested by the Texas DOT. However, the testing and interpretation time is still somewhat long for use in pavement applications (Newcomb and Birgisson, 1999, and Wightman et al., 2003).

A general summary for each of the more common geophysical methods used in pavement design is outlined in Tables 4-2 through 4-7. Application examples are provided following the tables.

Table 4-2. Falling weight deflectometer (FWD).

Reference Procedures	ASTM D4694 (*Deflections with a Falling Weight Type Impulse Load Device*); *LTPP Manual for Falling Weight Deflectometer Measurement: Operational Field Guidelines* (August 2000).
Purpose	Used to determine the variation of pavement layer and subgrade stiffness along a length of pavement. For geotechnical features, can be used to backcalculate resilient modulus of subgrade and previously constructed base layers and to identify areas (*e.g.,* weak subgrade conditions) requiring boring and sampling.
Procedure	As described in ASTM D4694, the FWD method consist of applying an impulse load to the paved or unpaved road surface using a falling weight, typically between 4 – 107 kN (1,000 – 24,000 lbs), dropped on a plate resting on the pavement surface, as shown in Figure 4-4 below. The peak force at impact is measured by a load cell and can be recorded as the impact force or the mean stress (by dividing the load by the plate area). The vertical deflection of the pavement surface is measured at the center of the applied load and at various distances (up to eight locations are typical) away from the load, as shown in Figure 4-4. The method usually uses a vehicle or a trailer that is brought to a stop with the loading plate positioned over the desired test location. Several tests may be performed at the same location and at the same or different heights. By measuring the deflection response at the same location under different loads (drop heights), the linear or non-linear characteristics of the pavement system and individual layers can be evaluated. The vehicle is then moved to the next location. The plate and deflection sensors are lowered to the pavement surface. For routine surveys, the tests are typically performed on a spacing of 20 – 50 m (70 – 160 ft) along the road. The deflection at the center gives an idea of the overall pavement performance and the difference in deflection between deflections at various distances indicates the conditions of the pavement layers (bound, unbound, and subgrade). Profiles of the deflections can then be plotted over the length of the pavement (Figure 4.4c), in order to show the variation of pavement layer and subgrade stiffness. The deflection bowls obtained from the FWD data can be analyzed to back calculate the effective stiffness (or load spreading ability) of the various pavement and subgrade layers, by matching measured deflections to computed values. Back calculation is most commonly performed using a multi-layer linear elastic model for the pavement layers. For example, the effective pavement modulus, which is a measure of the effective or combined stiffness of all layers above the subgrade, can be determined from the center deflection as follows (AASHTO, 1993): $$d_0 = 1.5pa\left\{\frac{1}{M_R\sqrt{1+\left(\frac{D}{a}\sqrt[3]{\frac{E_P}{M_R}}\right)^2}} + \frac{1-\frac{1}{\sqrt{1+\left(\frac{D}{a}\right)^2}}}{E_P}\right\}$$ where, d_0 = deflection measured at the center of the load plate (and adjusted to a standard temperature of 20°C {68°F} for hot mix asphalt), inches p = load plate pressure, psi

a = load plate radius, inches
D = total thickness of pavement layers above the subgrade, inches
E_p = effective modulus of all pavement layers above the subgrade, psi
1 in = 25.4 mm, 1 psi = 6.7 kPa

Outer deflection reading of the deflection basin primarily reflects the in-situ modulus of the lower soil (or subgrade). The subgrade resilient modulus can be calculated as follows:

$$M_R = \frac{0.24P}{d_r r}$$

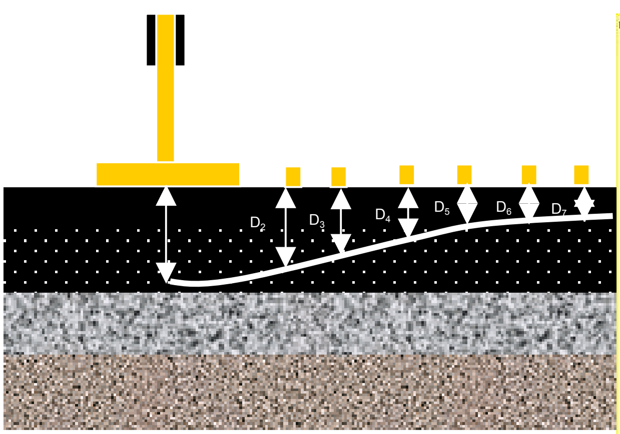
$$r \geq 0.7a_e$$

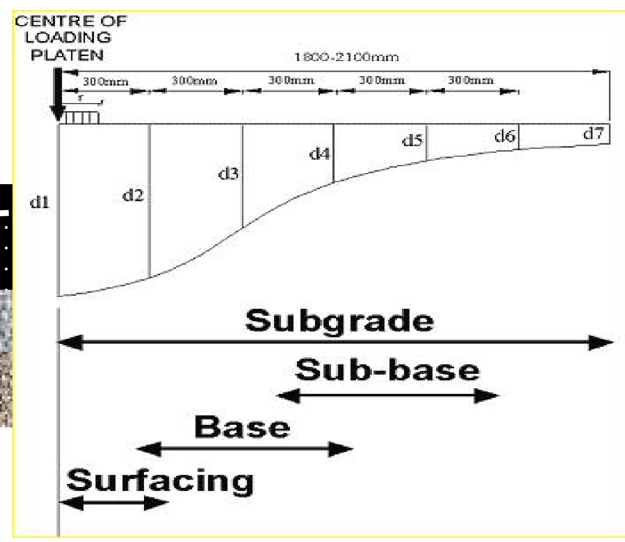
$$a_e = \sqrt{\left[a^2 + \left(D\sqrt[3]{\frac{E_P}{M_R}}\right)^2\right]}$$

where,
M_R = back-calculated subgrade resilient modulus, psi
P = applied load, pound
d_r = deflection at a distance r from the center of the load, inches
r = distance from center of load, inches
a_e = radius of the stress bulb at subgrade-pavement interface, inches
a = load plate radius, inches
D = total pavement thickness above subgrade, inches
1 in = 25.4 mm, 1 psi = 6.7 kPa,

Subgrade resilient modulus for design purposes is usually less than the value directly from FWD data. The AASTHO design guide (1993) recommends a design subgrade resilient modulus equal to 33% of that back calculated from FWD data for flexible pavement and 25% of the back-calculated value for PCC pavement.

a)





b) **c)**

Figure 4.4. FWD showing: a) typical equipment; b) schematic of procedure; and, c) sketch of deflection bowl for interpretation of results.

Commentary

The FWD produces a dynamic impulse load that simulates a moving wheel load, rather than a static, semi-static, or vibratory load. FWD tests can be used for all construction types (*i.e.,* new construction, rehabilitation, or reconstruction). For new construction, testing can be performed directly on cleared subgrade, or done during construction, after placement of the subbase, base, or pavement surface layer. The method can also be used to evaluate the effectiveness of subgrade improvements (for weak subgrades). Based on the results of FWD, the roadway section can be delineated into design sections with similar properties and the intrusive explorations methods (*i.e.,* in-situ testing and borings) located accordingly to obtain the thickness of layers, confirm subgrade stiffness values, and obtain other characteristics of the subgrade (*e.g.,* soil type and moisture conditions).

The deflection profile along the project may be examined to determine if changes exist in the pavement's structural response. The profile can be used to assist in locating areas where more intensive sampling and testing will be required, greatly improving the efficiency of laboratory evaluation. The profile may also be used to divide the project into design sections. For example, in rehabilitation projects, FWD results can be used to optimize the overlay design and/or subdrainage design in each of the design sections.

It should be noted that the influence depth for elastic deflections measured with FWD may extend more than 9 m (30 ft) and, as a result, may miss near-surface critical features. Also, the results may be influenced by deep and often unknown conditions. FWD results are also affected by temperature and freezing conditions. Thus, it is important to consider the time of day and the season when scheduling the FWD program. As previously indicated, deflection measurements are corrected to a standard temperature, typically 20°C (68°F), and critical season equivalent deflections based on locally-developed procedures.

ADVANTAGES	*DISADVANTAGES*
- Speed, repeatability & equipment robustness	- Static method requires stopping between readings
- Easily transported	- Traffic control required
- Simulates moving traffic loads	- Deep features (*e.g.,* water table and bedrock) and temperature affect results
- Direct evaluation of design M_R values	- M_R over predicted
- Non-destructive	- Requires well-defined layer thickness

Table 4-3. Surface resistivity (SR).

Reference Procedures	ASTM G57 (*Field Measurement of Soil Resistivity [Wenner Array]*)
Purpose	Resistivity is used to locate bedrock, stratigraphy, wet regions, compressible soils, map faults, karstic features, contamination plumes, buried objects, and other uses.
Procedure	Resistivity is a fundamental electrical property of geomaterials and can be used to evaluate soil types and variations of pore fluid and changes in subsurface media (Santamarina et al., 2001). The resistivity (ρ_R) is measured in ohm-meters and is the reciprocal of electrical conductivity ($k_E = 1/\rho_R$). Conductivity is reported in siemens per meter (S/m), where S = amps/volts. Electrical current is put into the ground using two electrodes, and the resulting voltage is measured using two other electrodes. Using pairs or arrays of electrodes embedded into the surface of the ground, a surface resistitivity survey can be conducted to measure the difference in electrical potential of an applied current across a site. The spacing of the electrodes governs the depth of penetration by the resistivity method and the interpretation is affected by the type of array used (Wenner, dipole-dipole, Schlumberger). The entire site is gridded and subjected to parallel arrays of SR-surveys, if a complete imaging map is desired. Mapping allows for relative variations of soil types to be discerned, as well as unusual features. Recently developed *automated resistivity system*s collects much more data than simple SR and combines resistivity sounding and traverse data to form a resistivity section with detailed interpretation, as shown in Figure 4-5.
Commentary	In general, resistivity values increase with soil grain size. Figure 4-6 presents some illustrative values of bulk resistivity for different soil and rock types. Figure 4-7 shows the field resistivity illustrative showing stratigraphic changes. Downhole resistivity surveys can also be performed using electronic probes that are lowered vertically down boreholes, or are direct-push placed. The latter can be accomplished using a resistivity module that trails a cone penetrometer, termed a resistivity piezocone (RCPTu). Downhole resistivity surveys are particularly advantageous in distinguishing the interface between upper freshwater and lower saltwater zones in coastal regions. For new pavement design, surface resistivity can be used to evaluate the areal extent of soil deposits and assist in identifying sample locations. Resistivity is also related to moisture content and can be used to map variations in moisture, and, thus, regions of compressible soils can be delineated. This moisture relation can also be valuable for rehabilitation and reconstruction projects, indicating areas requiring special considerations, such as improved drainage. SR may also be useful in determining the depth of rock, which as previously indicated may have an influence on FWD results and is a design input for ME design. SR can also be used in construction to assist in locating prospective sand, gravel, or other sources of borrow material.

ADVANTAGES	*DISADVANTAGES*
- Moderately fast: ~150 m/hr (500 ft/hr)	- Requires coring concrete or asphalt to insert electrodes
- Fairly simple	- Traffic control required
- Can evaluate significant depths	- Lateral resistivity variations affect results
- Works for higher- or lower-resistance sublayers	- Nearby grounded metal objects affect data
- Automation improves interpretation	- Wetting electrodes required in dry ground

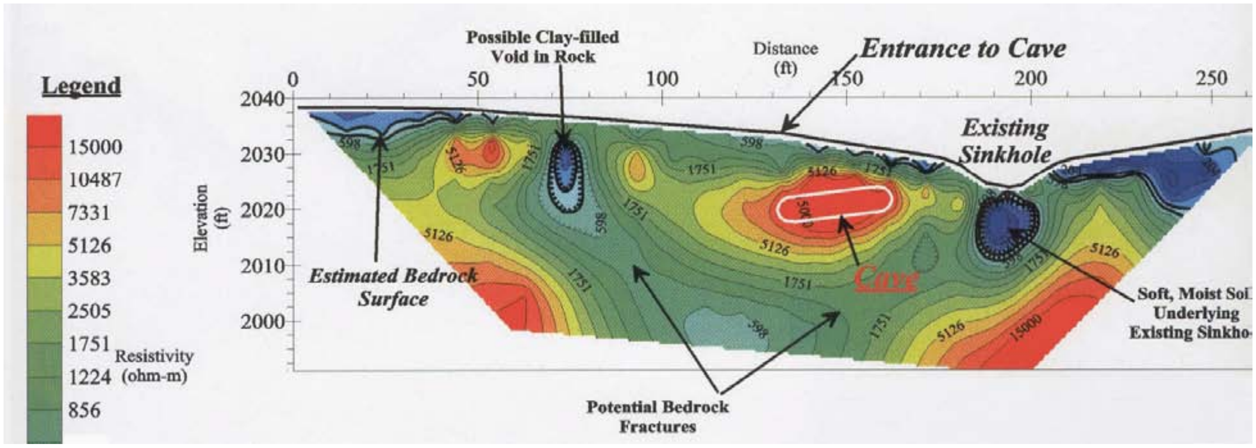


Figure 4-5. Two-dimensional cross-section resistivity profile for detection of sinkholes and caves in limestone (from Schnabel Engineering Associates).

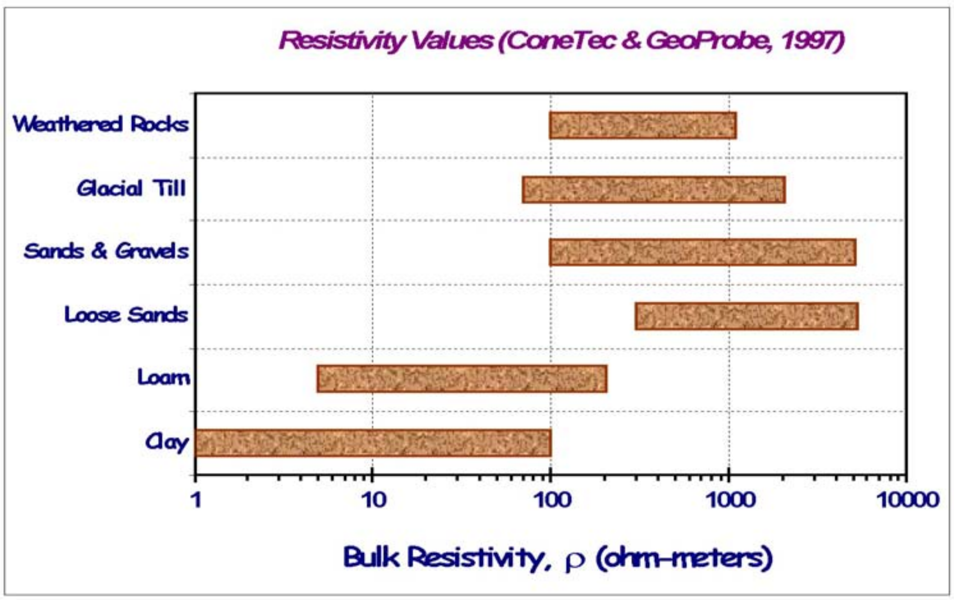


Figure 4-6. Representative values of resistivity for different soils (Mayne et al., 2002).

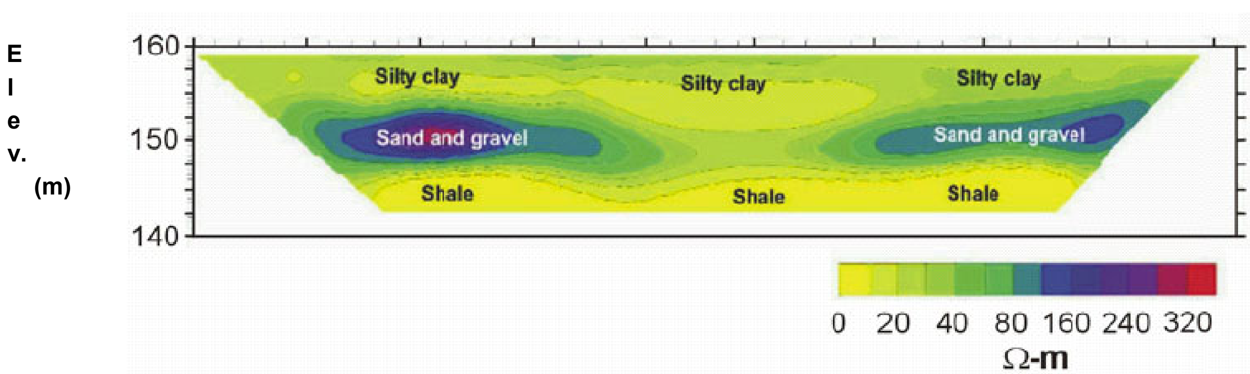


Figure 4-7. Resistivity data showing stratigraphic changes (Advanced Geosciences, Inc.).

Table 4-4. Ground-penetrating radar (GPR).

Reference Procedures	Wightman et al., (2003) *Application of Geophysical Methods to Highway Related Problems*, FHWA Contract Number DTFH68-02-P-00083.
Purpose	GPR can be a valuable tool used to define subsoil strata, moisture variation, depth to rock, voids beneath pavement, buried pipes, cables, as well as to characterize archaeological sites before soil borings, probes, or excavation operations. It can also be utilized to determine the thickness of pavement layers, thus complementing FWD evaluation, and mapping reinforcing steel in concrete surface pavements.
Procedure	Short impulses of a high-frequency electromagnetic waves are transmitted into the ground using a pair of transmitting & receiving antennae. The reflected signals, which occur at dielectric discontinuities in the pavement system and subgrade, are recorded. Thus changes in the dielectric properties (permittivity) of the soil reflect relative changes in the subsurface environment. The GPR surveys are made by driving over the surface with air-coupled antennas mounted on the vehicle or pulling a tracking cart with ground-coupled antenna mounted on a sled across the ground surface. Air-coupled antennas are used to evaluate shallow depths (*e.g.,* thickness of pavement layers) at highway speeds. Ground-coupled antennas are used to evaluate greater depths (up to 18 m (60 ft)). The EM frequency and electrical conductivity of the ground control the depth of penetration of the GPR survey. Many commercial systems come with several sets of paired antennas to allow variable depths of exploration, as well as accommodate different types of ground.
Commentary	The GPR surveys provide a quick imaging of the subsurface conditions, leaving everything virtually unchanged and undisturbed. In pavement engineering practice, GPR using air-coupled antenna is most commonly used to identify layer thickness of pavement materials and perform condition evaluation of pavement surface materials. Methods for improving the accuracy of thickness measurements are reported by Al-Qadi et al., 2003. For subsurface evaluation, ground-coupled GPR is required. The GPR subsurface surveys are particularly successful in deposits of dry sands with depths of penetration up to 20 m or more (65 ft). In wet, saturated clays, GPR is limited to shallow depths of only 1 – 3 m (3 – 10 ft) (still adequate for pavement subgrade evaluation. Searches below the water table are difficult and, in some cases, not possible. Several illustrative examples of GPR surveys are shown in Figure 4-8. Additional information on current usage of GPR by state agencies is contained in NCHRP Synthesis 255 on *Ground Penetrating Radar for Evaluating Subsurface Conditions for Transportation Facilities* (Morey 1998). A recent development (GeoRadar) uses a variably-sweeping frequency to capture data at a variety of depths and soil types. Other developments include combining the use of air-coupled antenna with ski-mounted, ground-coupled antenna to allow for surface and subsurface evaluation at highway speeds.

ADVANTAGES	*DISADVANTAGES*
- Fast: 2 – 80 km/h (1 – 50 mph) & easy to use	- Perception of difficult interpretation
- Different antenna provide different penetration depths and resolution	- May require traffic control
- Produces real-time, continuous subsurface data	- Restricted depth in saturated clay soils
- Non-destructive	

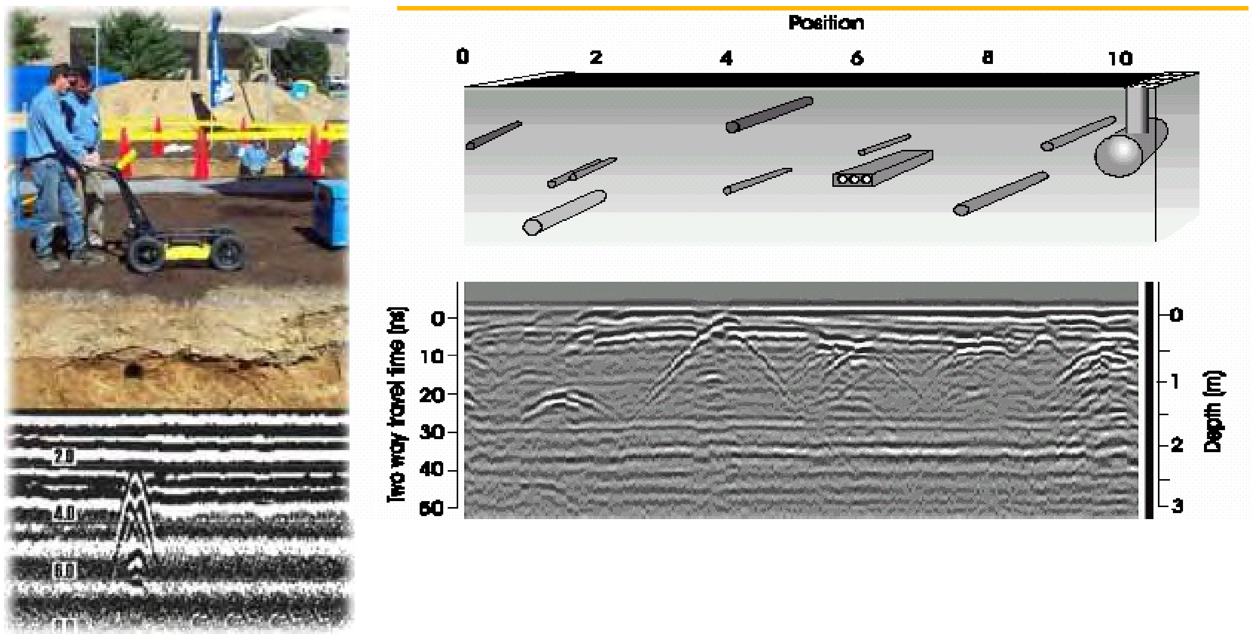
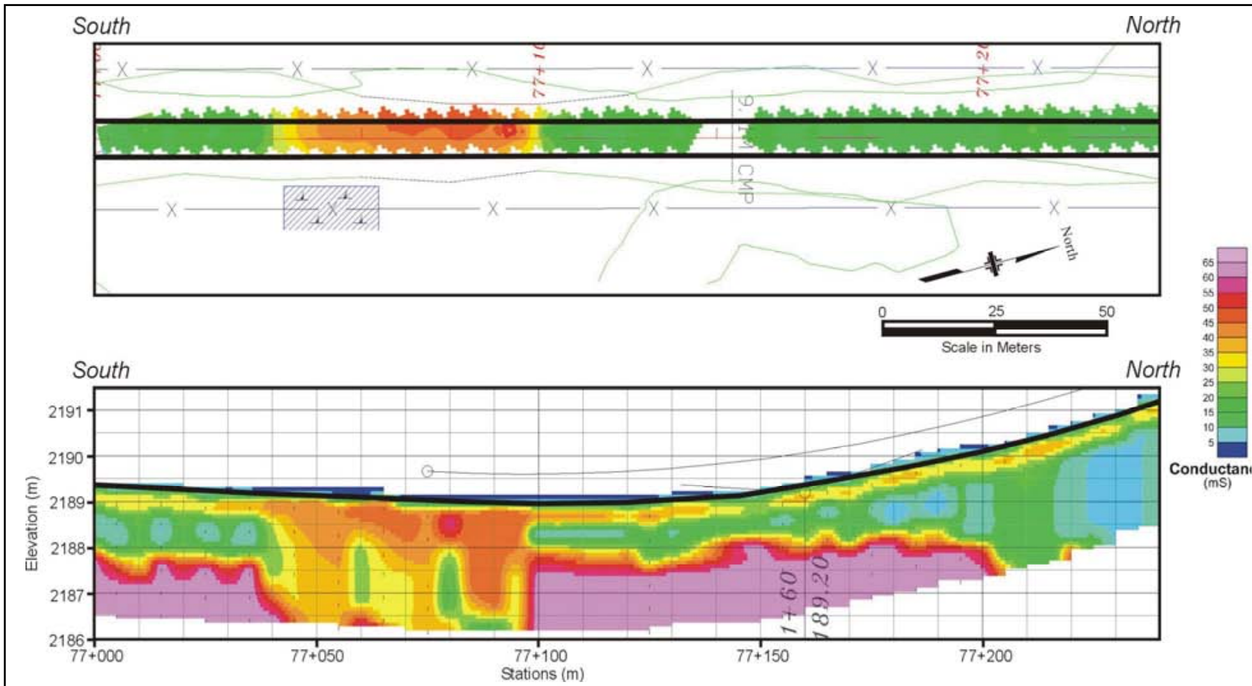


Figure 4-8. Representative ground-coupled GPR results showing buried utilities and soil profile (from EKKO Sensors & Software: www.sensoft.on.ca).



Note: Plan view 0.75 m (2.5 ft) below road surface

Figure 4-9. Conductivity results along a road in New Mexico (Blackhawk GeoServices, Inc).

Table 4-5. Electromagnetic conductivity (EM).

Reference Procedures	Wightman et al., (2003) *Application of Geophysical Methods to Highway Related Problems*, FHWA Contract Number DTFH68-02-P-00083.
Purpose	The EM methods provide a very good tool for identifying areas of clay beneath existing pavements in rehabilitation and road widening projects, or in the subgrade for new construction. These methods are also excellent at tracking buried metal objects and are well known in the utility locator industry. They can also be used to detect buried tanks, map geologic units, and groundwater contaminants, generally best within the upper 1 or 2 m (3 or 6 ft), yet can extend to depths of 5 m (16 ft) or more.
Procedure	Several types of electromagnetic (EM) methods can be used to image the ground and buried features, including: induction, frequency domain, low frequency, and time domain systems. Ground conductivity methods can rapidly locate conductive areas in the upper few meters of the ground surface. These measurements are recorded using several instruments that use electromagnetic methods. Electromagnetic instruments that measure ground conductivity use two coplanar coils, one for the transmitter and the other for the receiver. The transmitter coil produces an electromagnetic field, oscillating at several kHz, that produces secondary currents in conductive material in the ground. The amplitude of these secondary currents depends on the conductivity of the material. These secondary currents then produce secondary electromagnetic fields that are recorded by the receiver coil. Surveys are best handled by mapping the entire site to show relative variations and changes. Areas of high electrical conductivity are likely places to find clay. The choice of which instrument to use generally depends on the depth of investigation desired. Instruments commonly used include the EM38, EM31, EM34 (Geonics Ltd, Canada), and GEM2 (Geophex, USA). The EM38 is designed to measure soil conductivities and has a maximum depth of investigation of about 1.5 m (5 ft). The EM31 has a depth of investigation to about 6 m (20 ft), and the EM34 has a maximum depth of investigation to about 60 m (200 ft). The depth of investigation of the GEM2 is advertised to be 30 – 50 m (100 – 165 ft) in resistive terrain (>1000 ohm-m) and 20 – 30 m (65 – 100 ft) in conductive terrain (<100 ohm-m).
Commentary	Clay is almost always electrically conductive, and areas of high conductivity have a reasonable chance that they will contain clay (*e.g.,* see Figure 4-9). However, estimating the amount of clay from conductivity measurements alone is generally not possible. Conductivity is influenced by many factors including the degree of saturation, porosity, and salinity of the pore fluids. Conductivity measurements taken with instruments that investigate to depths greater than the upper layer are also influenced by other layers.

ADVANTAGES	*DISADVANTAGES*
- Moderately fast: 2 – 8 km/hr (1 – 5 mph)	- Influenced by above- and below-ground metal (*e.g.,* fence post, utilities, rebar).
- Data is easy and efficient to record	- Several passes may be required
- Instruments used in different modes to maximize information at tailored depths	- Traffic control required

Table 4-6. Mechanical wave using seismic refraction.

Reference Procedures	ASTM D5777 . Wightman et al., (2003) *Application of Geophysical Methods to Highway Related Problems*, FHWA Contract Number DTFH68-02-P-00083.
Purpose	Seismic refraction surveys are used to locate depth and characteristics (*e.g.,* rippability) of bedrock, as well as evaluate dynamic elastic properties of the soil and rock .
Procedure	Seismic refraction involves placing a line of regularly spaced sensors (geophones) on the surface and measuring the relative arrival time of seismic energy transmitted from a specified source location. Seismic waves produced by the energy source penetrate the overburden and refract along the bedrock surface, continually radiating seismic waves back to the ground surface, as shown in Figure 4-10. Refraction data are recorded in the field using a portable seismograph, multiple (generally 12 per line) geophones (generally <15 Hz), a repeatable seismic source (*e.g.,* sledgehammer striking a metal plate or light explosive charges), and a power source. Sledgehammer sources are generally used for depths less than 10 – 15 m (30 – 50 ft) and explosives for greater depths up to 30 m (100 ft). Mechanical waves generated by the seismic source include the compression (P-wave) and shear (S-wave) wave types that are measured. P waves are the first arrival waves, are the easiest to measure, and are not absorbed by saturated soil units (*i.e.,* shear waves cannot transmit through water). Seismic energy travels with a compression velocity that is characteristic of the density, porosity, structure, and water content of each geologic layer. The seismic refraction survey is planned with respect to anticipated soil/rock velocities to be encountered, the approximate depth to rock, and the end-use of the data (*e.g.,* rippability of the rock). Multiple seismic source points permit improved delineation of soil/rock interfaces.
Commentary	Seismic surveys are not intended to supplant subsurface sampling investigations, but aid in quickly and economically extending subsurface characterization over large areas, filling in the gaps between discrete borings. Although a number of parameters (*e.g.,* uniaxial strength, degree of weathering, abrasivenesss, frequency of planes of weakness) relate to rippability of rock, seismic refraction has historically been the geophysical method utilized to predetermine the degree of rippability. Correlations of rock rippability as published by the Caterpillar Company are shown in Figure 4-11.

ADVANTAGES	*DISADVANTAGES*
- Lightweight equipment, 2-person crew	- Slow (but faster than borings)
- Very effective at locating bedrock	- Traffic control required
- Well-established correlation with rippability	- Velocities must increase in successively deeper strata.
- Can be used where drilling is physically or economically limited	- Water has a higher velocity than soil and some weak or highly jointed rock
- Background seismic noise may interfere with data refinement and interpretation	
- Lateral deposition may influence results	

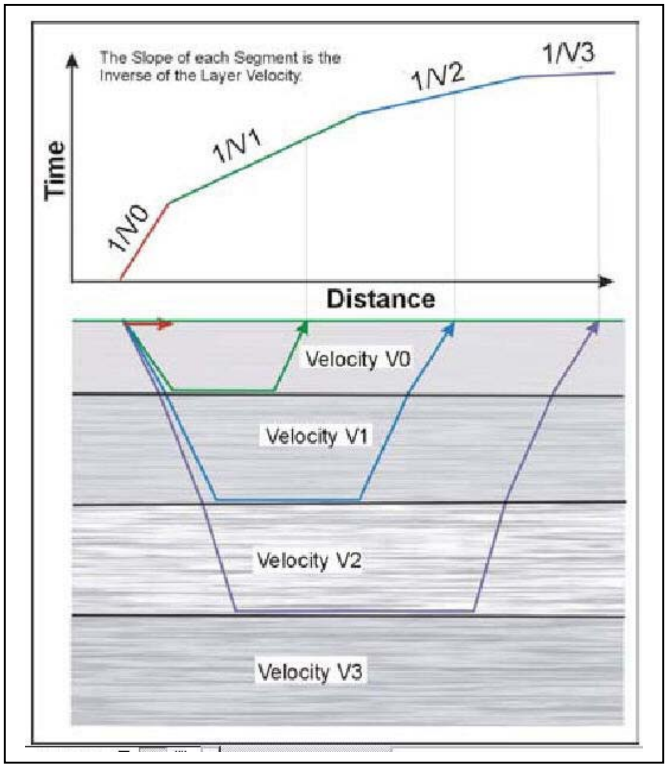


Figure 4-10. Seismic refraction survey (Blackhawk GeoServices, Inc.).

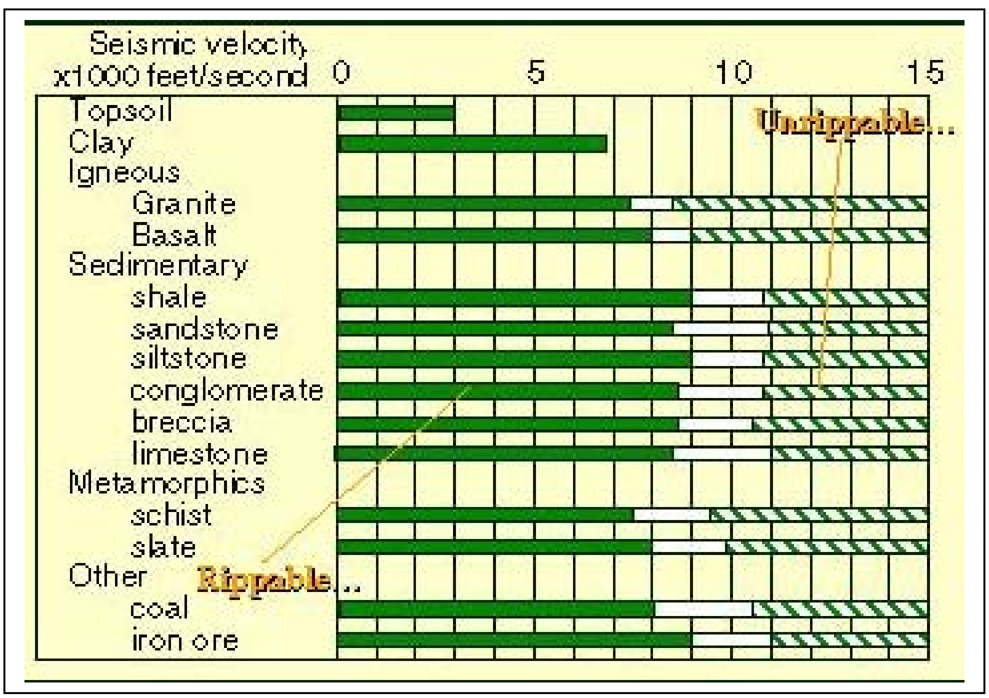


Figure 4-11. Rippability (using a D9 Caterpillar tractor) versus seismic velocity (Caterpillar Handbook of Ripping, 8th Edition).

Figure 4-9 shows an example of using a geophysical method (*i.e.,* EM) for locating clay seams on a project in New Mexico. This project demonstrates the ability to correlate the data with soil type and clay content. For this project survey, data were recorded with one of the EM instruments mounted on a trailer constructed primarily from non-conductive materials. The trailer was towed by an All Terrain Vehicle (ATV). A GPS receiver was also mounted on the trailer to provide position information. Data were recorded automatically at half-second intervals with the EM31 and EM38. Recordings with the EM31 were made at two different instrument heights above the ground, giving two different penetration depths. This procedure required several passes along the road. Having obtained conductivity measurements at a number of different depths at each recording location, the data were modeled and provided the interpreted vertical distribution of conductivity with depth. This interpretation is shown in the lower plot in Figure 4-9. The upper plot shows the ground conductivity measured with the EM38 at a depth of 0.75 m (2.5 ft) below the road surface. This data clearly shows the location of clay materials and provides a clear road map for planning additional exploration and sampling (Wightman et al., 2003).

Another example of a project that effectively incorporated geophysical testing into the investigation program has been reported by the Missouri DOT. Geophysical surveys were conducted for the Missouri Department of Transportation (MoDOT) by the Department of Geology and Geophysics at the University of Missouri-Rolla to determine the most probable cause or causes of ongoing subsidence along a distressed section of Interstate 44 in Springfield, Missouri. Ground penetrating radar (GPR) and reflection seismic survey quickly assessed roadway and subsurface conditions with non-destructive, continuous profiles. The GPR proved to be of useful utility in defining upward-propagating voids in embankment fill material. The reflection seismic survey established the presence of reactivated paleosinkholes in the area. These were responsible for swallowing the fill material as water drained through the embankment. On the basis of interpretation of these data, MoDOT personnel were able to drill into the voids that had developed beneath the pavement (as a result of washing out of the fine-grained material of the embankment fill), and to devise an effective grouting plan for stabilization of the roadway (Newton, et al., 1998).

Geophysical data was used to preclude additional subsurface exploration on a rehabilitation project by the Texas DOT. The project consisted of a 16.9 km (10.5 mile) section of road, which was exhibiting substantial alligator cracking and potholes in the southbound lane, as observed in 1999. The project was constructed in 1979 with 152 mm (6 in.) of lime stabilized subgrade where clay subgrade was present, 254 mm (10 in.) of granular base, and a 54-mm (2-in.) ACP surface. ACP level up courses and two open graded friction courses were then placed in 1988 and 1992, respectively. Maintenance forces had subsequently placed several seal coat patches, AC patches, and ACP overlays. FWD and GPR data were taken in the outside wheelpath at 160 m (0.1 mile) and 3 m (10 ft), respectively. Cores were taken at select locations based on the GPR data analysis. The data indicated that the open graded friction courses were holding water where the maintenance forces had placed the ACP overlays. Cores indicated that the open graded friction course was disintegrating; however, the original ACP layer underneath the friction course was in good shape. FWD data analysis indicated that the base material was structurally in good shape and no base repair and, consequently, no additional subsurface exploration were needed. The overlay and friction course were removed and replaced with a 127 mm (5 in.) ACP overlay in 2001, and was reported to be performing well (Wimsatt, A.J. and Scullion, T., 2003).

4.5.5 In-Situ Testing

In-situ testing can also be used to supplement soil borings and compliment geophysical results. In-situ geotechnical tests include penetration-type and probing-type methods, in most cases without sampling, to directly obtain the response of the geomaterials under various loading situations and drainage conditions. In-situ methods can be particularly effective when they are used in conjunction with conventional sampling to reduce the cost and the time for field work. These tests provide a host of subsurface information, in addition to developing more refined correlations between conventional sampling, testing, and in-situ soil parameters.

With respect to pavement design, in-situ tests can be used to rapidly evaluate the variability of subgrade support conditions, locate regions that require sampling, identify the location of rock and groundwater with some methods, and, with correlation, provide estimates of design values. Design values should always be confirmed through sampling and testing. Table 4-7 provides a summary of in-situ subsurface exploration tests that have been used for design of pavements and evaluation of pavement construction considerations.

For new pavement design, the most utilized in-situ method is the standard penetration test (SPT); however, the dynamic cone penetrometer (DCP) and/or electronic cone penetrometer test (CPT) (see Figure 4-12) should be given special consideration for pavement design and evaluation (as many agencies are currently doing). DCP and CPT offer a more efficient and rapid method for subgrade characterization and have a significantly greater reliability than SPT, as explained in Table 4-8 on the SPT, Table 4-9 on the DCP, and Table 4-10 on the CPT.

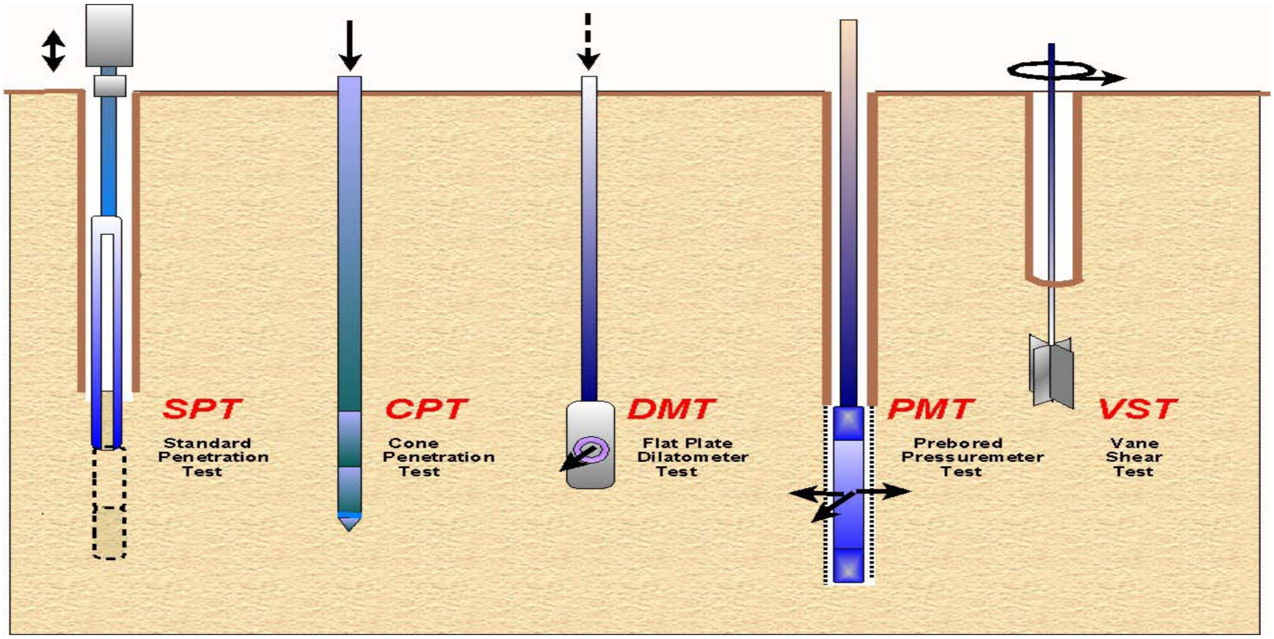


Figure 4-12. Common in-situ tests for pavement evaluation (Mayne et al., 2002).

The CPT and DCP provide information on subsurface soils, without sampling disturbance effects, with data collected continuously on a real-time basis. Stratigraphy and strength characteristics are obtained as the CPT or DCP progresses. Since all measurements are taken during the field operations and there are no laboratory samples to be tested, considerable time and cost savings may be appreciated. DCP is more qualitative than CPT, and is only useful for identifying variation in the upper meter of soil; however, it is performed with low cost, lightweight equipment, with a one- to two-person crew. DCP offers an excellent tool to perform initial exploration through core holes in the surface pavement in rehabilitation projects. Results of DCP tests through the pavement can be compared to test in the shoulders for road widening projects. DCP can also be an effective tool in the construction of pavement to evaluate the suitability of the subgrade after cut, fill, or stabilization operations, as discussed in Chapter 8, and the requirements for stabilization (as discussed in Chapter 7). The CPT provides more quantitative results, can be correlated directly to design properties and types of subgrade, and is useful to greater depths than the DCP in fine grained and sand type soils. Use of CPT and these correlations are detailed in the FHWA Subsurface Investigation Manual (FHWA NHI-01-031).

Other in-situ tests, such as pressuremeter (PM), dilatometer test (DMT) and vane shear test (VST), are also useful in obtaining in-situ design properties, as outline in Table 4-7 but require special skilled personnel and are time intensive. Thus, they are not often used for pavement design. There are also a number of static load tests (*e.g.,* plate load and field CBR) that can be used to assess stiffness and/or strength of the subgrade surface. These tests are most valuable for reconstruction and rehabilitation projects Additional information on in-situ testing can be found on the website http://www.ce.gatech.edu/~geosys/misc/links.htm.

The relevance of each test also depends on the project type and its requirements. The general applicability of the test method depends in part on the geomaterial types encountered during the site investigation, as shown in Table 4-7.

4.5.6 Borings and Sampling

The final exploration method includes drilling bore holes or, in some cases, making excavations to obtain samples. This is the most complex and expensive part of the exploration program, and requires a great degree of care. 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 and verify geophysical and in-situ exploration results.

Disturbed samples are generally obtained to determine the soil type, gradation, classification, consistency, moisture-density relations (Proctor), CBR, presence of contaminants, stratification, etc. The methods for obtaining disturbed samples vary from hand or mechanical excavation of test pits using truck-mounted augers and other rotary drilling techniques. These samples are considered "disturbed," since the sampling process modifies their natural structure.

Undisturbed samples are obtained where necessary to determine the in-place stiffness and strength, compressibility (settlement), natural moisture content, unit weight, permeability, discontinuities, fractures, and fissures of subsurface formations. Even though such samples are designated as "undisturbed," in reality they are disturbed to varying degrees. The degree of disturbance depends on the type of subsurface materials, type and condition of the sampling equipment used, the skill of the drillers, and the storage and transportation methods used. **Serious and costly inaccuracies may be introduced into the design if proper protocol and care is not exercised during recovery, transporting, or storing of the samples.**

Table 4-11 provides a summary of the use and limitation of boring methods using disturbed and undisturbed sampling equipment. Additional information on each of these methods is contained in FHWA NHI-01-031.

Table 4-7. In-situ tests for subsurface exploration in pavement design and construction.

Type of Test	Best Suited For	Not Applicable	Properties That Can be Determined for Pavement Design and Construction	Remarks
Standard Penetration Test (SPT)* AASHTO T 206 & ASTM D1586	Sand & Silt	Gravel, questionable results in saturated Silt	Crude estimate of modulus in sand. Disturbed samples for identification and classification. Evaluation of density for classification.	Test best suited for sands. Estimated clay shear strengths are crude & should not be used for design. See Table 4-8 and FHWA NHI-01-031.
Dynamic Cone Test (DCP)* ASTM D6951	Sand, Gravel, & Clay	Clay with varying gravel content	Qualitative correlation to CBR. Identify spatial variation in subgrade soil and stratification.	See Table 4-9 and FHWA TS-78-209.
Static Piezocone Test (CPT)* ASTM D3441	Sand, Silt, Clay		Undrained shear strength and correlation to CBR in clays, density & strength of sand & gravel. Evaluation of subgrade soil type, vertical strata limits, and groundwater level.	Use piezocone for pore pressure data. Tests in clay are reliable only when used in conjunction with other calibration tests (*e.g.,* vane tests). See Table 4-10 and FHWA NHI-01-031.
Field CBR	Sand, Gravel, Silt, Clay	Granular Soils (Lab and field correlations erratic.)	Load-deflection test providing direct evaluation of CBR and can be correlated with subgrade modulus k-value.	Slow, and field moisture may not represent worst-case condition.
Plate Load Test AASHTO T222 & ASTM D1196	Sand, Gravel, Silt, Clay		Subgrade modulus k-value.	Slow and labor intensive.
Vane Shear Test (VST) AASHTO T-223	Clay	Silt, Sand, Gravel	Undrained shear strength, C_u with correlation to CBR.	Test should be used with care, particularly in fissured, varved, & highly plastic clays. See FHWA NHI-01-031.
Permeability Test ASTM D51216 & ASTM D6391	Sand, Gravel	Clay	Evaluation of coefficient of permeability in base and subbase for rehabilitation projects.	Variable head tests in boreholes have limited accuracy. See FHWA NHI-01-031.
Pressuremeter Test (PMT) ASTM D4719	Soft rock, Sand, Silt, Clay		Subgrade modulus k-value & undrained shear strength with correlation to CBR.	Requires highly skilled field personnel. See FHWA IP-89-008 and FHWA NHI-01-031.
Dilatometer Test (DMT)	Sand, Clay		Soil stiffness can be related to subgrade modulus k and compressibility.	Limited database and requires highly skilled field personnel. See FHWA NHI-01-031.

* These tests can be used in pavement design to qualitatively evaluate subgrade stratification and determine optimum undisturbed sample locations required to obtain design property values.

Table 4-8. Standard penetration test (SPT).

Reference Procedures	AASHTO T 206 and ASTM D 1586. Standard Penetration Test and Split-Barrel Sampling of Soils.
Purpose	A quick means to evaluate the variability of the subgrade with correlation to density of granular soils and to obtain disturbed samples.
Procedure	The SPT involves the driving of a hollow, thick-walled tube into the ground and measuring the number of blows to advance the split-barrel sampler a vertical distance of 300 mm (1 ft). A drop weight system is used for the pounding where a 63.5-kg (140-lb) hammer repeatedly falls from 0.76 m (30 in.) to achieve three successive increments of 150-mm (6-in.) each. The first increment is recorded as a "seating," while the number of blows to advance the second and third increments are summed to give the N-value ("blow count") or SPT-resistance (reported in blows/0.3 m or blows per foot). If the sampler cannot be driven 450 mm, the number of blows per each 150-mm increment and per each partial increment is recorded on the boring log. For partial increments, the depth of penetration is recorded in addition to the number of blows. In current U.S. practice, three types of drop hammers (donut, safety, and automatic) and four types of drill rods (N, NW, A, and AW) are used in the conduct of the SPT. The test, in fact, is highly dependent upon the equipment used and the operator performing the test. Most important factor is the energy efficiency of the system. The range of energy efficiency for the current US standard of practice varies from 35 – 85% with cathead equipment, and 80 – 100% with automated trip Hammer equipment. A calibration of energy efficiency for a specific drill rig & operator is recommended by ASTM D-4633 using instrumented strain gages and accelerometer measurements in order to better standardize the energy levels. If the efficiency is measured (E_f), then the energy-corrected N-value (adjusted to 60% efficiency) is designated N_{60} and given by $N_{60} = (E_f/60)\ N_{meas}$ (5-1) The measured N-values should be corrected to N_{60} for all soils, if possible.
Commentary	The test can be performed in a wide variety of soil types, as well as weak rocks, yet is not particularly useful in the characterization of gravel deposits nor soft clays. The fact that the test provides both a sample and a number is useful, yet problematic, as one cannot do two things well at the same time. SPT correlations exist with angle of internal friction, undrained shear strength, and modulus. However, the SPT value and these correlations have large scatter, and should not be used alone for design. For pavement design and construction, SPT provides a measure of subgrade variability. In granular soils, the method provides an evaluation of relative density, which can be correlated to CBR. In addition, disturbed samples are obtained for identification of subgrade materials and for classification tests. SPT results can be sued to identify

locations where undisturbed samples should be taken. SPT data can also be compared with FWD results to confirm reasonableness (*i.e.,* low resilient modulus values should

compare to low SPT values). The following relationships have been suggested by Kulhowey and Mayne (1990) as a first order estimate of Young's modulus (E/P_a):

$E/P_a \sim 5\ N_{60}$ (sands with fines)
$E/P_a \sim 10\ N_{60}$ (clean, normally consolidated sands)
$E/P_a \sim 15\ N_{60}$ (clean, over consolidated sands)
Where, P_a = atmospheric pressure

Correlations have been attempted for estimating undrained shear strength and correspondingly CBR values in cohesive soils from N values. These relationships are extremely widespread in terms of interpretations, soil types, and testing conditions, such that a universal relationship cannot be advanced. In cohesive subgrades, SPT is better used to evaluate the variability of the subgrade (*e.g.,* based on identification and classification of soil types encountered) and identify locations where proper samples (*e.g.,* undisturbed tube samples for resilient modulus tests or bulk samples for CBR tests) should be taken. Alternatively, drill crews could be instructed to switch to tube samples when cohesive soils are encountered.

ADVANTAGES	*DISADVANTAGES*
- Obtain both a sample & a number	- Obtain both a sample & a number*
- Simple & rugged	- Disturbed sample (index tests only)
- Suitable in many soil types	- Crude number for analysis
- Can perform in weak rocks	- Not applicable in soft clays & silts
- Available throughout the U.S.	- High variability and uncertainty (COV of N =15 to 100%)**

Note: *Collection simultaneously results in poor quality for both the sample and the number.
**COV, coefficient of variation, as reported by Kulhawy and Mayne, 1990.

Table 4-9. Dynamic cone penetrometer (DCP).

Reference Procedures	ASTM D 6951. Standard Test Method for Use of the Dynamic Core Penetrometer in Shallow Pavement Applications. The method is also described in FHWA TS-78-209.
Purpose	Another type of test that can be performed in the field to measure the strength of soils in-place, and is being used more commonly for pavement design purposes to estimate the in-place strength of both fine- and coarse-grained soils.
Procedure	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, such as cones (Newcombe and Birgisson, 1999). 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 4.6 and 8 kg (10 and 18 lbs) with the lighter weight applicable for weaker soils. According to NCHRP Synthesis 278 (Newcomb and Birgisson, 1999), more recent versions of the DCP have a cone angle of 60 degrees, with a diameter of 20 mm (0.8 in.).
Commentary	A number of empirical correlations exist to relate the DCP penetration index (DPI) to subgrade strength parameters required for pavement design. The most widely used is (Webster et al., 1994):

$CBR = 292/(DPI)^{1.12}$	for gravel, sand, and silt
$CBR = 1/0.002871\ DCP$	for highly plastic clays
$CBR = 1/(0.017\ DCP)^2$	for low plasticity clays

The above methods were based on a database of field CBR versus DCP penetration rate values collected for many sites and different soil types, and correlated to test results by others (*e.g.,* log CBR = 2.61 – 1.26 log DCP as developed by Kleyn, 1975, and currently used by the Illinois DOT). For DCPs with automatic release hammers (*e.g.,* the Israeli automated DCP), CBR values are about 15% greater than the above correlations for manual hammers (after Newcomb and Birgisson, 1999).

ADVANTAGES
- Can be operated by one or two people
- Site access for testing not a problem
- Equipment is simple, rugged, and inexpensive
- Continuous record of soil properties with depth & immediate results
- Can be used in pavement core holes
- Suitable in many soil types & can perform in weak rocks
- Fair reliability (COV ~ 15 – 22 %)*
- Available throughout the U.S.

DISADVANTAGES
- Does not obtain a sample
- Index tests only
- High variability and uncertainty in gravelly soils
- Limited depth to 1 m (3.3 ft) (however, adequate for most rehab projects and good for rapid surficial characterization).
- Extraction of cone can be difficult.

Note: *COV, coefficient of variation as reported by Kulhawy and Mayne, 1990.

Table 4-10. Cone penetrometer test (CPT).

Reference Procedures	ASTM D-3441 (mechanical systems) and ASTM D 5778 (electric and electronic systems). Test Method for Electronic Cone Penetration Testing of Soils.
Purpose	Fast, economical, and provides continuous profiling of geostratigraphy and soil properties evaluation.
Procedure	The test consists of pushing a cylindrical steel probe into the ground at a constant rate of 20 mm/s (0.8 in/s) and measuring the resistance to penetration. The standard penetrometer has a conical tip with 60^o angle apex, 35.7-mm (1.4 in.) diameter body (10-cm^2 (1.6-in^2) projected area), and 150-cm^2 (23-in^2) friction sleeve. The measured point or tip resistance is designated q_c and the measured side or sleeve resistance is f_s. The ASTM standard also permits a larger 43.7-mm (1.72-in.) diameter shell (15-cm^2 (2.3-in^2) tip and 200-cm^2 (31-in^2) sleeve). Piezocones are cone penetrometers with added transducers to measure penetration porewater pressures during the advancement of the probe. Most electric/electronic cones require a cable that is threaded through the rods to connect with the power supply and data acquisition system at the surface. An analog-digital converter and Pentium notebook are sufficient for collecting data at approximate 1-sec intervals. Depths are monitored using either a potentiometer (wire-spooled LVDT), depth wheel that the cable passes through, or ultrasonic sensor. Systems can be powered by voltage using either generator (AC) or battery (DC), or alternatively run on current. New developments include (1) the use of audio signals to transmit digital data up the rods without a cable and (2) memocone systems where a computer chip in the penetrometer stores the data throughout the sounding.
Commentary	The CPT can be used in very soft clays to dense sands, yet is not particularly appropriate for gravel or rocky terrain. The pros and cons are listed below. As the test provides more accurate and reliable numbers for analysis, yet no soil sampling, it provides an excellent complement to the more conventional soil test boring with SPT measurements. Figure 4-13 provides a comparison of CPT and SPT logs. For pavement design, the CPT provides a continuous log of the vertical variability of the subgrade. CPT can be used to identify soil types and soil consistency, which in turn can be used to determine appropriate type(s) and location(s) for sampling. Empirical relations have been developed for undrained shear strength and elastic modulus, as reviewed in FHWA-NHI-01-031 (Mayne et al., 2002). The Lousiana Tansportation Research Center in cooperation with FHWA has recently developed a correlation between cone parameters and resilient modulus (Muhammad et al., 2002).

ADVANTAGES	*DISADVANTAGES*
- Fast and continuous profiling	- High capital investment
- Economical and productive	- Requires skilled operator to run
- Results not operator-dependent	- Electronic drift, noise, and calibration
- Strong theoretical basis in interpretation	- No soil samples are obtained
- Particularly suitable for soft soils	- Unsuitable for gravel or boulder deposits*
- Good reliability (COV ~ 7 – 12 %)**	

Note: *Except where special rigs are provided and/or additional drilling support is available.
**COV, coefficient of variation as reported by Kulhawy and Mayne, 1990.

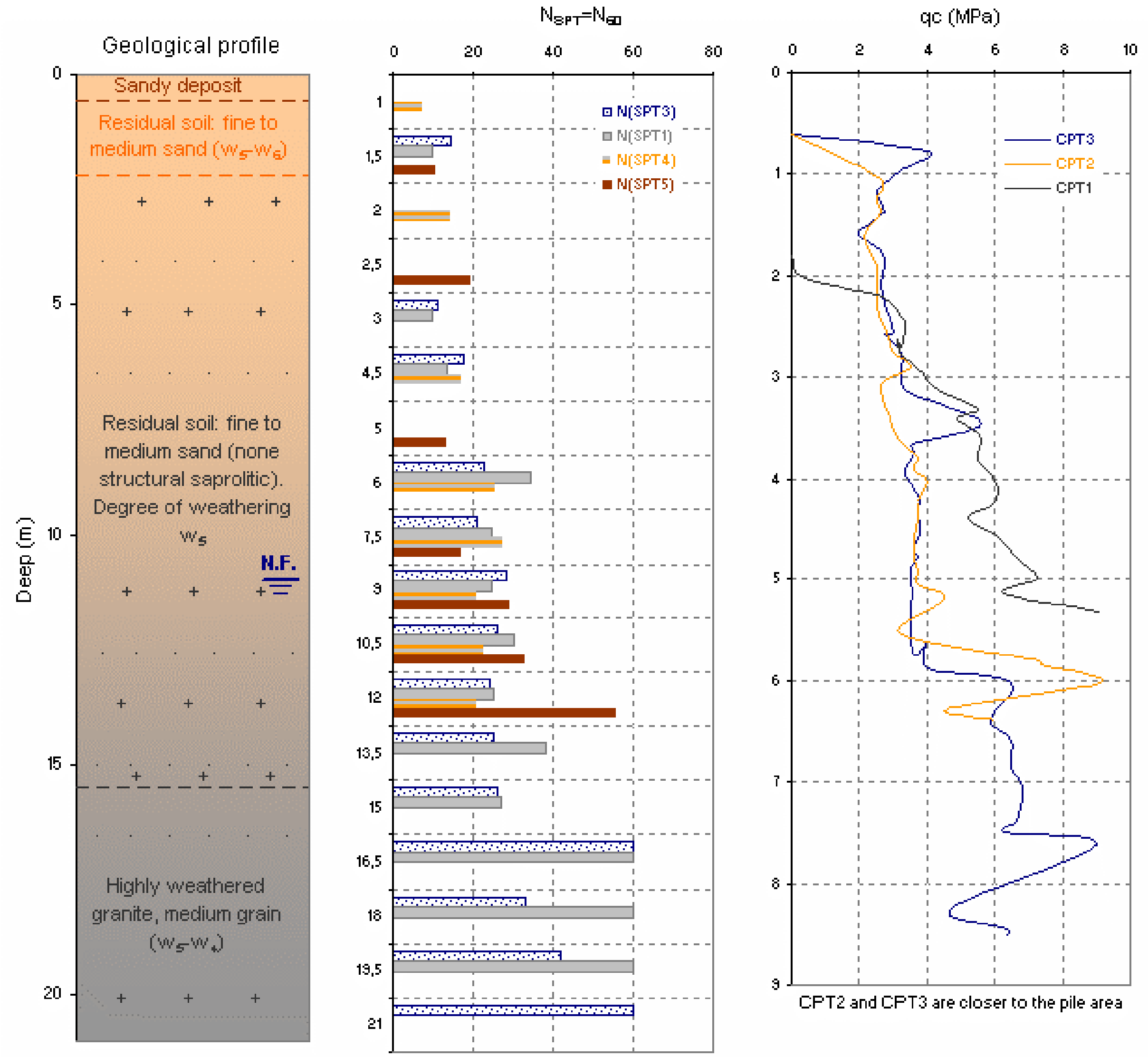


Figure 4-13. CPT log in comparison to SPT data from several locations.

To begin the boring and sampling exploration 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. A typical design practice for pavements is to assign one subgrade support value to long roadway lengths, *i.e.,* 1 – 16 km (0.6 – 10 mi). This approach may be reasonable for uniform soil deposits, especially considering the construction advantage of maintaining a uniform pavement cross section. However, for highly variable sites, this approach is questionable, as it invariably leads to either an overly conservative design or premature pavement distress in some sections. Significant local variations can best be handled as special design features. There may be more variation of soil properties vertically (drill holes) than horizontally at shallow depths; however, again, only one value is assigned.

Thus, one of the primary sampling issues is how best to sample such that appropriate values can be assigned to long sections of roadway. Two sampling options are available: systematic or representative.

Systematic sampling is a common agency practice. It is done at uniform horizontal and/or vertical intervals. Intermediate locations are sampled when varying conditions are encountered. A large number of samples can be obtained, but the testing may either be on a random basis to obtain an average value for similar materials or a representative basis for variable conditions.

Representative sampling and testing consists of taking samples that are believed to be representative of the typical or conservative soil support values. This type of sampling is based primarily on engineering judgment based on other information about the site (*i.e.,* evaluation of available information, site reconnaissance, remote sensing, and geophysical testing) and involves fewer samples.

Table 4-11. Subsurface exploration-exploratory boring methods.

Method	Use	Limitations
Auger Boring ASTM D – 1452	Obtain samples and identify changes in soil texture above water table. Locate groundwater.	Grinds soft particles – stopped by rocks, etc.
Test Boring ASTM D – 1586	Obtain disturbed split spoon samples for soil classification. Identify texture and structures; estimate density or consistency in soil or soft rock using SPT (N).	Poor results in gravel, hard seams.
Thin Wall Tube ASTM D – 1587	Obtain 51 – 86 mm (2 – 3-3/8 in.) diameter undisturbed samples of soft-firm clays and silts for later lab testing (*e.g.,* resilient modulus tests).	Cutting edge wrinkled in gravel. Samples lost in very soft clays and silts below water table.
Stationary Piston Sampler	Obtain undisturbed 51 – 86 mm (2 – 3-3/8 in.) diameter samples in very soft clays. Piston set initially at top of tube. After press is completed, any downward movement of the sample creates a partial vacuum, which holds the sample in the tube. In pavement design, these samples can be used for evaluating pavement settlement and/or treatability studies.	Cutting edge wrinkled in gravel.
Pits, Trenches	Visual examination of shallow soil deposits and man-made fill above water table. Disturbed samples for density and CBR tests, or undisturbed block samples for resilient modulus tests, may be extracted.	Caving of walls, groundwater. Requires careful backfill and compaction.

AASHTO 1993 requires the use of average subgrade support values along the alignment, and uses reliability to account for variation in subgrade strength along the alignment. To obtain a true statistical average, random sampling would be appropriate, provided the soil conditions are rather homogeneous. Systematic is not random, but it may be close with respect to averaging. Unfortunately, with systematic, additional borings are often not performed in areas where varying conditions are encountered. So while an average may be achieved, localized conditions along the alignment that could significantly impact performance are often missed. Statistically, the objective is to delineate locations with similar properties (origins and moisture conditions) and assign design values using random methods for the defined population. This is best accomplished by a combination of methods, as outlined in the following subsections.

Frequency (number/spacing) of Borings

The design engineer should prescribe the spacing and depth of the borings based on an evaluation of available information. As indicated in the previous section, only limited representative borings and sampling are required if geophysical and in-situ testing have been performed. Again, some borings should be performed at several cone locations for calibration and at critical locations identified by the preceding methods. A more extensive program is required in the absence of this alternative exploration information.

The spacing and depth of these borings depend on the variability of the existing soil conditions, both vertically and horizontally, and the type of pavement project. Spacing of borings vary considerably among agencies, on the order of 12 per km (20 per mi) to as few as 2 per km (3 per mi), with spacing generally decreased with high-volume roads and fine-grained soils, as reported by Newcomb and Birgisson, 1999. Considering the variability of soils and the tests used to evaluate geotechnical materials, even the high number appears relatively low. The following provides a review of recommended practice from a geotechnical perspective based on guidelines from textbooks, several state agencies, and the FHWA.

The spacing of borings along the roadway alignment generally should not exceed 60 m (200 ft) for a fully invasive program. Where subsurface conditions are known to be uniform, a minimum spacing of 120 m (400 ft) is generally recommended. In a program supported by geophysical and in-situ tests, such as recommended in Sections 4.5.4 and 4.5.5, a spacing of 150 – 450 m (500 – 1500 ft) as indicated in NCHRP 1-37A may be all that is necessary, depending on the uniformity of site conditions. For new pavement projects, most agencies locate borings along the centerline, unless conditions are anticipated to be variable. Borings should be located to disclose the nature of subsurface materials at the deepest points of cuts, areas of transition from cut to fill, and subgrade areas beneath the highest points of embankments. The spacing and location of the borings should be selected considering the geologic complexity and soil/rock strata continuity within the project area, with the objective of defining the vertical and horizontal boundaries of distinct soil and rock units within the project limits. It should be noted that **the cost for a few extra borings is insignificant in comparison to the cost of unanticipated field conditions or premature pavement failure**.

The spacing of borings for rehabilitation and reconstruction projects will depend on the condition of the existing pavement, the performance of non-destructive geophysical tests, and the availability of previous subsurface information. As indicated in the NHI (1998) “Techniques for Pavement Rehabilitation” Participants Manual, drilling and sampling is performed on three levels: 1) a high level in the absence of non-destructive geophysical tests, 2) a low level to complement geophysical tests, and 3) at a diagnostic level to evaluate mechanisms of distress where it occurs. In the absence of non-destructive geophysical tests, spacing on the order of one boring every 150 m (500 ft) would appear to be a minimum for pavements with no unusual distressed conditions. Additional borings should be located in problem areas (*e.g.,* areas of rutting or fatigue cracking, which are often associated with subgrade issues) identified in the condition survey as discussed in Sections 4.2.2 and 4.2.3. The number of borings should be increased to the level of new pavement projects when rehabilitation projects include substantial pavement removal and replacement. Again, performance of geophysical tests (*e.g.,* FWD) and/or in-situ tests (*e.g.,* DCP) tests could be used to supplement borings, in which case, sampling at a minimum of every 450 m (1500 ft) may be adequate to complement the geophysical or non-destructive test results, provided there are no areas of significant distress that require special attention. Spacing of borings should be decreased as the variability of the geophysical or in-situ results increase to verify those results via laboratory testing.

For pavement rehabilitation projects, borings should be located in the wheel path to evaluate performance of existing unbound materials, as well as the subgrade. Borings should also be specifically located (and the number increased as required) to investigate the presence of wet or soft subgrade conditions indicated by site reconnaissance and/or maintenance records. If the project involves replacing or rubbilizing the existing pavement, all borings would be drilled through the existing pavement. If the project involves adding a lane, plus replacing or rubbilizing the existing pavement, half the borings should be in the new lane and half in the existing pavement.

As previously indicated in the introduction of Section 4.5.6, borings should be taken to a minimum depth of 1.5 – 2 m (5 – 7 ft) below the proposed pavement subgrade elevation, with at least a few borings taken to 6 m (20 ft) below the grade line. These deeper borings should also be used to determine the water table depth and occurrence of bedrock. Deeper borings are not generally required for rehabilitation projects, unless the previous section experienced premature failure due to subgrade conditions or there is a change in vertical alignment. 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 to support the pavement system. Borings should extend a minimum of 1.5 m (5 ft) into relatively stiff or dense soils beneath soft deposits. 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 (less than 10% of the applied surface stress) that consideration will not significantly influence surface settlement.

Greater depth of borings may be required where deep cuts are to be made, side hill cuts are required, large embankments are to be constructed, or subsurface information indicates the presence of weak (or water-saturated) layers. 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. For side hill cuts, additional borings should be performed on the uphill side in uniform soil conditions and on the uphill and downhill side for nonuniform conditions. Additional borings may be required for slope stability considerations and analysis.

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 (NCHRP 1-37A, 2003), as discussed later in this section.

Take sufficient and appropriate auger, split tube, or undisturbed samples of all representative subsoil layers, as discussed in the next section. 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.

Subsurface investigation programs, regardless of how well they may be planned, must be flexible to adjust to variations in subsurface conditions encountered during drilling. The project engineer should, at all times, be available to confer with the field inspector. On critical projects, the engineer responsible for the exploration program should be present during the field investigation. He/she should also establish communication with the design engineer to discuss unusual field observations and changes to be made in the investigation plans.

Soil Sampling (after NCHRP 1-37A)

Sampling will vary with the type of pavement project. For new construction projects, a majority of the samples taken will most likely 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. In rehabilitation projects, sampling to determine the potential of full depth reclamation or the potential for rubbilization of asphalt pavements is somewhat different, requiring the sampling of the in-place base, subbase, and surface pavement to determine its suitability for reuse and/or rubblizing. The condition survey, as discussed in section 4.2.3, will help in identifying areas requiring sampling and the types of samples required. In general, sampling of the subgrade is not as intensive as is needed for new pavements. Detailed sampling of the base, subbase, and surface pavement will be required to determine if there is a large amount of variability in materials along the project and the condition of those materials for reuse (*e.g.,* base and subbase that has been contaminated with large quantities of fines would not be desirable).

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 1.5 m (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 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. Undisturbed samples should comply with the following criteria:

1. The samples should contain no visible distortion of strata, or opening 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 1.5-m (5-ft) depth interval, or just below the planned surface elevation of the subgrade. Recommended procedures for obtaining undisturbed samples are described in AASHTO Standard T207, *Thin-Walled Tube Sampling of Soils*. If undisturbed samples cannot be recovered, disturbed samples should be taken.

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. Special care is required for undisturbed tube samples. When additional undisturbed sample borings are taken, the undisturbed samples are sent to a soils laboratory for testing. Drilling personnel should exercise great care in extracting, handling, and transporting these samples to avoid disturbing the natural soil structure. Tubes should only be pressed, not driven with a hammer. The length of press should be 100 – 150 mm (4 – 6 in.) less than the tube length (DO NOT OVERPRESS). A plug composed of a mixture of bees' wax and paraffin should be poured to seal the tube against moisture loss. The void at the upper tube end should be filled with sawdust, and then both ends capped and taped before transport. The most common sources of disturbance are rough, careless handling of the tube (such as dropping the tube samples in the back of a truck and driving 50 km (30 mi) over a bumpy road), or temperature extremes (leaving the tube sample outside in below zero weather or storing in front of a furnace). Proper storage and transport should be done with the tube upright and encased in an insulated box partially filled with sawdust or expanded polystryrene to act as a cushion. Each tube should be physically separated from adjacent tubes, like bottles in a case. A detailed discussion of sample preservation and transportation is presented in ASTM D 4220, *Practice for Preserving and Transporting Soil Samples,* along with a recommended transportation container design.

Rock Sampling

The need for sampling rock will depend on the location of bedrock with respect to the design subgrade elevation, geology of the region, the availability of geophysical data and local experience. The transition from soil to weathered rock to sound rock can be erratic and highly variable, often causing major geotechnical construction problems (*i.e.,* claims). Rock above the subgrade elevation will need to be removed by ripping or blasting. Considering blasting typically cost 4 to 20 times more than ripping, in addition to the noise and vibration problems associated with blasting, a determination of ripability is an important part of the subsurface exploration program. As previously discussed in Section 4.5.4, ripability can be determined by refraction survey methods, and should be confirmed by coring a sampling of the rock. SPT values have also been used to assess ripability, with values or 80 to 100 typically assumed to be the demarcation between ripping and blasting (Rolling and Rolling). However, there do not appear to be any hard-and-fast rules. The regional geology and the local ability of the contractor are both significant factors. Considering the determination of ripping versus basting is not an exact science, test pits are recommended to confirm the exploration results.

If the bedrock is near the subgrade level, then the pavement design will dictate requirements for additional samples. Technically pavements can be located directly above competent, intact rock with only a cushion/drainage layer, generally consisting of 150 mm (6 in.) of gravel required between flexible or rigid pavement and the rock. The rock surface should be sloped to promote drainage. It is imperative that the rock surface be level to provide a uniform bearing surface and prevent water from being trapped in local depressions. Undulating rock may therefore require additional excavation, especially if pockets contain poor quality materials, such as frost susceptible soils. For example, Figure 4-14 shows representative excavation requirements where frost susceptible soils exist over undulating rock.

Highly weathered rock and deleterious rock (*i.e.,* rock that degrades easily when exposed to the environment) such as shale, will be required to be removed to a greater depth, on the order of 0.6 – 1 m (2 – 3 ft) based on local experience. In either case, the reason for sampling is to determine the competency of the rock and the amount of excavation required.

It is generally recommended that a minimum 1.5-m (5-ft) length of rock core be obtained to verify that the boring has indeed reached bedrock and not terminated on the surface of a boulder (Mayne et al., 2002). Coring methods and evaluation of rock quality is covered in FHWA NHI-01-031. This rock core depth should be followed if rock is encountered within 1 m (3 ft) of pavement subgrade level, and could be reduced if rock is located at greater depths.

Cores should be used to identify the rock, determine the quality of the rock, and evaluate its durability. Evaluation of durability should be based on a review of past performance, slaking tests and physical degradation tests (Rollins and Rollins, 1996). Many problems with deleterious rocks have been regionally identified across the U.S. Durability tests are reviewed in Chapter 5.

Groundwater

Observations of the groundwater level and pressure are an important part of geotechnical explorations for pavement design and construction, and the identification of groundwater conditions should receive the same level of care given to soil descriptions and samples. The water level is part of the input in the mechanistic-empirical design approach. Also, as mention is Section 4.5.4, the location of the water level will influence interpretation of FWD and other geophysical results. The water level is also critical to determine the drainage requirements for construction and long-term performance of the pavement. In addition, the water level will influence the selection of appropriate stabilization methods, as discussed in Chapter 7.

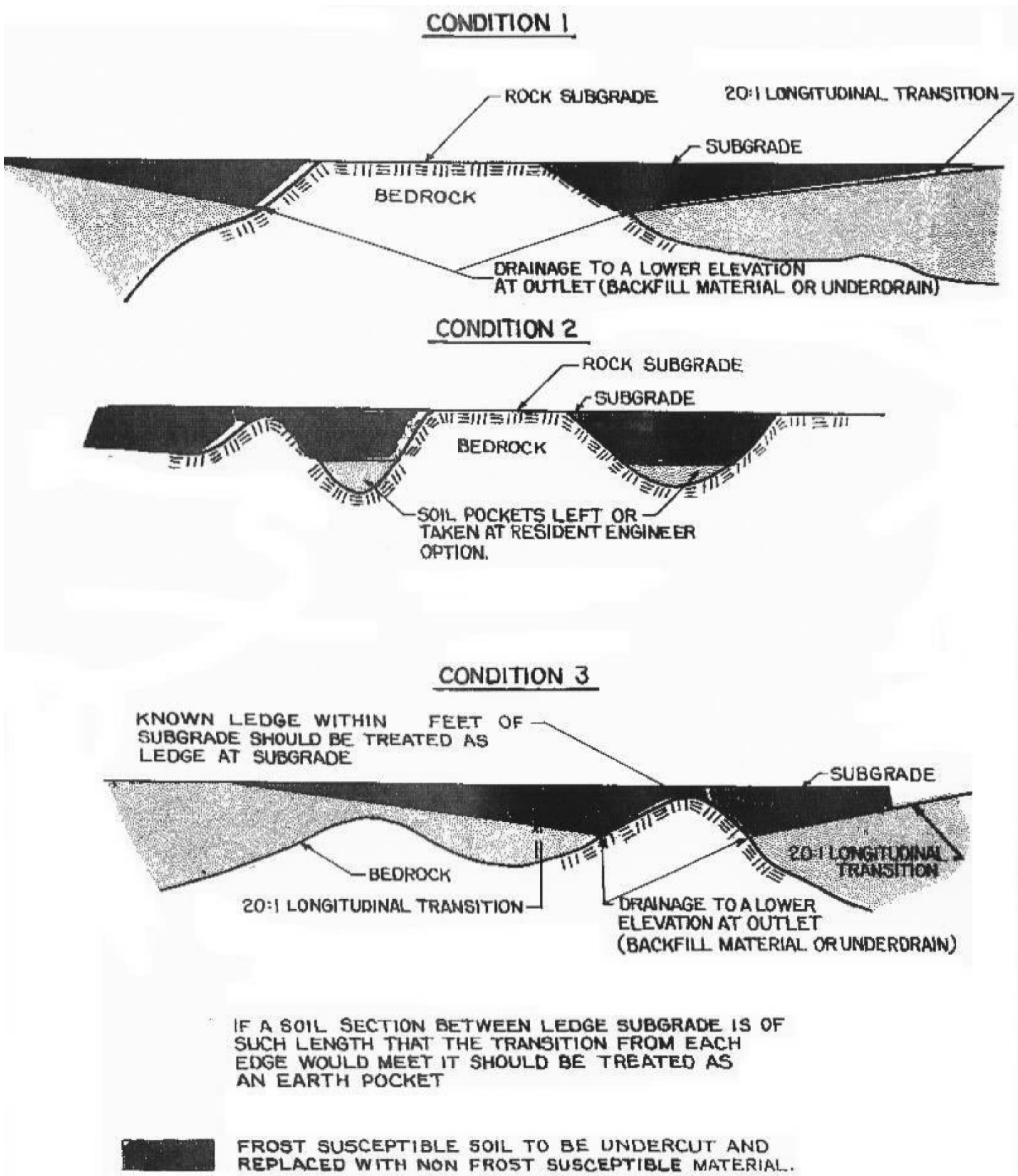


Figure 4-14. Excavation requirements for frost susceptible soils over undulating rock.

Measurements of water entry during drilling, and measurements of the groundwater level at least once following drilling, should be considered a minimum effort to obtain water level data, unless alternate methods, such as installation of observation wells or piezometers, are defined by the geotechnical engineer. Detailed information regarding groundwater observations can be obtained from ASTM D 4750, Standard Test Method For Determining Subsurface Liquid Levels in a Borehole or Monitoring Well and ASTM D 5092, *Design and Installation of Groundwater Wells in Aquifers*.

The water level in the boring is not the only indication of the groundwater level. If the borehole has caved, the depth to the collapsed region should be recorded and reported on the boring record, as this may have been caused by groundwater conditions. The elevations of the caved depths of certain borings may be consistent with groundwater table elevations at the site, and this may become apparent once the subsurface profile is constructed. Drilling mud obscures observations of the groundwater level owing to filter cake action and the higher specific gravity of the drilling mud compared to that of the water. If drilling fluids are used to advance the borings, the drill crew should be instructed to bail and flush the hole prior to making groundwater observations.

Unless the soils are granular with little or no fines (*i.e.,* clay and/or silt size particles), the water level in the boring may take days or weeks to rise to the actual groundwater level. Considering the potential for cave-in and infiltration of surface water during this period and with consideration for the potential for seasonal changes in the groundwater level, a bore hole is usually not the best means to get a true picture of the long-term water conditions at a site. For accurate measures of groundwater, observation wells or piezometers should be installed in the borehole. An "observation well" measures the level in a water table aquifer, while a "piezometer" measures the pressure in a confined aquifer, or at a specific horizon of the geologic profile (Powers, 1992).

The simplest type of observation well is formed by a small-diameter polyvinyl chloride (PVC) pipe set in an open hole. The bottom of the pipe is slotted and capped, and the annular space around the slotted pipe is backfilled with clean sand. The area above the sand is sealed with bentonite, and the remaining annulus is filled with grout, concrete, or soil cuttings. A surface seal, which is sloped away from the pipe, is commonly formed with concrete in order to prevent the entrance of surface water. The top of the pipe should also be capped to prevent the entrance of foreign material; a small vent hole should be placed in the top cap.

Piezometers are available in a number of designs. Commonly used piezometers are of the pneumatic and the vibrating wire type. Interested readers are directed to the reference manuals of the FHWA NHI course on Geotechnical Instrumentation (FHWA-NHI-98-034), FHWA NHI course on Subsurface Investigation (FHWA-NHI-01-031), or Dunnicliff (1988) for a detailed discussion of the various types of piezometers.

Permeability of the subgrade is rarely an issue for pavement design, but may be of interest in terms of dewatering requirements for excavations or installation of interceptor drains to lower groundwater. For rehabilitation projects, permeability of existing base and subbase may be of interest in order to evaluate drainage characteristics (*e.g.,* time to drain) of in-place materials. Field permeability tests may be conducted on natural soils (and rocks) by a number of methods, including simple falling head, packer (pressurized tests), pumping (drawdown), slug tests (dynamic impulse), and dissipation tests. Simple falling head tests are typically used for evaluating the permeability of in-place base and subbase materials. A brief listing of the field permeability methods is given in Table 4-12.

Test Pits

Exploration pits and trenches, excavated by hand, a backhoe, or bulldozer, permit detailed examination of the soil and rock conditions at shallow depths and relatively low cost. Exploration pits can be an important part of geotechnical explorations where significant variations in soil conditions occur (vertically and horizontally), large soil and/or non-soil materials exist (boulders, cobbles, debris) that cannot be sampled with conventional methods, or buried features must be identified and/or measured.

The depth of the exploration pit is determined by the exploration requirements, but is typically about 2 – 3 m (6.5 – 10 ft). In areas with high groundwater level, the depth of the pit may be limited by the water table. Exploration pit excavations are generally unsafe and/or uneconomical at depths greater than about 5 m (16 ft), depending on the soil conditions. **The U.S. Department of Labor's Construction Safety and Health Regulations, as well as regulations of any other governing agency, must be reviewed and followed prior to excavation of the exploration pit, particularly in regard to shoring requirements.**

During excavation, the bottom of the pit should be kept relatively level so that each lift represents a uniform horizon of the deposit. At the surface, the excavated material should be placed in an orderly manner adjoining the pit with separate stacks to identify the depth of the material. The sides of the pit should be cleaned by chipping continuously in vertical bands, or by other appropriate methods, so as to expose a clean face of rock or soil. Survey control at exploration pits should be done using optical survey methods to accurately determine the ground surface elevation and plan locations of the exploration pit. Measurements should be taken and recorded documenting the orientation, plan dimensions and depth of the pit, and the depths and the thicknesses of each stratum exposed in the pit. In logging the exploration pit, a vertical profile should be made parallel with one pit wall. After the pit is logged, the shoring will be removed and the pit may be photographed or video logged at the discretion of the geotechnical engineer. Photographs and/or video logs should be located with reference to project stationing and baseline elevation. A visual scale should be included in each photo or video.

Exploration pits can, generally, be backfilled with the spoils generated during the excavation. The backfilled material should be compacted to avoid excessive settlements. Tampers or rolling equipment may be used to facilitate compaction of the backfill.

Sampling for Fill/Borrow Materials

Samples are also required to determine the suitability of cut materials to be used as fill and to evaluate suitable borrow sources for additional fill, as required, and for base and subbase materials. Many different soils may be suitable for use in the construction of the roadway embankment or fill. The fill for the subgrade material must be of high quality and, preferably, granular material. Silt and clay type soils are less desirable for subgrade, as they will dictate a thicker pavement section. Bulk samples should be obtained in order to determine the moisture-density relations (Proctor) of each soil type encountered. Moisture-density tests should be used to determine the compaction characteristics for embankment and/or surface soils and untreated pavement materials. AASHTO T99 should be used for medium to high plasticity fine-grained soils, whereas AASHTO T180 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. Design tests (*e.g.,* resilient modulus, CBR, etc.) are also required on the compacted subgrade material.

Standards and Guidelines

Field exploration by borings should be guided by local practice, by applicable FHWA and state agency procedures, and by the AASHTO and ASTM standards listed in Table 4-12. Current copies of these standards and manuals should be maintained in the engineer's office for ready reference. The geotechnical engineer and field inspector should be thoroughly familiar with the contents of these documents, and should consult them whenever unusual subsurface situations arise during the field investigation. The standard procedures should always be followed; improvisation of investigative techniques may result in erroneous or misleading results that may have serious consequences on the interpretation of the field data.

Table 4-12. Frequently-used standards for field investigations.

Standard AASHTO	Standard ASTM	Title
M 146	C 294	Descriptive Nomenclature for Constituents of Natural Mineral Aggregates
T 86	D 420	Guide for Investigating and Sampling Soil and Rock
-	D 1195	Test Method for Repetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for Airport and Highway Pavements
-	D 1196	Test Method for Nonrepetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements
T 203	D 1452	Practice for Soil Investigation and Sampling by Auger Borings
T 206	D 1586	Standard Penetration Test and Split-Barrel Sampling of Soils
T 207	D 1587	Practice for Thin-Walled Tube Sampling of Soils
T 225	D 2113	Practice for Diamond Core Drilling for Site Investigation
M 145	D 2487	Test Method for Classification of Soils for Engineering Purposes
-	D 2488	Practice for Description and Identification of Soils (Visual-Manual Procedure)
T 223	D 2573	Test Method for Field Vane Shear Test in Cohesive Soil
-	D 3385	Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer
-	D 3550	Practice for Ring-Lined Barrel Sampling of Soils
-	D 4220	Practice for Preserving and Transporting Soil Samples
-	D 4428	Test Method for Crosshole Seismic Test
-	D 4544	Practice for Estimating Peat Deposit Thickness
-	D 4694	Test Method for Deflections with a falling-Weight-Type Impulse Load Device
-	D 4700	General Methods of Augering, Drilling, & Site Investigation
-	D 4719	Test Method for Pressuremeter Testing in Soils
-	D 4750	Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well)
-	D 5079	Practices for Preserving and Transporting Rock Core Samples
-	D 5092	Design and Installation of Ground Water Monitoring Wells in Aquifers
	D 5126	Guide for Comparison of Field Methods for Determining Hydraulic Conductivity in the Vadose Zone
-	D 5777	Guide for Seismic Refraction Method for Subsurface Investigation
-	D 5778	Test Method for Electronic Cone Penetration Testing of Soils
-	D 6391	Field Measurement of Hydraulic Conductivity Limits of Porous Materials Using Two Stages of Infiltration from a Borehole
-	D 6635	Procedures for Flat Plate Dilatometer Testing in Soils
-	D 6951	Test Method for Use of Dynamic Cone Penetrometer in Shallow Pavement Applications
-	G 57	Field Measurement of Soil Resistivity (Wenner Array)

4.5.7 Guidelines for Idealized Subsurface Exploration Program

The ideal exploration program would begin with remote sensing to survey the area for site access issues and to identify geologic formations and other features that would guide the selection and suitability of geophysical test methods. Next, geophysical testing would be performed using FWD as the principle tool, where possible, for back-calculation of resilient modulus values and/or profiling the site, thus, potentially reducing the number of borings required and the cost of laboratory testing. Resistivity would be used in conjunction with FWD to evaluate the extent of significant soil strata, and ground probing radar could be used to provide continuous thickness profiles for the pavement layers, as well as the location of groundwater. CPT or DCP would then be used to classify soil strata, obtain characteristic strength values, and confirm thickness profiles. This would be followed by limited borings and sampling, with some borings performed at several cone locations for calibration/verification, and at critical locations identified by the preceding methods. Again, disturbed samples are generally obtained to determine the soil type, gradation, classification, consistency, moisture-density relations (Proctor), CBR, presence of contaminants, stratification, etc. Undisturbed samples are obtained where necessary to determine the in-place stiffness and strength, compressibility (settlement), natural moisture content, unit weight, permeability, discontinuities, fractures, and fissures of subsurface formations.

The primary reason for following this idealized program is to develop a detailed understanding of subgrade and/or the existing unbound pavement layers that will impact design, construction, and the long-term performance of the pavement structure. There is also a cost implication for this program. Figure 4-15 provides an indication of relative cost for each phase. However, the reduced number of borings and sampling, and the improved reliability of the pavement system, should more than offset the cost of this program.

The Finnish roadway authority has fully integrated this approach into their pavement design. For example, to obtain an initial evaluation of the existing pavement section in rehabilitation and reconstruction projects, they use 1) GPR to provide an evaluation of the thickness of existing pavement components (using air-coupled antenna) and subgrade quality information (using ground-coupled antenna); 2) FWD to obtain the existing roadway support conditions; 3) roughness and rutting measurements; 4) pavement distress mapping; 5) GPS positioning; and reference drilling based on GPR results. The collected road survey data is processed, interpreted, analyzed, and classified, using Road Doctor^{TM} software specifically developed for this purpose, as shown in Figure 4-16a. Most recently, they have added resistivity surveys to evaluate moisture content. By combining technologies, they are able to develop a complete map of the subgrade system, including moisture (Figure 4-16b) and corresponding settlement profiles (Dumas et al., 2003). The analysis includes a classification of the critical elements affecting the lifetime of the road, including 1) overall pavement condition, 2) condition assessment of the unbound pavement structure, 3) road fatigue related to subgrade frost-action, 4) drainage condition, and 5) local damages, such as settlement of the road (Roimela et al, 2000). This information provides a better understanding of the causes of pavement distress and more precise rehabilitation measures for problem layers in the existing pavement system. Similar combinations of technology are used for the evaluation of subgrade conditions for new pavement design. This approach supports Finnish philosophy in pavement design, which presumes that any treatment to the subgrade should last from 60 – 100 years, the base and subbase should last from 30 – 50 years, and the surface should have a life of from 15 – 20 years. This sound philosophy is based on the relative cost of rehabilitation associated with each of these layers, and the importance of engineering in characterizing the soil and selecting material of the lower pavement layers.

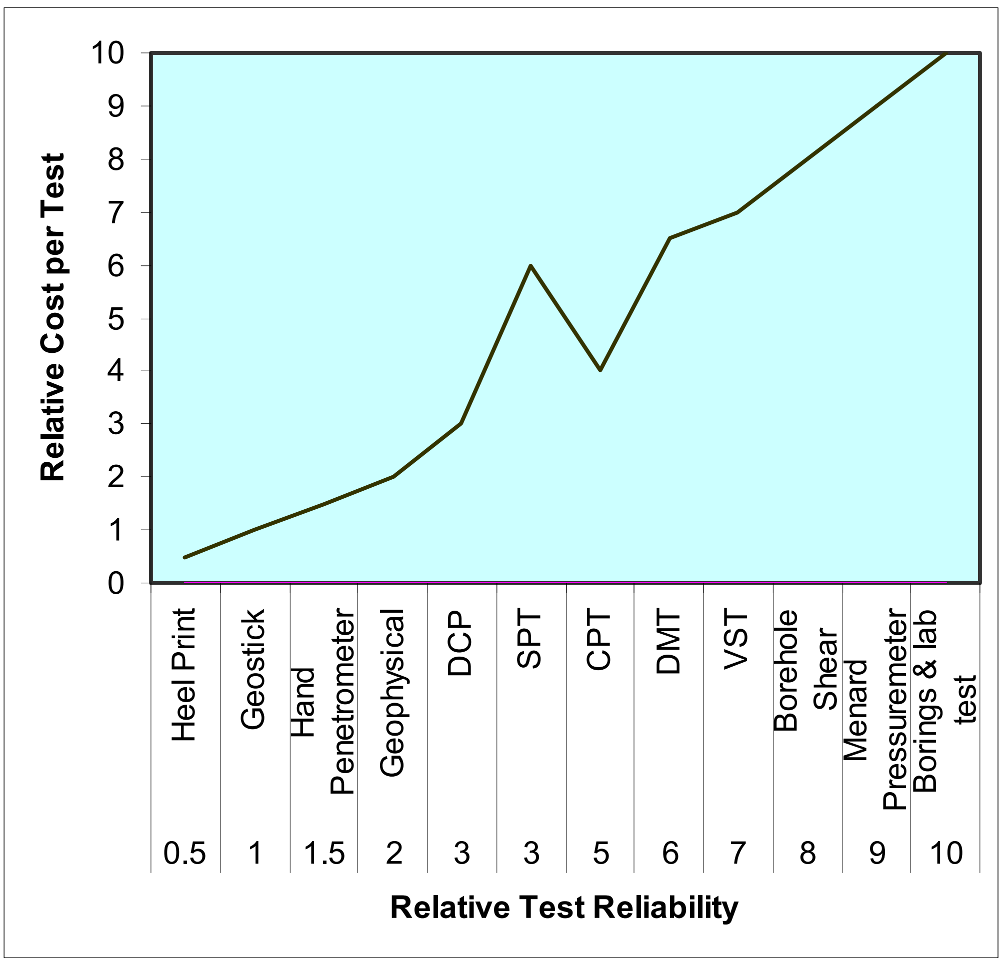
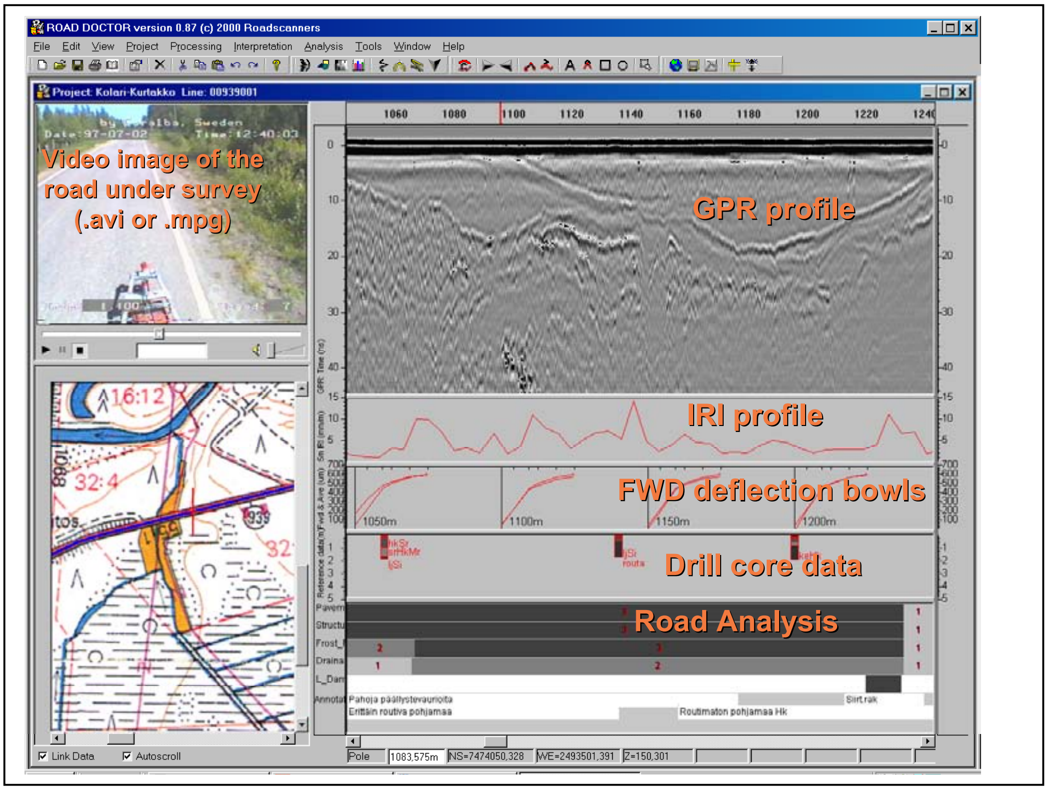
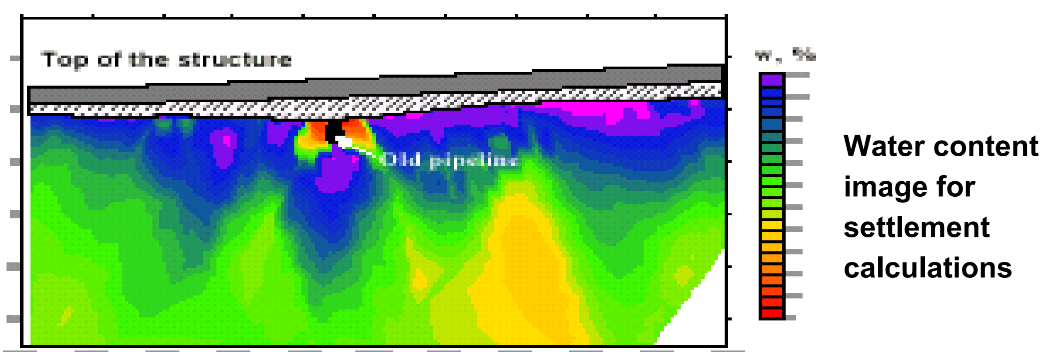


Figure 4-15. Qualitative relationship between relative subsurface exploration cost and reliability (after Handy, 1980).



a)



b)

Figure 4-16. Geophysical evaluation used by the Finnish National Road Administration for rehabilitation and reconstruction projects showing a) results from road analysis and b) moisture profile beneath the pavement (Tolla, 2002).

Texas DOT has recently developed a guideline, which supports the approach of using GPR and FWD data supported by DCP testing in the rehabilitation/reconstruction project evaluation process, as reported by Wimsatt and Scullion (2003). Computer programs have been developed to analyze the GPR and FWD data. GPR data is processed specifically to determine pavement layer thicknesses and the presence of excessive moisture or excessive air voids in pavement layers. FWD data is processed to generate remaining life estimates and pavement and subgrade layer moduli values. The DCP data is then used as required to verify the results of FWD data analysis, such as measuring base, subbase, and stiffness, or determining the depth to a stiff layer. Cores are generally collected at locations based on the GPR results (*e.g.,* in suspect areas).

4.6 IDENTIFY SOURCE FOR OTHER GEOTECHNICAL COMPONENTS

As indicated in section 4.1, the next subsurface exploration step is to evaluate conceptual designs and determine sources for other geotechnical components (*e.g.,* base and subbase materials). The requirements for subsurface drainage and subgrade stabilization, as well as construction material properties, should also be determined. Sampling of construction materials was discussed briefly in Section 4.5.6. The detailed requirements for these components will be covered in Chapter 7.

4.7 SUBGRADE CHACTERIZATION

The last step in the exploration process is to characterize the subgrade through 1) an evaluation of the field data, 2) performance of classification tests to support the field-identified subsurface stratigraphy, 3) develop stratigrahic profiles of the site, and 4) use that information to select representative soil layers for laboratory testing. Evaluation of the field data includes compiling and examining the stratigraphic information from the field investigation steps (*i.e.,* existing information, geophysical results, in-situ tests and borings), and the generation of final boring logs. The final logs are generated using classification tests to establish and support stratigraphy in relation to the design parameters. Soil profiles and plan views along the roadway alignment can then be created and examined to determine resilient modulus or other design testing requirements for each influential soil strata encountered.

4.7.1 Boring Logs

The boring log is the basic record of almost every geotechnical exploration and provides a detailed record of the work performed and the findings of the investigation. A boring log is a description of exploration procedures and subsurface conditions encountered during drilling, sampling, and coring. The field log should be written or printed legibly, and should be kept as clean as is practical.

Boring logs provide the basic information for the selection of test specimens. They provide background data on the natural condition of the formation, on the groundwater elevation, appearance of the samples, and the soil or rock stratigraphy at the boring location, as well as areal extent of various deposits or formations. The subsurface conditions observed in the soil samples and drill cuttings or perceived through the performance of the drill rig (for example, rig chatter in gravel, or sampler rebounding on a cobble during driving) should be described in the wide central column on the log labeled "Material Description," or in the remarks column, if available. The driller's comments are valuable and should be considered as the boring log is prepared. All appropriate portions of the logs should be completed in the field prior to completion of the field exploration. Following is a brief list of items, which should be included in the logs.

- Topographic survey data, including boring location and surface elevation, and bench mark location and datum, if available.
- An accurate record of any deviation in the planned boring locations.
- Identification of the subsoils and bedrock, including density, consistency, color, moisture, structure, geologic origin.
- For rehabilitation and reconstruction projects, an accurate thickness (+/- 2 mm {0.1 in.}) of each existing pavement layer should be carefully documented.
- The depths of the various generalized soil and rock strata encountered.
- Sampler type, depth, penetration, and recovery.
- Sampling resistance in terms of hydraulic pressure or blows per depth of sampler penetration. Size and type of hammer. Height of drop.
- Soil sampling interval and recovery.
- Rock core run numbers, depths/lengths, core recovery, and Rock Quality Designation (RQD).
- Type of drilling operation used to advance and stabilize the hole.
- Comparative resistance to drilling.
- Loss of drilling fluid.

- Water level observations with remarks on possible variations due to tides and river levels.
- The date/time that the borings are started, completed, and of water level measurements.
- Closure of borings.

A wide variety of drilling forms are used by various agencies, with some agencies using computerized logs entered on hand-held computers in the field. The specific forms to be used for a given type of boring will depend on local practice. A typical boring log is presented in Figure 4-17. A key or legend should be established by the agency for use by either in-house or outsource drilling in order to maintain uniformity in boring log preparation. A representative legend for soil boring logs and for core boring logs is included in Appendix D.

In addition to the description of individual samples, the boring log should also describe various strata. The record should include a description of each soil layer, with solid horizontal lines drawn to separate adjacent layers. Soil **description/identification** is the systematic, precise, and complete naming of individual soils in both written and spoken forms (ASTM D-2488, AASHTO M 145). During progression of a boring, the field personnel should only describe the soils encountered. Group symbols associated with classification should not be used in the field. Samples are later returned to the lab where samples may be classified. Soil **classification** is the grouping of the soil with similar engineering properties into a category based on index test results; *e.g.,* group name and symbol (ASTM D-2487, AASHTO M 145). A key part of classification of soil classification is the assignment of group symbols, which should only be assigned after supporting laboratory tests have been performed.

It is important to distinguish between visual identification (*i.e.,* a general visual evaluation of soil samples in the field) versus classification (*i.e.,* a more precise laboratory evaluation supported by index tests) in order to minimize conflicts between field and final boring logs. Some agencies have assigned symbols in the field based on visual observation and later corrected them on final boring logs based on lab tests. This practice leads to discrepancies between the field logs and the final logs that have on several occasions been successfully used to support contractor's claims in litigation procedures. In order to avoid these problems, it is recommended that group symbols not be included on field logs, but be reserved only for classification based on lab tests. Some states have avoided these problems by using lower-case symbols for field logs and upper-case symbols for lab-supported classification results, with the lower-case symbols clearly defined on the logs as based on visual observation only.

Project: Project Location: Project Number:	**Log of Boring ___** Sheet 1 of ___

Date(s) Drilled	Logged By	Checked By
Drilling Method	Drill Bit Size/Type	Total Depth Drilled (meters)
Drill Rig Type	Drilled By	Hammer Weight/ Drop (N/m)
Apparent Groundwater Depth ____ m ATD ___ m after ___ hrs ____ m after ___ hrs		Surface Elevation (meters)
Comments	Borehole Backfill	Elevation Datum

Depth, meters	SAMPLES Location	Type	Number	Sampling Resistance	MATERIAL DESCRIPTION and other remarks	Elevation, meters	Pocket Pen., kPa	Water Content, %	Liquid Limit	Plasticity Index	Other Tests
0											
1											
2											
3											
4											

Template: Proj ID: Printed:

Figure 4-17. Subsurface exploration log.

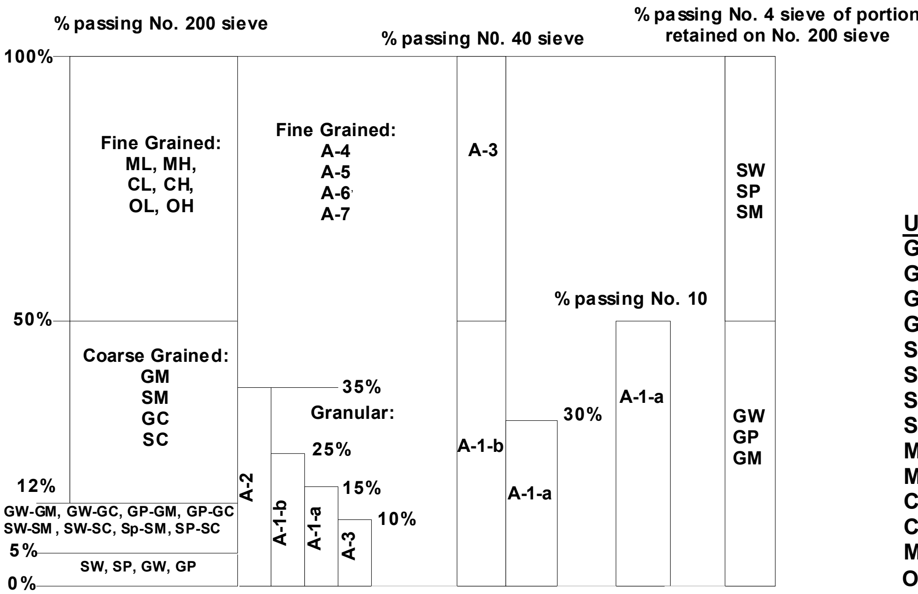
The stratigraphic observations should include identification of existing fill, topsoil, and pavement sections. Visual descriptions in the field are often subjected to outdoor elements, which may influence results. It is important to send the soil samples to a laboratory for accurate verification of visual identification, classification tests, and the assignment of appropriate group symbols, as discussed in the next section.

Data from the boring logs are combined with laboratory test results and other field information (*i.e.,* historical logs, soil survey and geological information, geophysical and in-situ tests) to identify subgrade profiles showing the extent and depth of various materials along the roadway alignment. Detailed boring logs, including the results of laboratory tests, are included in the geotechnical investigation report. Guidelines for completion of the boring log forms, preparation of soil descriptions and classifications, and preparation of rock descriptions and classifications are covered in detail in FHWA NHI-01-031, Subsurface Investigation manual.

4.7.2 Soil Classification

All soils should be taken to the laboratory and classified using the AASHTO (or Unified) soil classification system (see Figure 4-18). As previously indicated, final identification with classification can only be appropriately performed in the laboratory. This will lead to more consistent final boring logs and will avoid conflicts with field descriptions. The Unified Soil Classification System (USCS) Group Name and Symbol (in parenthesis) appropriate for the soil type in accordance with AASHTO M 145, ASTM D 3282, or ASTM D 2487 is the most commonly used system in geotechnical work and, more recently, highway subgrade material. It is covered in detail in this section. The AASHTO classification system has been often used for classification of highway subgrade material, and is shown in comparison with the USCS in Figures 4-18 and 4-19. While both methods are based on grain size and plasticity, USCS groups soils with similar engineering properties.

Table 4-13 provides an outline of the laboratory classification method. Table 4-14 relates the Unified soil classification of a material to the relative value of a material for use in a pavement structure.



Unified
GW: well graded gravel
GP: poorly graded gravel
GM: silty gravel
GC: clayey gravel
SW: well graded sand
SP: poorly graded sand
SM: silty sand
SC: clayey sand
ML: silt
MH: silt with high liquid limit
CL: clay
CH: clay with high liquid limit
ML-CL: silty clay
OL: organic silt or clay with low liquid limit
OH: organic silt or clay with high liquid limit

S - sand
G = gravel
M = silt
C = clay
W = well graded
L = low liquid limit (<50)
H = high liquid limit (>50)

AASHTO
A-1-a: mostly gravel or coarser with or without a well graded binder
A-1-b: coarse sand with or without a well graded binder
A-2: granular soils borderline between A-1 and A-3
A-3: fine sand with a small aount of nonplastic silt
A-4: silty soils
A-5: silty soils with high liquid limit
A-6: clayey soils
A-7: clayey soils

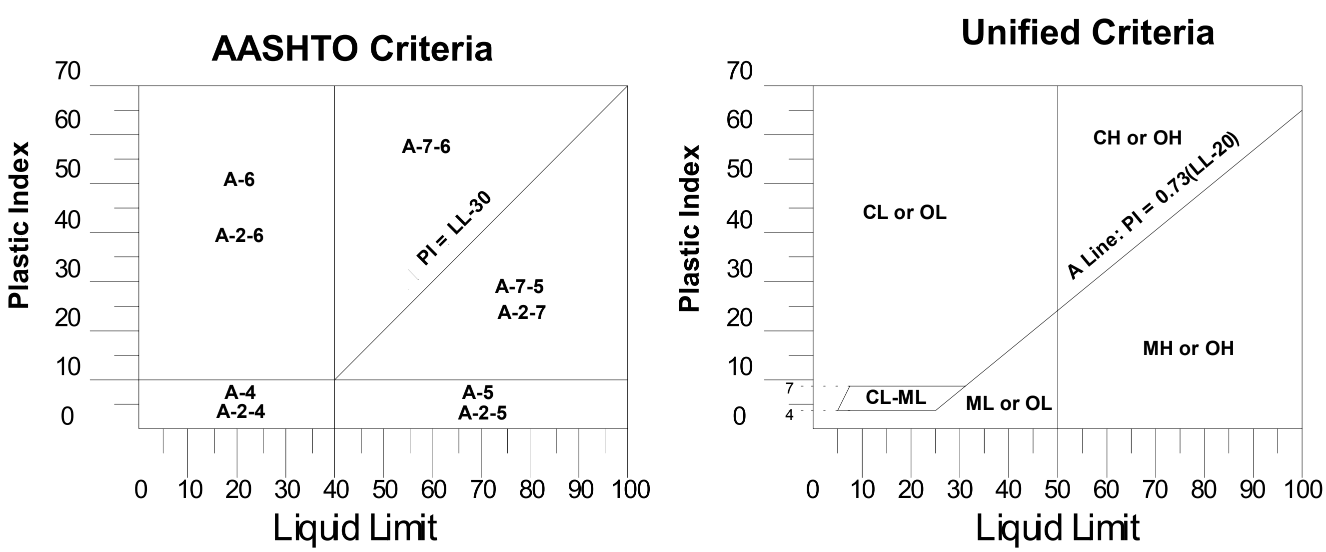


Figure 4-18. The AASHTO and the Unified Soil Classification System (after Utah DOT, 1998).

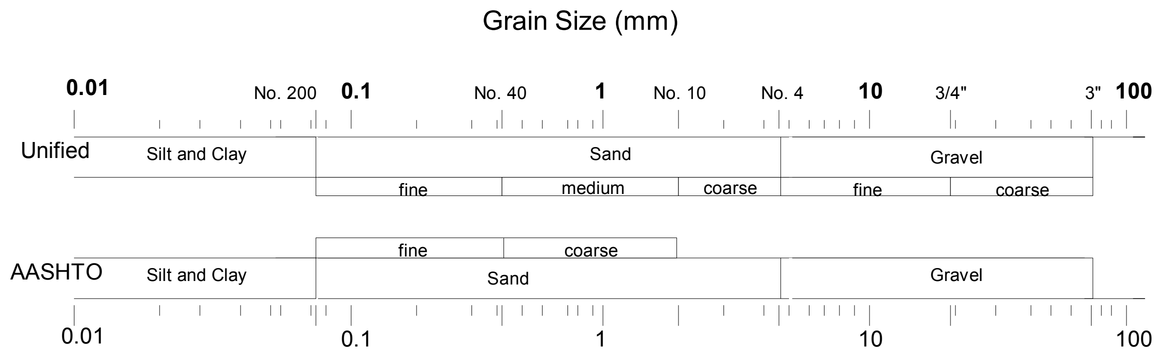


Figure 4–19. Particle size limit by different classifications systems.

Table 4-13. Classification of soils.

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[a]			Soil Classification	
			Group Symbol	**Group Name[b]**
GRAVELS More than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVELS Less than 5% fines	$C_U \geq 4$ and $1 \leq C_C \leq 3$[e]	GW	Well-graded Gravel
		$C_U \leq 4$ and $1 \geq C_C \geq 3$[e]	GP	Poorly-graded Gravel[f]
	GRAVELS WITH FINES More than 12% of fines[c]	Fines classify as ML or MH	GM	Silty Gravel[f,g,h]
		Fines classify as CL or CH	GC	Clayey Gravel[f,g,h]
SANDS 50% or more of coarse fraction retained on No. 4 sieve	CLEAN SANDS Less than 5% fines[d]	$C_U \geq 6$ and $1 \leq C_C \leq 3$[e]	SW	Well-graded Sand[i]
		$C_U \leq 6$ and $1 \geq C_C \geq 3$[e]	SP	Poorly-graded Sand[i]
	SANDS WITH FINES More than 12% fines[d]	Fines classify as ML or MH	SM	Silty Sand[g,h,i]
		Fines classify as CL or CH	SC	Clayey Sand[g,h,i]
SILTS AND CLAYS Liquid limit less than 50%	Inorganic	PI > 7 and plots on or above "A" line[j]	CL	Lean Clay[k,l,m]
		PI < 4 or plots below "A" line[j]	ML	Silt[k,l,m]
	Organic	$\frac{\textbf{Liquid limit - ovendried}}{\textbf{Liquid limit – not dried}} < \mathbf{0.75}$	OL	Organic Clay[k,l,m,n]
				Organic Silt[k,l,m,o]
SILTS AND CLAYS Liquid limit more than 50%	Inorganic	PI plots on or above "A" line	CH	Fat Clay[k,l,m]
		PI plots below "A" line	MH	Elastic Silt[k,l,m]
	Organic	$\frac{\textbf{Liquid limit - ovendried}}{\textbf{Liquid limit – not dried}} < \mathbf{0.75}$	OH	Organic Silt[k,l,m,p]
				Organic Silt[k,l,m,q]
Highly fibrous organic soils	Primary organic matter, dark in color, and organic odor		Pt	Peat and Muskeg

NOTES:

a. Based on the material passing the 75-mm sieve.
b. If field sample contained cobbles and/or boulders, add "with cobbles and/or boulders" to group name.
c. Gravels with 5 – 12% fines require dual symbols:
   - GW-GM well-graded gravel with silt
   - GW-GC well-graded gravel with clay
   - GP-GM poorly graded gravel with silt
   - GP-GC poorly graded gravel with clay

d. Sands with 5 – 12% fines require dual symbols:
   - SW-SM well-graded sand with silt
   - SW-SC well-graded sand with clay
   - SP-SM poorly graded sand with silt
   - SP-SC poorly graded sand with clay

e.

$$C_U = \frac{D_{60}}{D_{10}} = Uniformity\ Coefficient\ (also\ UC)$$

$$C_C = \frac{(D_{30})^2}{(D_{10})(D_{60})} = Coefficient\ of\ Curvature$$

f. If soil contains ≥ 15% sand, add "with sand" to group name.
g. If fines classify as CL-ML, use dual symbol GC-GM, SC-SM.
h. If fines are organic, add "with organic fines" to group name.
i. If soil contains ≥ 15% gravel, add "with gravel" to group name.
j. If the liquid limit & plasticity index plot in hatched area on plasticity chart, soil is a CL-ML, silty clay.
k. If soil contains 15 – 29% plus No. 200, add "with sand" or "with gravel", whichever is predominant.
l. If soil contains ≥ 30% plus No. 200, predominantly sand, add "sandy" to group name.
m. If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
n. Pl ≥ 4 and plots on or above "A" line.
o. Pl < 4 or plots below "A" line.
p. Pl plots on or above "A" line.
q. Pl plots below "A" line.

FINE-GRAINED SOILS (clays & silts): 50% or more passes the No. 200 sieve

COARSE-GRAINED SOILS (sands & gravels): more than 50% retained on No. 200 sieve

Table 4-14. Summary of soil characteristics as a pavement material. (from NCHRP 1-37A Pavement Design Guide)

Major Divisions		Name	Subgrade Strength when Not Subject to Frost Action	Potential Frost Action	Compressibility & Expansion	Drainage Characteristics
Gravel And Gravelly Soils	GW	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	None to very slight	Almost none	Excellent
	GP	Poorly graded gravels or gravel-sand mixtures, little or no fines	Good to excellent	None to very slight	Almost none	Excellent
	*d GM --- u	Silty gravels, gravel-sand silt mixtures	Good to excellent	Slight to medium	Very slight	Fair to poor
			Good	Slight to medium	Slight	Poor to practically impervious
	GC	Clayey gravels, gravel-sand-clay mixture	Good	Slight to medium	Slight	Poor to practically impervious
Sand and Sandy Soils	SW	Well-graded sands or gravelly sands, little or no fines	Good	None to very slight	Almost none	Excellent
	SP	Poorly graded sands or gravelly sands, little or no fines	Fair to good	None to very slight	Almost none	Excellent
	*d SM --- u	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
	SC	Clayey sands, sand-clay mixtures	Poor to fair	Slight to high	Slight to medium	Poor to practically impervious

*Division of GM and SM groups is based on Atterberg Limits (See Chapter 6) with suffix d used when L.L. is 28 or less and the PI is 6 or less. The suffix u is used when L.L. is greater that 28.

Table 4-14. Summary of soil characteristics as a pavement material (continued).

Major Divisions		Name	Subgrade Strength when Not Subject to Frost Action	Potential Frost Action	Compressibility & Expansion	Drainage Characteristics
Silts & Clays with Liquid Limit Less Than 50	ML	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
	CL	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
	OL	Organic silts & organic silt-clays or low plasticity	Poor	Medium to High	Medium to high	Poor
Silts & Clays with Liquid Limit Greater Than 50	MH	Inorganic silts, micaceous or diatomaceous fine sand or silty soils, elastic silts	Poor	Medium to Very High	High	Fair to Poor
	CH	Inorganic clays of high plasticity, fat clays	Poor to Fair	Medium to Very High	High	Practically Impervious
	OH	Organic clays of medium to high plasticity, organic silts	Poor to Very Poor	Medium	High	Practically Impervious
Highly Organic Soils	(Pt)	Peat & other highly organic soils	Not Suitable	Slight	Very high	Fair to Poor

4.7.3 Subsurface Profile

On the basis of all subsurface information (*i.e.,* from the literature review, geophysical evaluation, in-situ testing, soil borings, and laboratory test data), a subsurface profile can be developed. Longitudinal profiles are typically developed along the roadway alignment, and a limited number of transverse profiles may be included for key locations, such as at major bridge foundations, cut slopes, or high embankments. The subsurface information should also be presented in plan view, providing a map of general trends and changes in subsurface conditions. Vertical and plan view profiles provide an effective means of summarizing pertinent subsurface information and illustrating the relationship of the various investigation sites. By comparing the vertical profiles with the plan view, the subsurface conditions can be related to the site's topography and physiography, providing a sense of lateral distribution over a large horizontal extent. Subsurface profiles should be developed by a geotechnical engineer, as the preparation requires geotechnical judgement and a good understanding of the geologic setting for accurate interpretation of subsurface conditions between the investigation sites.

In developing a two-dimensional subsurface profile, the profile line (typically the roadway centerline) needs to be defined on the base plan, and the relevant borings, projected to this line. Judgment should be exercised in the selection of the borings since projection of the borings, even for short distances, may result in misleading representation of the subsurface conditions in some situations. **Due to the subjective nature of the interpretation required, subsurface profiles and plan views should not be included in either the subsurface investigation report or the construction bid documents**.

The subsurface profile should be presented at a scale appropriate to the depth of the borings, frequency of the borings and soundings, and overall length of the cross-section. Generally, an exaggerated scale of 1(V):10(H) or 1(V):20(H) should be used. A representative example of an interpreted subsurface profile is shown in Figure 4-20, and a plan view profile is shown in Figure 4-21. The subsurface profile can be presented with reasonable accuracy and confidence at the locations of the borings. Generally, however, owners and designers would like the geotechnical engineer to present a continuous subsurface profile that shows an interpretation of the location, extent, and nature of subsurface formations or deposits between borings. At a site where rock or soil profiles vary significantly between boring locations, the value of such presentations becomes questionable. The geotechnical engineer must be very cautious in presenting such data. Such presentations should include clear and simple caveats explaining that the profiles, as presented, cannot be fully relied upon. Should there be a need to provide highly reliable, continuous subsurface profiles, the geotechnical engineer should increase the frequency of borings and/or utilize geophysical methods to determine the continuity, or the lack of it, of subsurface conditions.

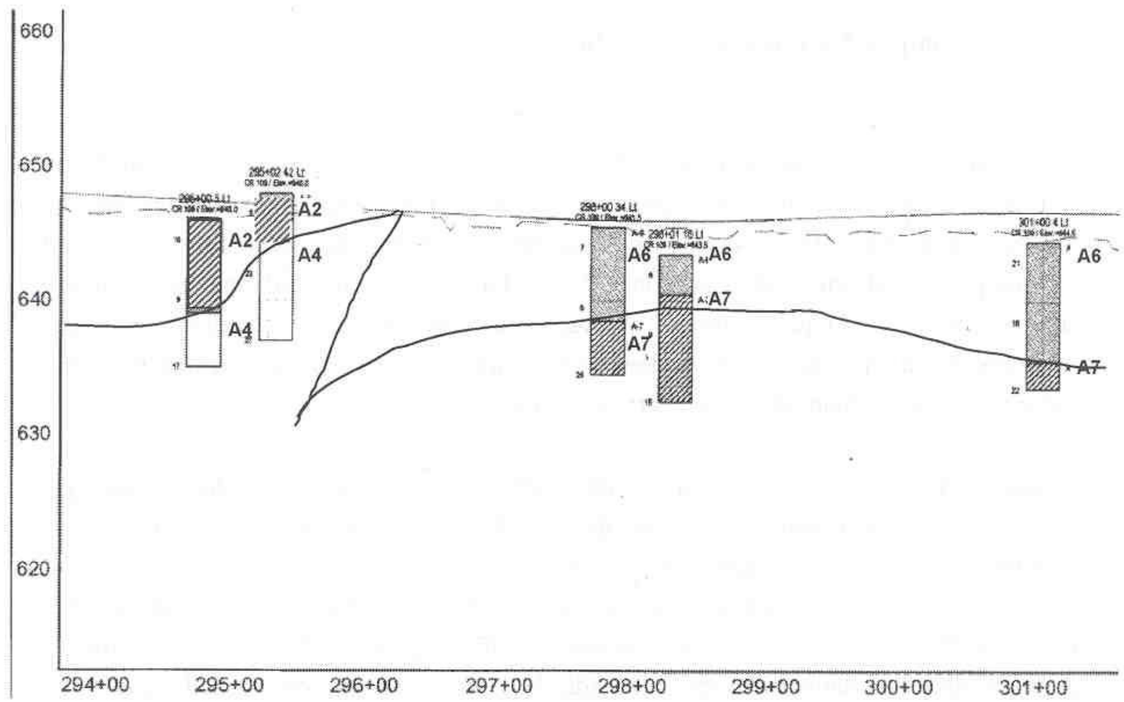


Figure 4-20. Subsurface profile based on boring data showing cross-sectional view.

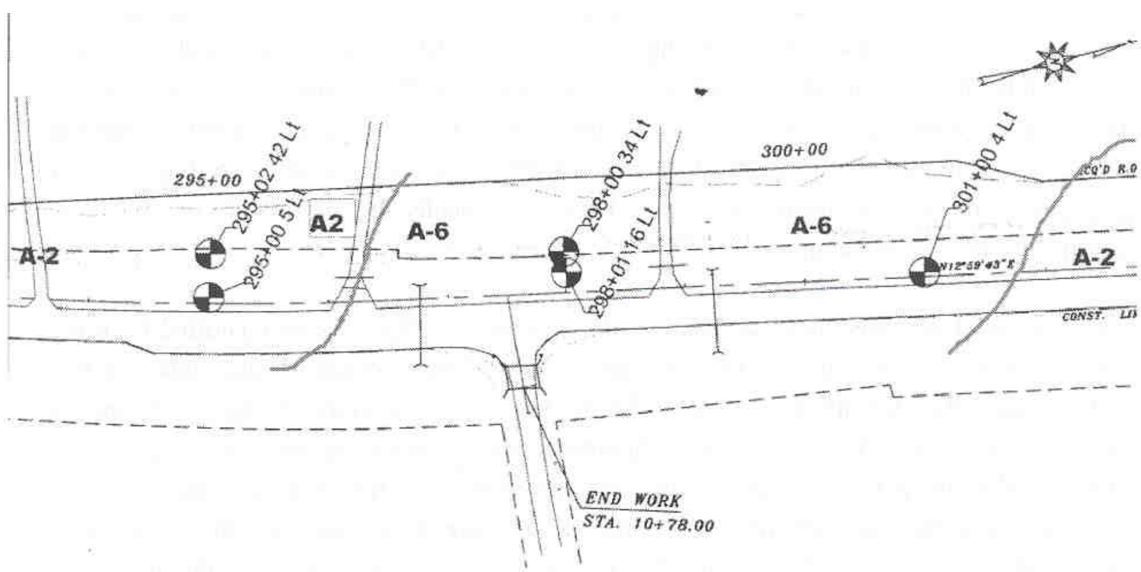


Figure 4-21. Plan view of subsurface information.

4.7.4 Select Samples for Laboratory Testing

A program of laboratory tests will be required 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 the 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 4-15. A deep cut or high embankment, as used in the table, general implies greater than a few meters (6 ft or more).

Table 4-15. Minimum laboratory testing requirements for pavement designs (NCHRP 1-37A Pavement Design Guide).

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

Representative soil layers are selected for laboratory testing by examining the boring logs, soil profiles, and classification tests. The primary test for design will be either resilient modulus tests, CBR, or other agency-specific design value, as outlined in Chapter 5, along with other properties required for each design level. Where possible, resilient modulus tests should be performed on undisturbed specimens that represent the natural conditions (moisture content and density) of the subgrade. For disturbed or reconstituted specimens, bulk materials should be recompacted to as close to the natural conditions as possible. For rehabilitation projects, the type of distress is also an important consideration, with engineering properties required for structural design of the selected rehabilitation strategy. These tests must also indicate the existing condition of the pavement and highlight any degradation that has taken place during the life of the pavement. Geophysical tests will significantly help in this effort. Tests to evaluate stabilization alternatives typically can be performed on material from disturbed, undisturbed, or bulk samples, prepared and compacted to the field requirements, as detailed in Chapter 7. Tests will also be required for constructability and performance. These tests can usually be performed on disturbed specimens and/or bulk samples.

The number of test specimens depends on the number of different soils identified from the borings, as well as the condition of those soils. The availability of geophysical and/or in-situ tests will also affect the number and type of tests. Most of the subgrade test specimens should be taken from as close to the top of the subgrade as possible, extending down to a depth of 0.6 m (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 or other design tests (*e.g.,* CBR) should be performed on any soil type that may have a detrimental impact on pavement performance (NCHRP 1-37A Pavement Design Guide). Other properties, such as shrink/swell and consolidation, will be required for evaluating stabilization requirements and long-term performance (*e.g.,* potential deformation).

For construction, as was discussed in section 4.5.6, moisture-density tests will be required on each soil type that will be used as fill in the pavement section, as well as the roadway embankment. AASHTO T99 should be used for medium to high plasticity fine-grained soils, whereas AASHTO T180 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. Design tests (*e.g.,* resilient modulus, CBR, etc.) are also required on the compacted subgrade material.

For rehabilitation projects, the number of tests will depend on the condition of the existing pavement. The condition survey as discussed in section 4.2.3, should be analyzed to show where problems may exist and require detailed material property information.

Another important point to remember in selecting the number of specimens to be tested is that the resilient modulus or other design value measured on different soils and soil structures (density, moisture) from repeated load tests can be highly variable. A coefficient of variation exceeding 25 percent for the resilient modulus on similar soils measured at the same stress-state is not uncommon. Repeatability studies indicate that coefficients of variation below 5 percent are not uncommon when testing replicated soil specimens (Boudreau, 2003). The potential high variability in test results requires increased testing frequencies (*i.e.,* many more than two or three resilient modulus tests along a project). As a general guide and suggested testing frequency, 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 (NCHRP 1-37A).

Student Exercise 4-1.

Boring logs and a stratagraphic profile from a proposed roadway alignment will be provided and the teams will be asked to 1) determine if the information is adequate, 2) evaluate method(s) for obtaining additional subsurface information, and 3) develop a laboratory testing program. One team will be randomly selected to present the results, followed by a solution discussion with the entire class.

Student Exercise 4-2.

The students will be asked to provide considerations regarding selection, assignment, and number of laboratory tests. Each item will be noted on a flip chart and, upon completion, reviewed with the list in the Reference Manual. The class will discuss the implications of not running the right test, running too many tests, and incorrectly running tests.

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CHAPTER 5.0 GEOTECHNICAL INPUTS FOR PAVEMENT DESIGN

5.1 INTRODUCTION

This chapter describes the determination of the specific geotechnical inputs required for the design of flexible and rigid pavements. Although the focus here is strictly on geotechnical inputs, there is obviously much other important information required for pavement design, including traffic characteristics, material properties for the bound asphalt and/or Portland cement concrete layers, desired reliability, and other details. These inputs are usually provided by agency units other than the geotechnical group.

Most of the inputs described in this chapter relate to the material properties of the unbound pavement layers and subgrade soil. Other required inputs include geometric information like layer thickness, but these are generally self-explanatory and are not discussed here. Environmental/climate inputs are also covered in this chapter. Although these inputs are not "geotechnical" *per se*, they directly influence the behavior of the unbound materials through their effects on moisture content and freeze/thaw cycles. In addition, in many agencies, the group responsible for determining the environmental inputs is poorly defined, and thus this responsibility may end up with the geotechnical group.

The coverage of the material in this chapter is guided by several considerations:

- Only the explicit design inputs are treated. As described in Chapter 3, there may be other geotechnical issues (*e.g.,* embankment slope stability) that can have a significant impact on pavement performance but that are not considered explicitly in the pavement design process.
- Project-specific measured input parameters are often unavailable at design time, particularly for preliminary design. This is especially true for material properties. Consequently, much emphasis is placed in this chapter on "typical" values and/or empirical correlations that can be used to estimate the design inputs. These estimates can be used for preliminary design, sensitivity studies, and other purposes. Clearly, though, project-specific measured values are preferred for final design.
- Many material property inputs can either be determined from laboratory or field tests. Field testing is covered in Chapter 4, and appropriate links to the Chapter 4 material are included here where appropriate.
- The treatment in this chapter attempts to balance coverage between the current empirical 1993 AASHTO Design Guide and the forthcoming mechanistic-empirical NCHRP 1-37A design approach (hereafter referred to as the NCHRP 1-37A Design Guide). Although there is some overlap in the geotechnical inputs required by these

two design approaches (*e.g.,* subgrade resilient modulus), there are substantial differences. The inputs to the 1993 AASHTO Guide are fewer in number and mostly empirical (*e.g.,* layer drainage coefficients), while the inputs to the NCHRP 1-37A Guide are more numerous and fundamental (*e.g.,* hydraulic conductivity vs. moisture content relations).

- Only design inputs are described in this chapter. In cases where some intermediate analysis is required to determine the design input (*e.g.,* for effective modulus of subgrade reaction in the 1993 Guide—see Section 5.4.6), the analysis methodology is described here, as well. The usage of the design inputs in the overall design calculations is described separately in Appendices C and D for the 1993 and NCHRP 1-37A Design Guides, respectively.

One consequence of all of the above is that this chapter is quite long; this is necessary to give sufficient coverage to all of the diverse geotechnical inputs required by the two design procedures. First, the geotechnical inputs required by the 1993 AASHTO and NCHRP 1-37A Design Guides are summarized (Section 5.2). Then, the geotechnical inputs are described in detail by category. The following is a road map of the sections in this chapter that describe the various categories of geotechnical design inputs:

5.2 REQUIRED GEOTECHNICAL INPUTS
- 5.2.1 1993 AASHTO Design Guide
- 5.2.2 NCHRP 1-37A Design Guide
- 5.2.3 Other Geotechnical Properties

5.3 PHYSICAL PROPERTIES
- 5.3.1 Weight-Volume Relationships
- 5.3.2 Physical Property Determination
- 5.3.3 Problem Soil Identification
- 5.3.4 Other Aggregate Tests

5.4 MECHANICAL PROPERTIES
- 5.4.1 California Bearing Ratio (CBR)
- 5.4.2 Stabilometer (R-Value)
- 5.4.3 Elastic (Resilient) Modulus
- 5.4.4 Poisson's Ratio
- 5.4.5 Structural Layer Coefficients
- 5.4.6 Modulus of Subgrade Reaction
- 5.4.7 Interface Friction
- 5.4.8 Permanent Deformation Characteristics
- 5.4.9 Coefficient of Lateral Pressure

5.5 THERMO-HYDRAULIC PROPERTIES
- 5.5.1 1993 AASHTO Guide
- 5.5.2 NCHRP 1-37A Design Guide

5.6 ENVIRONMENT/CLIMATE INPUTS

5.6.1 1993 AASHTO Guide
5.6.2 NCHRP 1-37A Design Guide

The chapter concludes with a section describing the development of final design values for each input when there are several estimates, e.g., material properties measured both in the field and in the laboratory. Most of the design inputs also exhibit significant spatial, temporal, and inherent variability. All of these issues must be reconciled to develop defensible input values for use in the final pavement design.

5.2 REQUIRED GEOTECHNICAL INPUTS

5.2.1 1993 AASHTO Design Guide

As described previously in Chapter 3, the AASHTO Pavement Design Guide has evolved through several versions over the 40+ years since the AASHO Road Test. The current version is the 1993 Guide. The geotechnical inputs required for flexible pavement design using the 1993 Guide are summarized in Table 5-1. Also shown are cross references to the sections in this manual in which the determination of the respective geotechnical inputs are described. As previously described in Chapter 3, the geotechnical inputs for the 1986 Guide are identical to those for the 1993 Guide. Note that the thicknesses D_i for the unbound layers are included as flexible pavement geotechnical inputs in Table 5-1; although these would typically be considered outputs from the design (*i.e.,* determined from *SN* and the other defined inputs), there may be cases where the layer thicknesses are fixed and for which the design then focuses on selecting layer materials having sufficient structural capacity.

The geotechnical inputs required for rigid pavement design using the 1993 Guide are summarized in Table 5-2. Again, these inputs are identical to those for the 1986 Guide. The first five properties in Table 5-2 are used to determine the effective modulus of subgrade reaction *k* in the 1993 Guide procedure. The geotechnical inputs required for rigid pavement design using the optional alternate approach in the 1998 supplement are the same as for the 1993 approach; only the analysis procedure changed in the 1998 supplement.

The last six parameters in both tables are the environmental parameters required by the 1993 Guide for determining the serviceability loss due to swelling of expansive subgrade soils and frost heave. Although these are not geotechnical parameters in the strictest sense, the detrimental effects of swelling and frost heave are concentrated in the subgrade and other unbound layers and thus are important geotechnical aspects of pavement design.

Table 5-1. Required geotechnical inputs for flexible pavement design using the 1993 AASHTO Guide.

Property	Description	Section
M_R	Resilient modulus of subgrade	5.4.3
E_{BS}	Resilient modulus of base (used to determine structural layer coefficient)	5.4.3
m_2	Moisture coefficient for base layer	5.5.1
D_2	Thickness of base layer	
E_{SB}	Resilient modulus of subbase (used to determine structural layer coefficient)	5.4.3
m_3	Moisture coefficient for subbase layer	5.5.1
D_3	Thickness of subbase layer	
θ	Swell rate	5.6.1
V_R	Maximum potential swell	5.6.1
P_S	Probability of swelling	5.6.1
ϕ	Frost heave rate	5.6.1
ΔPSI_{MAX}	Maximum potential serviceability loss from frost heave	5.6.1
P_F	Probability of frost heave	5.6.1

Note: Additional sets of layer properties (E_i, m_i, D_i) are required if there are more than two unbound layers in the pavement structure (exclusive of the natural subgrade).

Table 5-2. Required geotechnical inputs for rigid pavement design using the 1993 AASHTO Guide.

Property	Description	Section
M_R	Resilient modulus of subgrade	5.4.3
E_{SB}	Resilient modulus of subbase	5.4.3
D_{SB}	Thickness of subbase	
D_{SG}	Depth from top of subgrade to rigid foundation	
LS	Loss of Support factor	5.4.6
C_d	Drainage factor	5.5.1
F	Friction factor (for reinforcement design in JRCP)	5.4.7
θ	Swell rate	5.6.1
V_R	Maximum potential swell	5.6.1
P_S	Probability of swelling	5.6.1
ϕ	Frost heave rate	5.6.1
ΔPSI_{MAX}	Maximum potential serviceability loss from frost heave	5.6.1
P_F	Probability of frost heave	5.6.1

5.2.2 NCHRP 1-37A Design Guide

The mechanistic-empirical methodology that is the basis of the NCHRP 1-37A Design Guide requires substantially more input information than needed by the empirical design procedures in the 1993 AASHTO Guide. These inputs also tend to be more fundamental quantities, as compared to the often empirical inputs in the 1993 Guide. This is understandable given the inherent differences between mechanistic-empirical and empirical design methodologies.

Hierarchical Approach to Design Inputs

The level of design effort in any engineering design should be commensurate with the significance of the project being designed. Low-volume secondary road pavements do not require—and most agencies do not have the resources to provide—the same level of design effort as high-volume urban primary roads.

In recognition of this reality, a hierarchical approach has been developed for determining the pavement design inputs in the NCHRP 1-37A Design Guide. The hierarchical approach is based on the philosophy that the level of engineering effort exerted in determining the design inputs, including the material property values, should be consistent with the relative importance, size, and cost of the design project. Three levels are provided for the design inputs in the NCHRP 1-37A Guide:

> ***Level 1*** inputs provide the highest level of accuracy and the lowest level of uncertainty. Level 1 inputs would typically be used for designing heavily trafficked pavements or wherever there are serious safety or economic consequences of early failure. Level 1 material inputs require laboratory or field evaluation, such as resilient modulus testing or non-destructive deflection testing. Level 1 inputs require more resources and time to obtain than the other lower levels.
>
> ***Level 2*** inputs provide an intermediate level of accuracy and are closest to the typical procedures used with earlier editions of the AASHTO Pavement Design Guides. This level could be used when resources or testing equipment are not available for Level 1 characterization. Level 2 inputs would typically be derived from a limited testing program or estimated via correlations or experience (possibly from an agency database). Resilient modulus estimated from correlations with measured CBR values is one example of a Level 2 material input.
>
> ***Level 3*** inputs provide the lowest level of accuracy. This level might be used for designs in which there are minimal consequences of early failure (*e.g.,* low-volume roads). Level 3 material inputs typically are default values that are based on local

agency experience. A default resilient modulus based on AASHTO soil class is an example of a Level 3 material input.

Although it is intuitively clear that higher level (i.e., higher quality) design inputs will provide more precise estimates of pavement performance, the current state-of-the-art of pavement design and the limited availability of Level 1 input data make it difficult to quantify these benefits at present. One exception to this is thermal cracking prediction in the NCHRP 1-37A Design Guide. Complete Level 1 material property and environmental data were available from the U.S. and Canadian Strategic Highway Research Programs for approximately 35 pavement sites in the northern United States and Canada. Predictions of thermal cracking were made based on these Level 1 material inputs as well as on Level 3 default material properties. Figure 5-1 summarizes the differences between predicted and observed thermal cracking in units of lineal feet of cracking per 500 feet of pavement length for each of the field sites based on the Level 1 material inputs; Figure 5-2 summarizes the same results based on the Level 3 material inputs. The comparison of these two figures clearly shows that the higher quality Level 1 material inputs dramatically reduce the variability between predicted and observed cracking.

Design inputs in the NCHRP 1-37A methodology may be specified using a mix of levels for any given project. For example, the modulus of rupture of a concrete surface layer may be specified as a Level 1 input, while the traffic load spectra are determined using a Level 2 approach, and the subgrade resilient modulus via a Level 3 estimate based on subgrade soil class. The computational algorithms and distress models in the NCHRP 1-37A Design Guide (see Appendix D) are applied in the same way regardless of the input levels. However, the higher level inputs implicitly increase the accuracy and reliability of the predicted pavement performance.

In summary, the advantages of the hierarchical approach for the material and other design inputs are as follows:

- It provides the engineer with great flexibility in selecting an engineering approach consistent with the size, cost, and overall importance of the project.
- It allows each agency to develop an initial design methodology consistent with its internal technical capabilities.
- It provides a very convenient method for gradually increasing over time the technical skills and sophistication within the organization.
- In concept, it provides the most accurate and cost-efficient design consistent with agency financial and technical resources.

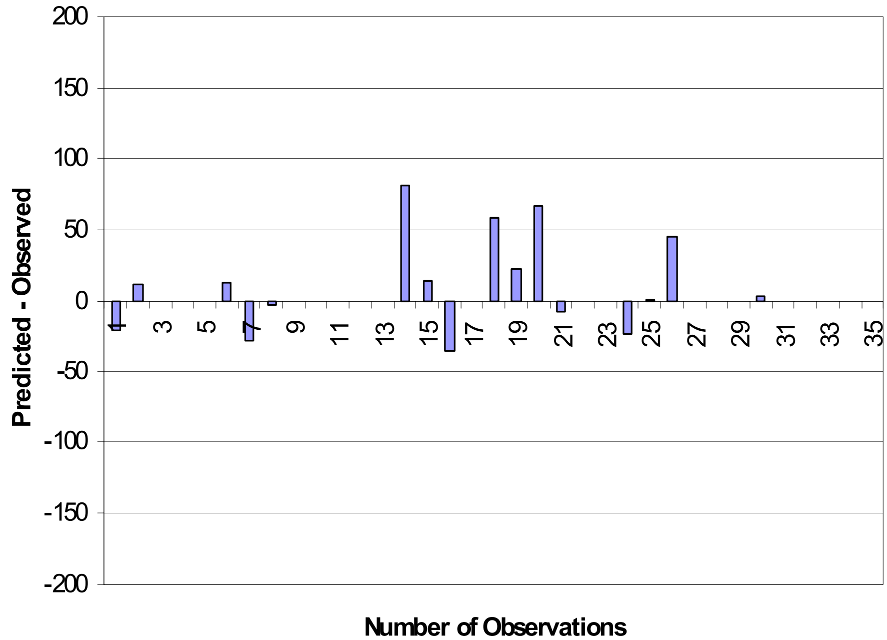


Figure 5-1. Thermal crack prediction from NCHRP 1-37A Design Guide using Level 1 material inputs.

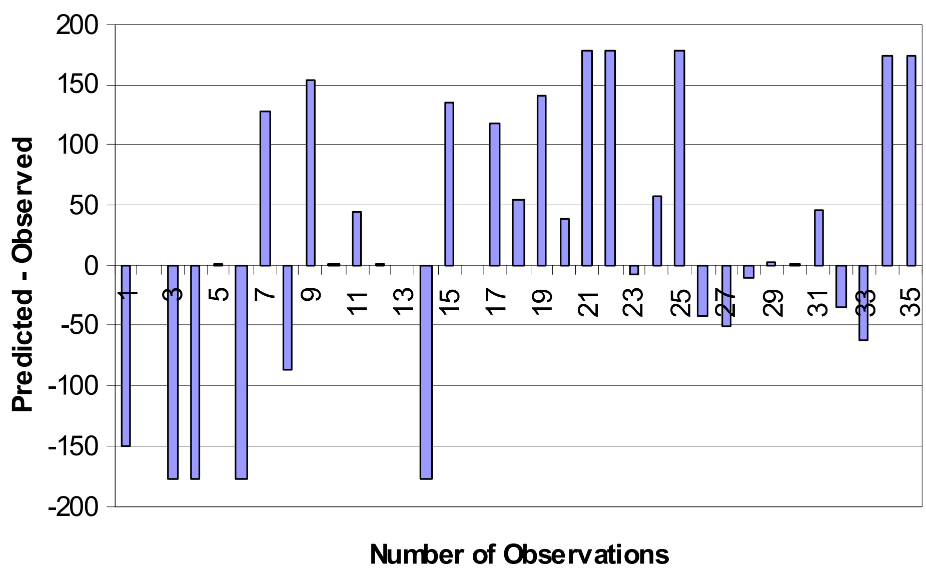


Figure 5-2. Thermal crack prediction from NCHRP 1-37A Design Guide using Level 3 material inputs.

Required Geotechnical Inputs

The geotechnical inputs for the NCHRP 1-37A Design Guide are organized into the following categories:

- *Mechanical* properties that are used in an analysis model to relate applied structural loads to structural response (Table 5-3 and Table 5-4).

- *Thermo-hydraulic* inputs that are used to relate environmental influences to the thermal and hydraulic state of the system (Table 5-5).

- *Distress model* properties that enter directly in the empirical models for pavement performance (Table 5-6).

As described previously, the NCHRP 1-37A Design Guide provides for three different hierarchical levels of input quality: Level 1 (highest), Level 2 (intermediate), and Level 3 (lowest). For any given input parameter, different properties may be required for Level 1 vs. Level 2 vs. Level 3 inputs. For example, a Level 1 estimate of subgrade resilient modulus for new construction requires laboratory-measured properties, while Level 2 instead requires CBR or other similar index properties, and Level 3 requires only the AASHTO or USCS soil class. The hierarchical levels for each of the geotechnical inputs are included in Table 5-3 through Table 5-6. The NCHRP 1-37A Guide recommends that the best available data (the highest level of inputs) be used for design. However, it does not require the same quality level for all inputs in the design.

Table 5-3. Geotechnical mechanical property inputs required for the flexible pavement design procedure in the NCHRP 1-37A Design Guide.

Property	Description	Level 1	Level 2	Level 3	Section
General					
	Material type	✓	✓	✓	3.3.2
γ_t	In-situ total unit weight	✓	✓	✓	
K_0	Coefficient of lateral earth pressure	✓	✓	✓	5.4.9
Stiffness/Strength of Subgrade and Unbound Layers[a]					
k_1, k_2, k_3	Nonlinear resilient modulus parameters	✓[b]			5.4.3
M_R	Backcalculated resilient modulus	✓[c]			5.4.3
M_R	Estimated resilient modulus		✓[d]	✓	5.4.3
CBR	California Bearing Ratio		✓[d]		5.4.1
R	R-Value		✓[d]		5.4.2
a_i	Layer coefficient		✓[d,e]		5.4.5
DCP	Dynamic Cone Penetration index		✓[d]		4.5.5
PI	Plasticity Index		✓[d]		5.3.2
P200	Percent passing 0.075 mm (No. 200 sieve)		✓[d]		5.3.2
	AASHTO soil class			✓	4.7.2
	USCS soil class			✓	4.7.2
ν	Poisson's ratio	✓	✓	✓	5.4.4
	Interface friction	✓	✓	✓	5.4.7

[a]Estimates of M_R and ν are also required for shallow bedrock.
[b]For new construction/reconstruction designs only.
[c]Primarily for rehabilitation designs.
[d]For level 2, M_R may be estimated directly or determined from correlations with one of the following: *CBR*; *R*; a_i; *DCP*; or *PI* and *P200*.
[e]For unbound base and subbase layers only.

Table 5-4. Geotechnical mechanical property inputs required for the rigid pavement design procedure in the NCHRP 1-37A Design Guide.

Property	Description	Level			Section
		1	2	3	
General					
	Material type	✓	✓	✓	3.3.2
γ_t	In-situ total unit weight	✓	✓	✓	5.3.2
K_0	Coefficient of lateral earth pressure	✓	✓	✓	5.4.9
Stiffness/Strength of Subgrade and Unbound Layers[a]					
$k_{dynamic}$	Backcalculated modulus of subgrade reaction	✓[b]			5.4.3
M_R	Estimated resilient modulus		✓[c]	✓	5.4.3
CBR	California Bearing Ratio		✓[c]		5.4.1
R	R-Value		✓[c]		5.4.2
a_i	Layer coefficient		✓[c]		5.4.5
DCP	Dynamic Cone Penetration index		✓[c]		4.5.5
PI	Plasticity Index		✓[c]		5.3.2
P200	Percent passing 0.075 mm (No. 200 sieve)		✓[c]		5.3.2
	AASHTO soil class			✓	4.7.2
	USCS soil class			✓	4.7.2
ν	Poisson's ratio	✓	✓	✓	5.4.4
	Interface friction	✓	✓	✓	5.4.7

[a]Estimates of M_R and ν are also required for shallow bedrock in new/reconstruction designs.
[b]From FWD testing for rehabilitation designs. For new/reconstruction designs, $k_{dynamic}$ is determined from Level 2 estimates of M_R.
[c]For Level 2, M_R may be estimated directly or determined from correlations with one of the following: *CBR*; *R*; a_i; *DCP*; or *PI* and *P200*.

Table 5-5. Thermo-hydraulic inputs required for the NCHRP 1-37A Design Guide.

Property	Description	Level 1	Level 2	Level 3	Section
	Groundwater depth	✓	✓	✓	5.5.2
Infiltration and Drainage					
	Amount of infiltration	✓	✓	✓	5.5.2
	Pavement cross slope	✓	✓	✓	5.5.2
	Drainage path length	✓	✓	✓	5.5.2
Physical Properties					
G_s	Specific gravity of solids	✓			5.3.2
$\gamma_{d\,max}$	Maximum dry unit weight	✓			5.3.2
w_{opt}	Optimum gravimetric water content	✓			5.3.2
PI	Plasticity Index		✓		5.3.2
D_{60}	Gradation coefficient		✓		5.3.2
$P200$	Percent passing 0.075 mm (No. 200 sieve)		✓		5.3.2
Hydraulic Properties					
a_f, b_f, c_f, h_r	Soil water characteristic curve parameters	✓			5.5.2
k_{sat}	Saturated hydraulic conductivity (permeability)	✓			5.5.2
PI	Plasticity Index		✓	✓	5.3.2
D_{60}	Gradation coefficient		✓	✓	5.3.2
$P200$	Percent passing 0.075 mm (No. 200 sieve)		✓	✓	5.3.2
Thermal Properties					
K	Dry thermal conductivity	✓			5.5.2
Q	Dry heat capacity	✓			5.5.2
	AASHTO soil class			✓	4.7.2

Table 5-6. Distress model material properties required for the NCHRP 1-37A Design Guide.

Property	Description	Level 1	Level 2	Level 3	Section
k_1	Rutting parameter (Tseng and Lytton model)	✓	✓	✓	5.4.8

5.2.3 Other Geotechnical Properties

In addition to the explicit design inputs listed in Table 5-1 and Table 5-2 for the 1993 AASHTO Guide and Table 5-3 through Table 5-6 for the NCHRP 1-37A Guide, other geotechnical properties are typically required during pavement design and construction. These include standard properties required for soil identification and classification, compaction control, and field QC/QA.

5.3 PHYSICAL PROPERTIES

Physical properties provide the most basic description of unbound materials. These properties are also often used in correlations for more fundamental engineering properties, such as stiffness or permeability. The principal physical properties of interest are specific gravity of solids, water content, unit weight (density), gradation characteristics, plasticity (Atterberg limits), classification, and compaction characteristics.

5.3.1 Weight-Volume Relationships

It is useful to review some common soil mechanics terminology and fundamental weight and volume relationships before describing the various soil test methods. Basic soil mechanics textbooks should be consulted for further explanation.

A sample of soil is a multi-phase material composed of solid soil grains, water, and air (Figure 5-3). The weight and volume of a soil sample depends on the specific gravity of the soil grains (solids), the size of the space between soil grains (voids and pores), and the amount of void space filled with water (moisture content and degree of saturation). Common terms associated with weight-volume relationships are shown in Table 5-7. Of particular note is the void ratio e, which is a general indicator of the relative strength and compressibility of a soil sample; *i.e.,* low void ratios generally indicate strong soils of low compressibility, while high void ratios are often indicative of weak and highly compressible soils. Selected weight-volume (unit weight) relations are presented in Table 5-8. Typical values for porosity, void ratio, water content, and unit weight are presented in Table 5-9 for a range of soil types.

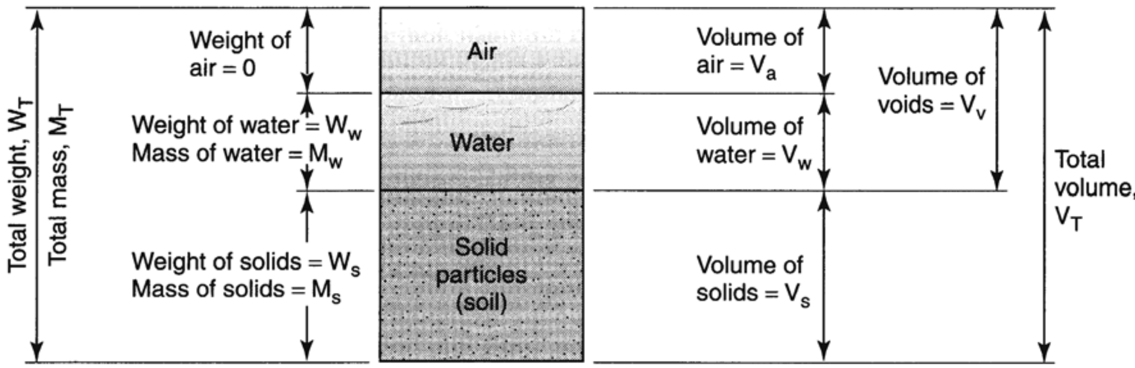


Figure 5-3. Relationships between volume and weight/mass of bulk soil (McCarthy, 2002).

Table 5-7. Terms in weight-volume relations (after Cheney and Chassie, 1993).

Property	Symbol	Units[1]	How obtained (AASHTO/ASTM)	Direct Applications
Moisture Content	w	D	By measurement (T 265/ D 4959)	Classification and weight-volume relations
Specific Gravity	G_s	D	By measurement (T 100/D 854)	Volume computations
Unit Weight	γ	FL^{-3}	By measurement or from weight-volume relations	Classification and pressure computations
Porosity	n	D	From weight-volume relations	Defines relative volume of solids to total volume of soil
Void Ratio	e	D	From weight-volume relations	Defines relative volume of voids to volume of solids

[1] F = Force or weight; L = Length; D = Dimensionless. Although by definition, moisture content is a dimensionless fraction (ratio of weight of water to weight of solids), it is commonly reported in percent by multiplying the fraction by 100.

Table 5-8. Unit weight-volume relationships.

Case	Relationship	Applicable Geomaterials
Soil Identities:	1. $G_s\, w = S\, e$ 2. Total Unit Weight: $\gamma_t = \dfrac{(1+w)}{(1+e)} G_s\, \gamma_w$	All types of soils & rocks
Limiting Unit Weight	Solid phase only: $w = e = 0$: $\gamma_{rock} = G_s\, \gamma_w$	Maximum expected value for solid silica is 27 kN/m^3
Dry Unit Weight	For $w = 0$ (all air in void space): $\gamma_d = G_s\, \gamma_w/(1+e)$	Use for clean sands and soils above groundwater table
Moist Unit Weight (Total Unit Weight)	Variable amounts of air & water: $\gamma_t = G_s\, \gamma_w\, (1+w)/(1+e)$ with $e = G_s\, w/S$	Partially-saturated soils above water table; depends on degree of saturation (S, as decimal).
Saturated Unit Weight	Set $S = 1$ (all voids with water): $\gamma_{sat} = \gamma_w\, (G_s+e)/(1+e)$	All soils below water table; Saturated clays & silts above water table with full capillarity.
Hierarchy:	$\gamma_d \leq \gamma_t \leq \gamma_{sat} < \gamma_{rock}$	Check on relative values

Note: γ_w = 9.8 kN/m^3 (62.4 pcf) for fresh water.

Table 5-9. Typical porosity, void ratio, and unit weight values for soils in their natural state (after Peck, Hanson, and Thornburn, 1974).

Soil Type	Porosity n	Void Ratio e	Water Content w	Unit Weight kN/m^3 γ_d	Unit Weight kN/m^3 γ_{sat}	Unit Weight lb/cu ft γ_d	Unit Weight lb/cu ft γ_{sat}
Uniform sand (loose)	0.46	0.85	32%	14.1	18.5	90	118
Uniform sand (dense)	0.34	0.51	19%	17.1	20.4	109	130
Well-graded sand (loose)	0.40	0.67	25%	15.6	19.5	99	124
Well-graded sand (dense)	0.30	0.43	16%	18.2	21.2	116	135
Windblown silt (loess)	0.50	0.99	21%	13.4	18.2	85	116
Glacial till	0.20	0.25	9%	20.7	22.8	132	145
Soft glacial clay	0.55	1.2	45%	11.9	17.3	76	110
Stiff glacial clay	0.37	0.6	22%	16.7	20.3	106	129
Soft slightly organic clay	0.66	1.9	70%	9.1	15.4	58	98
Soft very organic clay	0.75	3.0	110%	6.8	14.0	43	89
Soft montmorillonitic clay	0.84	5.2	194%	4.2	12.6	27	80

5.3.2 Physical Property Determination

Laboratory and field methods (where appropriate) for determining the physical properties of unbound materials in pavement systems are described in the following subsections and tables. Typical values for each property are also summarized. The soil physical properties are organized into the following categories:

- Volumetric properties
  - Specific gravity (Table 5-10)
  - Moisture content (Table 5-11)
  - Unit weight (Table 5-12)

- Compaction
  - Proctor compaction tests (Table 5-13)

- Gradation
  - Mechanical sieve analysis (Table 5-19)
  - Hydrometer analysis (Table 5-20)

- Plasticity
  - Atterberg limits (Table 5-21)

Gradation and plasticity are the principle determinants for engineering soil classification using either the AASHTO or Unified soil classification systems. Soil classification is described as part of subsurface exploration in Section 4.7.2.

The identification of problem soils (*e.g.,* expansive clays) is typically based on their physical properties; this topic is addressed at the end of this section. Other additional tests commonly used for quality control of aggregates used in base and subbase layers and in asphalt and Portland cement concrete are also briefly summarized.

Volumetric Properties

The volumetric properties of most interest in pavement design and construction are

- Specific gravity (Table 5-10)
- Moisture content (Table 5-11)
- Unit weight (Table 5-12)

Table 5-10. Specific gravity of soil and aggregate solids.

<table>
<tr><td>Description</td><td>The specific gravity of soil solids G_s is the ratio of the weight of a given volume of soil solids at a given temperature to the weight of an equal volume of distilled water at that temperature</td></tr>
<tr><td>Uses in Pavements</td><td>• Calculation of soil unit weight, void ratio, and other volumetric properties (see Section 5.3.1).
• Analysis of hydrometer test for particle distribution of fine-grained soils (Table 5-20).</td></tr>
<tr><td>Laboratory Determination</td><td>AASHTO T 100 or ASTM D 854.</td></tr>
<tr><td>Field Measurement</td><td>Not applicable.</td></tr>
<tr><td>Commentary</td><td>Some qualifying words like true, absolute, apparent, bulk or mass, etc. are sometimes added to "specific gravity." These qualifying words modify the sense of specific gravity as to whether it refers to soil grains or to soil mass. The soil grains have permeable and impermeable voids inside them. If all the internal voids of soil grains are excluded for determining the true volume of grains, the specific gravity obtained is called absolute or true specific gravity (also called the apparent specific gravity). If the internal voids of the soil grains are included, the specific gravity obtained is called the bulk specific gravity.
Complete de-airing of the soil-water mix during the test is imperative while determining the true or absolute value of specific gravity.</td></tr>
<tr><td>Typical Values (Coduto ,1999)</td><td><table><tr><th>Soil Type</th><th>G_S</th></tr><tr><td>Clean, light colored sand (quartz, feldspar)</td><td>2.65</td></tr><tr><td>Dark colored sand</td><td>2.72</td></tr><tr><td>Sand-silt-clay mixtures</td><td>2.72</td></tr><tr><td>Clay</td><td>2.65</td></tr></table></td></tr>
</table>

Table 5-11. Moisture content.

Description	The moisture content expresses the amount of water present in a quantity of soil. The gravimetric moisture or water content w is defined in terms of soil weight as $w = W_w / W_s$, where W_w is the weight of water and W_s is the weight of the soil solids in the sample.
Uses in Pavements	• Calculation of soil total unit weight, void ratio, and other volumetric properties (see Section 5.3.1). • Correlations with soil behavior, other soil properties.
Laboratory Determination	Drying of the soil in a conventional (temperature of 110±5°C) or microwave oven to a constant weight (AASHTO T 265, ASTM D 2216/conventional oven, or ASTM D 4643/microwave).
Field Measurement	Nuclear gauge (ASTM D2922).
Commentary	Determination of the moisture or water content is one of the most commonly performed laboratory procedures for soils. The water content of soils, when combined with data obtained from other tests, produces significant information about the characteristics of the soil. For example, when the in-situ water content of a sample retrieved from below the groundwater table approaches its liquid limit, it is an indication that the soil in its natural state is susceptible to larger consolidation settlement. For fluid flow applications, the moisture content is often expressed as the volumetric moisture content $\theta = V_w / V_t$, where V_w is the volume of water and V_t is the total volume of the sample. Volumetric moisture content can also be determined as $\theta = Sn$, where S is the saturation and n is the porosity.
Typical Values	See Table 5-9. For dry soils, $w \cong 0$. For most natural soils, $3 \leq w \leq 70\%$. Saturated fine-grained and organic soils may have gravimetric moisture contents in excess of 100%.

Table 5-12. Unit weight.

Description	The unit weight is the total weight divided by total volume for a soil sample.
Uses in Pavements	• Calculation of in-situ stresses. • Correlations with soil behavior, other soil properties. • Compaction control (see ***Compaction*** subsection).
Laboratory Determination	The unit weight for undisturbed fine-grained soil samples is measured in the laboratory by weighing a portion of a soil sample and dividing by its volume. This can be done with thin-walled tube (Shelby) samples, as well as piston, Sherbrooke, Laval, and NGI samplers. Where undisturbed samples are not available (*e.g.,* for coarse grained soils), the unit weight must be evaluated from weight-volume relationships (see Table 5-8).
Field Measurement	Nuclear gauge (ASTM D2922), sand cone (ASTM D1556).
Commentary	Unit weight is also commonly termed *density*. The total unit weight is a function of the moisture content of the soil (Table 5-8). Distinctions must be maintained between dry (γ_d), saturated (γ_{sat}), and moist or total (γ_t) unit weights. The moisture content should therefore be obtained at the same time as the unit weight to allow conversion from total to dry unit weights.
Typical Values	See Table 5-9.

Compaction

Soil compaction is one of the most important geotechnical concerns during the construction of highway pavements and related fills and embankments. Compaction improves the engineering properties of soils in many ways, including

- increased elastic stiffness, which reduces short-term resilient deformations during cyclic loading.
- decreased compressibility, which reduces the potential for excessive long-term settlement.
- increased strength, which increases bearing capacity and decreases instability potential (*e.g.,* for slopes).
- decreased hydraulic conductivity (permeability), which inhibits flow of water through the soil.
- decreased void ratio, which reduces the amount of water that can be held in the soil and, thus, helps maintain desired strength and stiffness properties.
- dncreased erosion resistance.

Compaction is usually quantified in terms of the equivalent dry unit weight γ_d of the soil as a measure of the amount of solid materials present in a unit volume. The higher the amount of solid materials, the stronger and more stable the soil will be. Standard laboratory testing (Table 5-13) involves compacting several specimens at different water contents (w). The total unit weight (γ_t) and water content are measured for each compacted specimen. The equivalent dry unit weight is then computed as

$$\gamma_d = \frac{\gamma_t}{1+w} \tag{5.1}$$

If the specific gravity of solids G_s is known, the saturation level (S) and void ratio (e) can also be determined using the following two identities:

$$G_s\, w = S\, e \tag{5.2}$$

$$\gamma_t = \frac{G_s \gamma_w (1+w)}{(1+e)} \tag{5.3}$$

The pairs of equivalent dry weight vs. water content values are plotted as a moisture-density of compaction curve, as in Figure 5-4. Compaction curves will typically exhibit a well defined peak corresponding to the maximum dry unit weight ($(\gamma_d)_{max}$) at an optimum moisture content (w_{opt}). It is good practice to plot the zero air voids (ZAV) curve corresponding to 100 percent saturation on the moisture-density graph (see Figure 5-4). The measured compaction curve cannot fall above the ZAV curve if the correct specific gravity has been used. The peak or maximum dry unit weight usually corresponds to saturation levels of between 70 – 85 percent.

Relative compaction (C_R) is the ratio (expressed as a percentage) of the density of compacted or natural in-situ soils to the maximum density obtainable in a specified compaction test:

$$C_R = \frac{\gamma_d}{(\gamma_d)_{\max}} \times 100\% \tag{5.4}$$

Specifications often require a minimum level of relative compaction (*e.g.,* 95%) in the construction or preparation of foundations, subgrades, pavement subbases and bases, and embankments. Requirements for compacted moisture content relative to the optimum moisture content may also be included in compaction specifications. The design and selection of methods to improve the strength and stiffness characteristics of deposits depend heavily on relative compaction.

Relative density (D_R) (ASTM D 4253) is often a useful parameter in assessing the engineering characteristics of granular soils. It is defined as

$$D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \times 100\% \tag{5.5}$$

in which $e_{\min}$ and $e_{\max}$ are the minimum and maximum void ratio values for the soil. Relative density can also be expressed in terms of dry unit weights:

$$D_r = \left[\frac{\gamma_d - (\gamma_d)_{\min}}{(\gamma_d)_{\max} - (\gamma_d)_{\min}}\right]\left[\frac{(\gamma_d)_{\max}}{\gamma_d}\right] \times 100\% \tag{5.6}$$

Table 5-14 presents a classification of soil consistency based on relative density for granular soils.

Table 5-13. Compaction characteristics.

Description	Compaction characteristics are expressed as the equivalent dry unit weight vs. moisture content relationship for a soil at a given compaction energy level. Of particular interest are the maximum equivalent dry unit weight and corresponding optimum moisture content at a given compaction energy level.
Uses in Pavements	• In conjunction with other tests (*e.g.,* resilient modulus), determines influence of soil density on engineering properties. • Field QC/QA for compaction of natural subgrade, placed subbase and base layers, and embankment fills.
Laboratory Determination	Two sets of test protocols are most commonly used: • AASHTO T 99 (Standard Proctor), T 180 (Modified Proctor) • ASTM D 698 (Standard Proctor), D 1557 (Modified Proctor) Compaction tests are performed using disturbed, prepared soils with or without additives. Normally, soil passing the No. 4 sieve is mixed with water to form samples at various moisture contents ranging from the dry state to wet state. These samples are compacted in layers in a mold by a hammer at a specified nominal compaction energy that is a function of the number of layers, hammer weight, drop height, and number of blows (see Table 5-15). Equivalent dry unit weight is determined based on the moisture content and the unit weight of compacted soil. A curve of dry unit weight versus moisture content is plotted (Figure 5-4) and the maximum ordinate on this curve is referred to as the maximum dry unit weight ($(\gamma_d)_{max}$). The water content at which this maximum occurs is termed as the optimum moisture content (w_{opt}) or OMC.
Field Measurement	Field determination of moisture content (Table 5-11) and unit weight (Table 5-12) is used to check whether field-compacted material meets construction specifications.
Commentary	Where a variety of soils are to be used for construction, a moisture-density relationship for each major type of soil or soil mixture anticipated at the site should be established. When additives such as Portland cement, lime, or flyash are used to determine the maximum density of mixed compacted soils in the laboratory, care should be taken to duplicate the expected delay period between mixing and compaction in the field. It should be kept in mind that these chemical additives start reacting as soon as they are added to the wet soil. They cause substantial changes in soil properties, including densities achievable by compaction. The period between mixing and compaction in the field is expected to be three hours, for example, then in the laboratory the compaction of the soil should also be delayed three hours after mixing the stabilizing additives.
Typical Values	See Table 5-16 for AASHTO recommended minimum compaction levels. Typical ranges for compacted unit weight and optimum moisture content for USCS and AASHTO soil classes are summarized in Table 5-17 and Table 5-18, respectively.

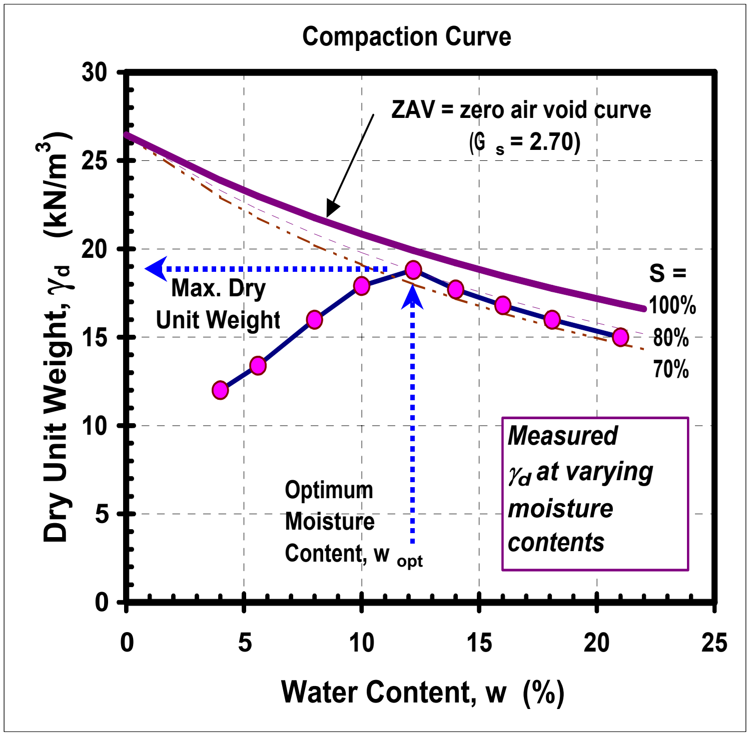


Figure 5-4. Typical moisture-density relationship from a standard compaction test.

Table 5-14. Consistency of granular soils at various relative densities.

Relative Density D_r (%)	Description
85 – 100	Very dense
65 – 85	Dense
35 – 65	Medium dense
15 – 35	Loose
0 - 15	Very loose

Table 5-15. Principal differences between standard and modified Proctor tests.

	Standard Proctor	Modified Proctor
Standards	AASHTO T 99 ASTM D 698	AASHTO T 180 ASTM D 1557
Hammer weight	5.5 lb (24.4 kN)	10.0 lb (44.5 kN)
Hammer drop height	12 in (305 mm)	18 in (457 mm)
Number of soil layers	3	5
Hammer blows per layer	25	25
Total compaction energy	12,400 ft-lb/ft^3 (600 kN-m/m^3)	56,000 ft-lb/ft^3 (2,700 kN-m/m^3)

Table 5-16. Recommended minimum requirements for compaction of embankments and subgrades (AASHTO, 2003).

AASHTO Soil Class	Minimum Percent Compaction (%)[a]		
	Embankments		Subgrades
	< 50 ft. high	> 50 ft. high	
A-1, A-3	≥ 95	≥ 95	100
A-2-4, A-2-5	≥ 95	≥ 95	100
A-2-6, A-2-7	≥ 95	--[b]	≥ 95[c]
A-4, A-5, A-6, A-7	≥ 95	--[b]	≥ 95[c]

[a]Based on standard Proctor (AASHTO T 99).
[b]Special attention to design and construction is required for these materials.
[c]Compaction at within 2% of the optimum moisture content.

Table 5-17. Typical compacted densities and optimum moisture contents for USCS soil types (after Carter and Bentley, 1991).

Soil Description	USCS Class	Compacted Dry Unit Weight		Optimum Moisture Content (%)
		(lb/ft^3)	(kN/m^3)	
Gravel/sand mixtures:				
well-graded, clean	GW	125-134	19.6-21.1	8-11
poorly-graded, clean	GP	115-125	18.1-19.6	11-14
well-graded, small silt content	GM	119-134	18.6-21.1	8-12
well-graded, small clay content	GC	115-125	18.1-19.6	9-14
Sands and sandy soils:				
well-graded, clean	SW	109-131	17.2-20.6	9-16
poorly-graded, small silt content	SP	94-119	15.7-18.6	12-21
well-graded, small silt content	SM	109-125	17.2-19.6	11-16
well-graded, small clay content	SC	106-125	16.7-19.6	11-19
Fined-grained soils of low plasticity:				
silts	ML	94-119	14.7-18.6	12-24
clays	CL	94-119	14.7-18.6	12-24
organic silts	OL	81-100	12.7-15.7	21-33
Fine-grained soils of high plasticity:				
silts	MH	69-94	10.8-14.7	24-40
clays	CH	81-106	12.7-18.6	19-36
organic clays	OH	66-100	10.3-15.7	21-45

Table 5-18. Typical compacted densities and optimum moisture contents for AASHTO soil types (after Carter and Bentley, 1991).

Soil Description	AASHTO Class	Compacted Dry Unit Weight		Optimum Moisture Content (%)
		(lb/ft^3)	(kN/m^3)	
Well-graded gravel/sand mixtures	A-1	115-134	18.1-21.1	5-15
Silty or clayey gravel and sand	A-2	109-134	17.2-21.1	9-18
Poorly-graded sands	A-3	100-119	15.7-18.6	5-12
Low plasticity silty sands and gravels	A-4	94-125	14.7-19.6	10-20
Diatomaceous or micaceous silts	A-5	84-100	13.2-15.7	20-35
Plastic clay, sandy clay	A-6	94-119	14.7-18.6	10.30
Highly plastic clay	A-7	81-115	12.7-18.1	15-35

Gradation

Gradation, or the distribution of particle sizes within a soil, is an essential descriptive feature of soils. Soil textural (*e.g.,* gravel, sand, silty clay, etc.) and engineering (see Section 4.7.2) classifications are based in large part on gradation, and many engineering properties like permeability, strength, swelling potential, and susceptibility to frost action are closely correlated with gradation parameters. Gradation is measured in the laboratory using two tests: a mechanical sieve analysis for the sand and coarser fraction (Table 5-19), and a hydrometer test for the silt and finer clay material (Table 5-20).

Gradation is quantified by the percentage (most commonly by weight) of the soil that is finer than a given size ("percent passing") vs. grain size. Gradation is occasionally expressed alternatively in terms of the percent coarser than a given grain size. Gradation characteristics are also expressed in terms of D_n parameters, where D is the largest particle size in the n percent finest fraction of soil. For example, D_{10} is the largest particle size in the 10% finest fraction of soil; D_{60} is the largest particle size in the 60% finest fraction of soil.

Table 5-19. Grain size distribution of coarse particles (mechanical sieve analysis).

Description	The grain size distribution is the percentage of soil finer than a given size vs. grain size. Coarse particles are defined as larger than 0.075 mm (0.0029 in, or No. 200 sieve).
Uses in Pavements	• Soil classification (see Section 4.7.2) • Correlations with other engineering properties
Laboratory Determination	The grain size distribution of coarse particles is determined from a mechanical washed sieve analysis (AASHTO T 88, ASTM D 422). A representative sample is washed through a series of sieves (Figure 5-5). The amount retained on each sieve is collected, dried, and weighed to determine the percentage of material passing that sieve size. Figure 5-7 shows example grain size distributions for sand, silt, and clay soils as obtained from mechanical sieve and hydrometer (Table 5-20) tests.
Field Measurement	Not applicable.
Commentary	Obtaining a representative specimen is an important aspect of this test. When samples are dried for testing or "washing," it may be necessary to break up the soil clods. Care should be taken to avoid crushing of soft carbonate or sand particles. If the soil contains a substantial amount of fibrous organic materials, these may tend to plug the sieve openings during washing. The material settling over the sieve during washing should be constantly stirred to avoid plugging. Openings of fine mesh or fabric are easily distorted as a result of normal handling and use. They should be replaced often. A simple way to determine whether sieves should be replaced is the periodic examination of the stretch of the sieve fabric on its frame. The fabric should remain taut; if it sags, it has been distorted and should be replaced. A common cause of serious errors is the use of "dirty" sieves. Some soil particles, because of their shape, size or adhesion characteristics, have a tendency to be lodged in the sieve openings.
Typical Values	Typical particles size ranges for various soil textural categories are as follows (ASTM D 2487): • Gravel: 4.75 – 75 mm (0.19 – 3 in; No. 4 to 3-inch sieves) • Sand: 0.075 – 4.75 mm (0.0029 – 0.19 in; No. 200 to No. 4 sieves) • Silt and clay: < 0.075 mm (0.0029 in; No. 200 sieve)

Table 5-20. Grain size distribution of fine particles (hydrometer analysis).

Description	The grain size distribution is the percentage of soil finer than a given size vs. grain size. Fine particles are defined as smaller than 0.075 mm (0.0029 in, or No. 200 sieve).
Uses	• Soil classification (see Section 4.7.2) • Correlations with other engineering properties
Laboratory Determination	The grain size distribution of fine particles is determined from a hydrometer analysis (AASHTO T 88, ASTM D 422). Soil finer than 0.075 mm (0.0029 in or No. 200 sieve) is mixed with a dispersant and distilled water and placed in a special graduated cylinder in a state of liquid suspension (Figure 5-6). The specific gravity of the mixture is periodically measured using a calibrated hydrometer to determine the rate of settlement of soil particles. The relative size and percentage of fine particles are determined based on Stoke's law for settlement of idealized spherical particles. Figure 5-5 shows example grain size distributions for sand, silt, and clay soils as obtained from mechanical sieve (Table 5-19) and hydrometer tests.
Field Measurement	Not applicable.
Commentary	The principal value of the hydrometer analysis is in obtaining the clay fraction (percent finer than 0.002 mm). This is because the soil behavior for a cohesive soil depends principally on the type and percent of clay minerals, the geologic history of the deposit, and its water content, rather than on the distribution of particle sizes. Repeatable results can be obtained when soils are largely composed of common mineral ingredients. Results can be distorted and erroneous when the composition of the soil is not taken into account to make corrections for the specific gravity of the specimen. Particle size of highly organic soils cannot be determined by the use of this method.
Typical Values	• Silt: 0.075 – 0.002 mm (0.0029 – 0.000079 in.) • Clay: < 0.002 mm (0.000079 in.)

Figure 5-5. Laboratory sieves for mechanical analysis of grain size distribution. Shown (right to left) are sieve Nos. 3/8-in. (9.5-mm), No. 10 (2.0-mm), No. 40 (250-µm) and No. 200 (750-µm) and example soil particle sizes including (right to left): medium gravel, fine gravel, medium-coarse sand, silt, and dry clay (kaolin).

Figure 5-6. Soil hydrometer apparatus (http://www.ce.siue.edu/).

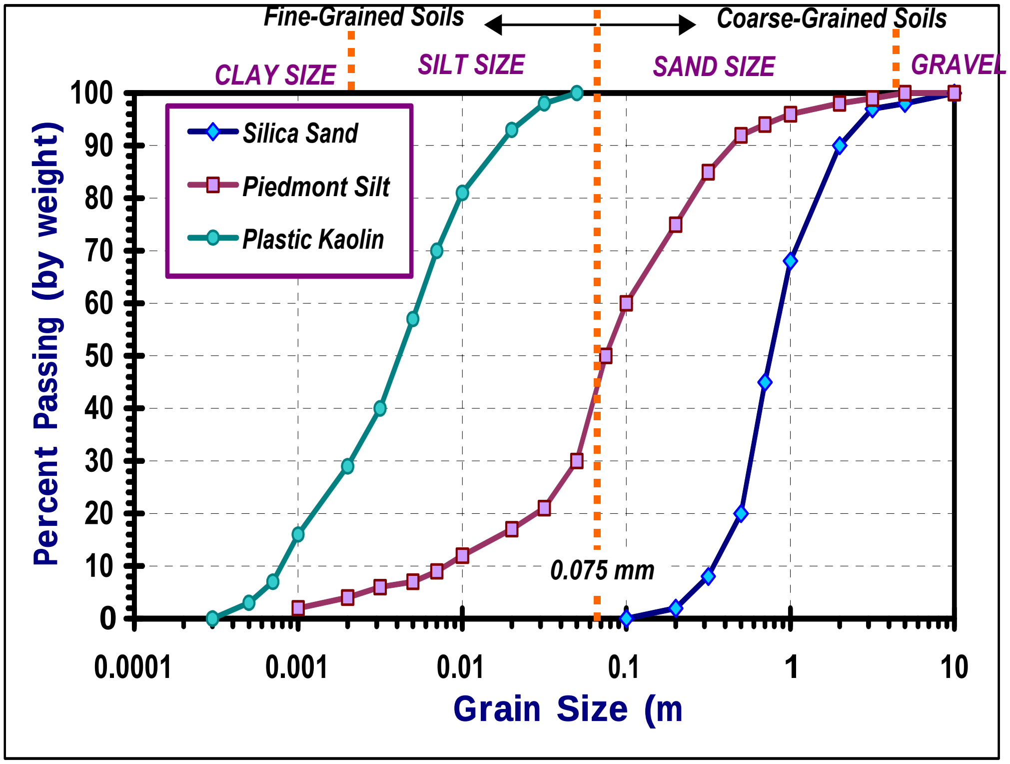


Figure 5-7. Representative grain size distributions for several soil types.

Plasticity

Plasticity describes the response of a soil to changes in moisture content. When adding water to a soil changes its consistency from hard and rigid to soft and pliable, the soil is said to exhibit plasticity. Clays can be very plastic, silts are only slightly plastic, and sands and gravels are non-plastic. For fine-grained soils, engineering behavior is often more closely correlated with plasticity than gradation. Plasticity is a key component of AASHTO and Unified soil classification systems (Section 4.7.2).

Soil plasticity is quantified in terms of Atterberg limits. As shown in Figure 5-8, the Atterberg limit values correspond to values of moisture content where the consistency of the soil changes as it is progressively dried from a slurry:

- The liquid limit (*LL*), which defines the transition between the liquid and plastic states.
- The plastic limit (*PL*), which defines the transition between the plastic and semi-solid states.

- The shrinkage limit (*SL*), which defines the transition between the semi-solid and solid states.
- Note in Figure 5-8 that the total volume of the soil changes as it is dried until the shrinkage limit is reached; drying below the shrinkage limit does not cause any additional volume change.

It is important to recognize that Atterberg limits are not fundamental material properties. Rather, they should be interpreted as index values determined from standardized test methods (Table 5-21).

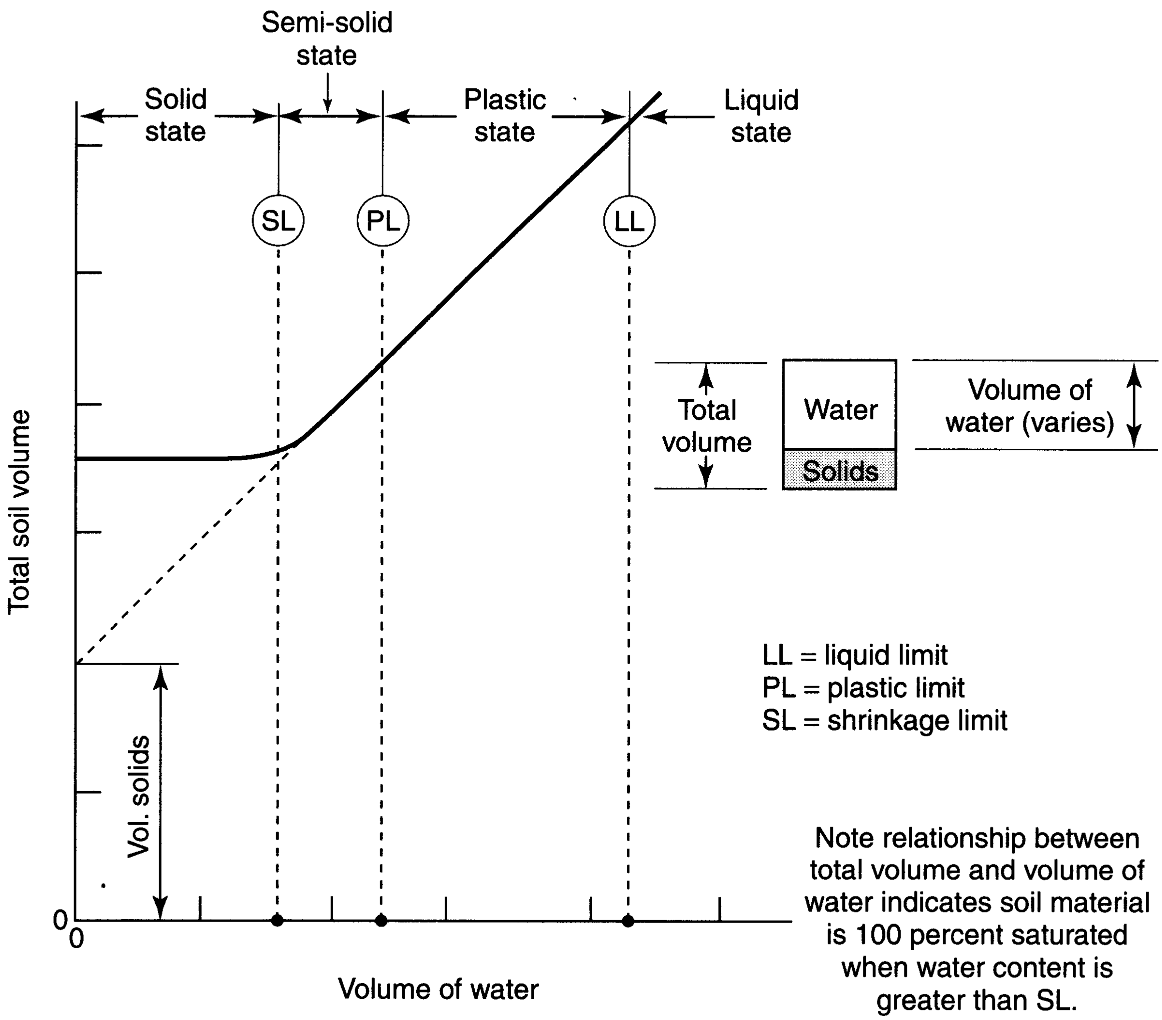


Figure 5-8. Variation of total soil volume and consistency with change in water content for a fine-grained soil (from McCarthy, 2002).

Table 5-21. Plasticity of fine-grained soils (Atterberg limits).

Description	Plasticity describes the response of a soil to changes in moisture content. Plasticity is quantified by Atterberg limits.
Uses in Pavements	• Soil classification (see Section 4.7.2) • Correlations with other engineering properties
Laboratory Determination	Atterberg limits are determined using test protocols described in AASHTO T89 (liquid limit), AASHTO T90 (plastic limit), AASHTO T 92 (shrinkage limit), ASTM D 4318 (liquid and plastic limits), and ASTM D 427 (shrinkage limit). A representative sample is taken of the portion of the soil passing the No. 40 sieve. The moisture content is varied to identify three stages of soil behavior in terms of consistency: • The *liquid limit* (*LL*) is defined as the water content at which 25 blows of the liquid limit machine (Figure 5-9) closes a standard groove cut in the soil pat for a distance of 12.7 cm (1/2 in.). An alternate procedure in Europe and Canada uses a fall cone device to obtain better repeatability. • The *plastic limit* (*PL*) is as the water content at which a thread of soil, when rolled down to a diameter of 3 mm (1/8 in.), will crumble. • The *shrinkage limit* (*SL*) is defined as that water content below which no further soil volume change occurs with additional drying.
Field Measurement	Not applicable.
Commentary	The Atterberg limits provide general indices of moisture content relative to the consistency and behavior of soils. The *LL* defines the lower boundary for the liquid state, while the *PL* defines the upper bound of the solid state The difference is termed the *plasticity index* ($PI = LL - PL$). The *liquidity index* (*LI*), defined as $LI = (w-PL)/PI$, where w is the natural moisture content, is an indicator of soil consistency in its natural in-situ conditions. It is important to recognize that Atterberg limits are approximate and empirical values. They were originally developed for agronomic purposes. Their widespread use by engineers has resulted in the development of a large number of empirical relationships for characterizing soils. Considering the somewhat subjective nature of the test procedure, Atterberg limits should only be performed by experienced technicians. Lack of experience and care can introduce serious errors in the test results. The optimum compaction moisture content is often in the vicinity of the plastic limit.
Typical Values	See Table 5-22.

Figure 5-9. Liquid limit test device.

Table 5-22. Characteristics of soils with different plasticity indices (after Sowers, 1979).

Plasticity Index	Classification	Dry Strength	Visual-Manual Identification of Dry Sample
0 – 3	Nonplastic	Very low	Falls apart easily
3 – 15	Slightly plastic	Slight	Easily crushed with fingers
15 – 30	Medium plastic	Medium	Difficult to crush with fingers
> 30	Highly plastic	High	Impossible to crush with fingers

5.3.3 Problem Soil Identification

Two special conditions that often need to be checked for natural subgrade soils are the potential for swelling clays (Table 5-23) or collapsible silts (Table 5-25).

Swelling soils exhibit large changes in soil volume with changes in soil moisture. The potential for volumetric swell of a soil depends on the amount of clay, its relative density, the compaction moisture and density, permeability, location of the water table, presence of vegetation and trees, and overburden stress. Swell potential also depends on the mineralogical composition of fine-grained soils. Montmorillonite (smectite) exhibits a high degree of swell potential, illite has negligible to moderate swell characteristics, and kaolinite exhibits almost none. A one-dimensional swell potential test is used to estimate the percent swell and swelling pressures developed by the swelling soils (Table 5-23).

Collapsible soils exhibit abrupt changes in strength at moisture contents approaching saturation. When dry or at low moisture content, collapsible soils give the appearance of a stable deposit. At high moisture contents, these soils collapse and undergo sudden decreases in volume. Collapsible soils are found most commonly in loess deposits, which are composed of windblown silts. Other collapsible deposits include residual soils formed as a result of the removal of organics by decomposition or the leaching of certain minerals (calcium carbonate). In both cases, disturbed samples obtained from these deposits will be classified as silt. Loess, unlike other non-cohesive soils, will stand on almost a vertical slope until saturated. It has a low relative density, a low unit weight, and a high void ratio. A one-dimensional collapse potential test is used to identify collapsible soils (Table 5-25).

Table 5-23. Swell potential of clays.

Description	Swelling is a large change in soil volume induced by changes in moisture content.
Uses in Pavements	Swelling subgrade soils can have a seriously detrimental effect on pavement performance. Swelling soils must be identified so that they can be either removed, stabilized, or accounted for in the pavement design.
Laboratory Determination	Swell potential is measured using either the AASHTO T 258 or ASTM D 4546 test protocols. The swell test is typically performed in a consolidation apparatus. The swell potential is determined by observing the swell of a laterally-confined specimen when it is surcharged and flooded. Alternatively, after the specimen is inundated, the height of the specimen is kept constant by adding loads. The vertical stress necessary to maintain zero volume change is the swelling pressure.
Field Measurement	Not applicable.
Commentary	This test can be performed on undisturbed, remolded, or compacted specimens. If the soil structure is not confined (*i.e.,* a bridge abutment) such that swelling may occur laterally and vertically, triaxial tests can be used to determine three dimensional swell characteristics.
Typical Values	Swell potential can be estimated in terms of soil physical properties; see Table 5-24.

Table 5-24. Estimation of swell potential (Holtz and Gibbs, 1956).

% finer than 0.001mm	Atterberg Limits		Probable expansion, % total volume change*	Potential for expansion
	PI (%)	SL (%)		
> 28	> 35	< 11	> 30	Very high
20-31	25-41	7-12	20-30	High
13-23	15-28	10-16	10-30	Medium
< 15	< 18	>15	< 10	Low

*Based on a loading of 6.9 kPa (1 psi).

Table 5-25. Collapse potential of soils.

Description	Collapsible soils exhibit large decreases in strength at moisture contents approaching saturation, resulting in a collapse of the soil skeleton and large decreases in soil volume.
Uses in Pavements	Collapsible subgrade soils can have a seriously detrimental effect on pavement performance. Collapsible soils must be identified so that they can be either removed, stabilized, or accounted for in the pavement design.
Laboratory Determination	Collapse potential is measured using the ASTM D 5333 test protocol. The collapse potential of suspected soils is determined by placing an undisturbed, compacted, or remolded specimen in a consolidometer ring. A load is applied and the soil is saturated to measure the magnitude of the vertical displacement.
Field Measurement	Not applicable.
Commentary	The collapse during wetting occurs due to the destruction of clay binding, which provides the original strength of these soils. Remolding and compacting may also destroy the original structure.
Typical Values	None available.

5.3.4 Other Aggregate Tests

There is a wide range of other mechanical property tests that are performed to measure the quality and durability of aggregates used as subbases and bases in pavement systems and as constituents of asphalt and Portland cement concrete. These other aggregate tests are summarized in Table 5-26. Additional information can be found in *The Aggregate Handbook* published by the National Stone Association (Barksdale, 2000). A recent NCHRP study provides additional useful information on performance-related tests of aggregates used in unbound pavement layers (Saeed, Hall, and Barker, 2001).

Table 5-26. Other tests for aggregate quality and durability.

Property	Use	AASHTO Specification	ASTM Specification
Fine Aggregate Quality			
Sand Equivalent	Measure of the relative proportion of plastic fines and dust to sand size particles in material passing the No. 4 sieve	T 176	D 2419
Fine Aggregate angularity (also termed Uncompacted Air Voids)	Index property for fine aggregate internal friction in Superpave asphalt mix design method	T 304	C 1252
Coarse Aggregate Quality			
Coarse Aggregate Angularity	Index property for coarse aggregate internal friction in Superpave asphalt mix design method		D 5821
Flat, Elongated Particles	Index property for particle shape in Superpave asphalt mix design method		D 4791
General Aggregate Quality			
Absorption	Percentage of water absorbed into permeable voids	T 84/T 85	C 127/C 128
Particle Index	Index test for particle shape		D 3398
Los Angeles degradation	Measure of coarse aggregate resistance to degradation by abrasion and impact	T 96	C 131 or C 535
Soundness	Measure of aggregate resistance to weathering in concrete and other applications	T 104	C 88
Durability	Index of aggregate durability	T 210	D 3744
Expansion	Index of aggregate suitability		D 4792
Deleterious Materials	Describes presence of contaminants like shale, clay lumps, wood, and organic material	T 112	C 142

5.4 MECHANICAL PROPERTIES

Stiffness is the most important mechanical characteristic of unbound materials in pavements. The relative stiffnesses of the various layers dictate the distribution of stresses and strains within the pavement system. Figure 5-10 and Figure 5-11 illustrate respectively how the stiffnesses of the subgrade and the unbound base layer influence the horizontal tensile strain at the bottom of the asphalt and the compressive vertical strain at the top of the subgrade for a simple three-layer flexible pavement system. These pavement response parameters are directly related to asphalt fatigue cracking and subgrade rutting performance, respectively.

It may seem odd that stiffness rather than strength is considered the most important unbound material property for pavements. Pavement structural design is usually viewed as ensuring sufficient load-carrying capacity for the applied traffic – *i.e.,* providing sufficient pavement strength. However, the stress levels in well-designed asphalt or PCC-surfaced pavement are well below the strength of the unbound materials, and thus failure under any given load application is not an issue. The situation for aggregate-surfaced roads is, of course, a bit different: strength of the aggregate surface will directly influence the road's durability and performance. Subgrade strength is also an important issue during pavement construction.

The preferred method for characterizing the stiffness of unbound pavement materials is the resilient modulus M_R (Section 5.4.3), which is defined as the unloading modulus in cyclic loading. The AASTHO Design Guides beginning in 1986 have recommended the resilient modulus for characterizing subgrade support for flexible and rigid pavements and for determining structural layer coefficients for flexible pavements. The resilient modulus is also the primary material property input for unbound materials in the NCHRP 1-37A Design Guide for both flexible and rigid pavements.

Both the AASHTO and NCHRP 1-37A design procedures recognize the need for backward compatibility with other properties used in the past to characterize unbound materials, in particular the California Bearing Ratio and the Stabilometer R-value. These index material properties continue to be used by many highway agencies. Correlations are provided in both design procedures for relating CBR and R-values to M_R (or, in the case of the AASHTO Guides, to the structural layer coefficients a_i). The modulus of subgrade reaction (k) used in the AASHTO Guides is also correlated to M_R.

Laboratory and field methods (where appropriate) for determining the stiffness and other relevant mechanical properties of unbound materials in pavement systems are described in the following subsections and tables. Typical values for each property are also summarized. The soil mechanical properties described here are:

- Index properties
  - o California Bearing Ratio (Section 5.4.1)
  - o Stabilometer R-Value (Section 5.4.2)
  - o Structural Layer Coefficients (Section 5.4.5)
- Stiffness properties
  - o Resilient Modulus (Section 5.4.3)
  - o Poisson's Ratio (Section 5.4.4)
  - o Modulus of Subgrade Reaction (Section 5.4.6)
- Other properties
  - o Interface Friction (Section 5.4.7)
  - o Permanent Deformation (Section 5.4.8)
  - o Coefficient of Lateral Pressure (Section 5.4.96)

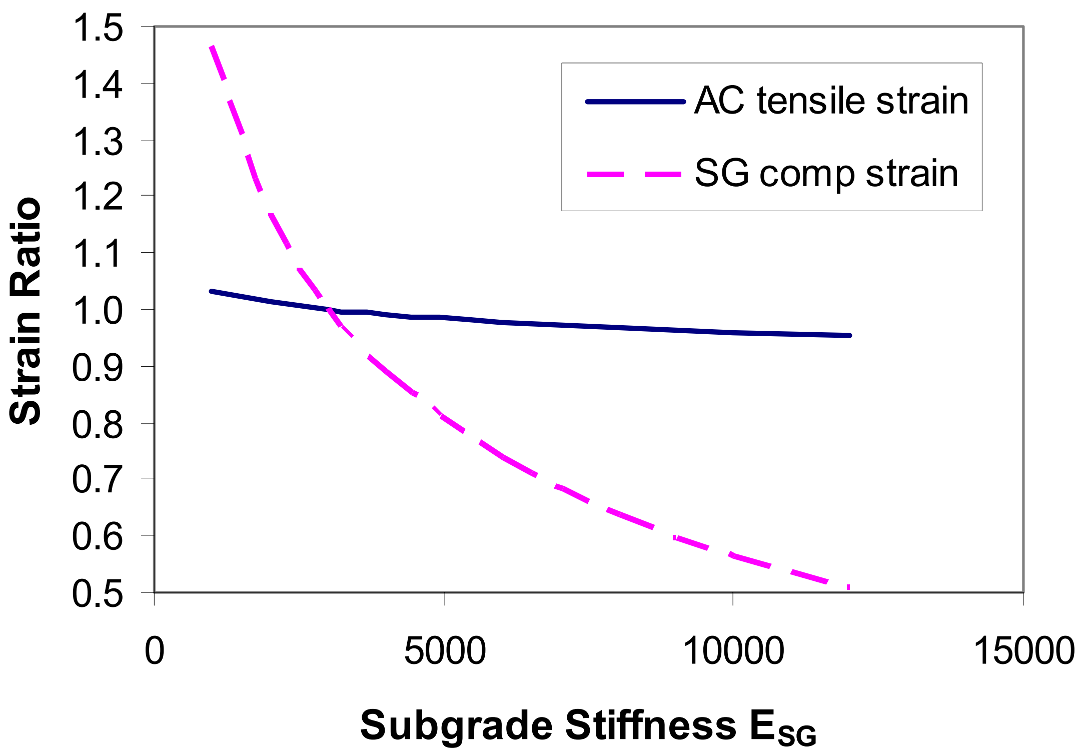


Figure 5-10. Influence of subgrade stiffness on critical pavement strains. (Elastic solution, 6 in./150 mm AC over 18 in./450 mm granular base. Reference elastic moduli: E_{AC} = 500,000 psi/3450 MPa; E_{BS} = 30,000 psi/207 Mpa; E_{SG} = 3000 psi/20.7 MPa. Load: 10 kip/44.5 kN single-wheel load, 100 psi/690 kPa contact pressure.)

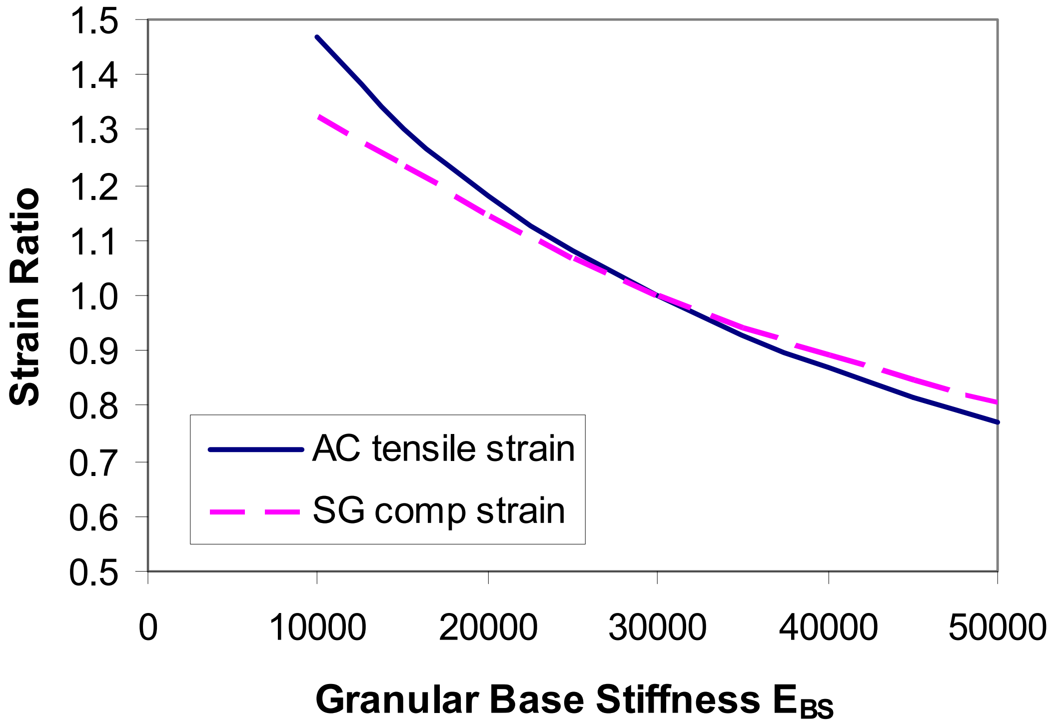


Figure 5-11. Influence of granular base stiffness on critical pavement strains. (Elastic solution, 6 in./150 mm AC over 18 in./450 mm granular base. Reference elastic moduli: EAC = 500,000 psi/3450 MPa; EBS = 30,000 psi/207 Mpa; ESG = 3000 psi/20.7 MPa. Load: 10 kip/44.5 kN single wheel load, 100 psi/690 kPa contact pressure.)

5.4.1 California Bearing Ratio (CBR)

The California Bearing Ratio or CBR test (Table 5-27) is an indirect measure of soil strength based on resistance to penetration by a standardized piston moving at a standardized rate for a prescribed penetration distance (Figure 5-12). CBR values are commonly used for highway, airport, parking lot, and other pavement designs based on empirical local or agency-specific methods (*i.e.,* FHWA, FAA, AASHTO). CBR has also been correlated empirically with resilient modulus and a variety of other engineering soil properties.

CBR is not a fundamental material property and thus is unsuitable for direct use in mechanistic and mechanistic-empirical design procedures. However, it is a relatively easy and inexpensive test to perform, it has a long history in pavement design, and it is reasonably well correlated with more fundamental properties like resilient modulus. Consequently, it continues to be used in practice.

Table 5-27. California Bearing Ratio (CBR).

Description	The California Bearing Ratio or CBR is an indirect measure of soil strength based on resistance to penetration.
Uses in Pavements	• Direct input to some empirical pavement design methods • Correlations with resilient modulus and other engineering properties
Laboratory Determination	AASTHO T 193 or ASTM D 1883. CBR is based on resistance to penetration by a standardized piston moving at a standardized rate for a prescribed penetration distance (Figure 5-12). CBR is defined as the ratio of the load required to cause a certain depth of penetration of a piston into a compacted specimen of soil at some water content and density, to the *standard load* required to obtain the same depth of penetration on a standard sample of crushed stone (usually limestone). Typically soaked conditions are used to simulate anticipated long-term conditions in the field. The CBR test is run on three identically compacted samples. Each series of the CBR test is run for a given relative compaction and moisture content. The geotechnical engineer must specify the conditions (dry, at optimum moisture, after soaking, 95% relative compaction, etc.) under which each test should be performed.
Field Measurement	ASTM D 4429. Test procedure is similar to that for laboratory determination.
Commentary	Most CBR testing is laboratory-based; thus, the results will be highly dependent on the representativeness of the samples tested. It is also important that the testing conditions be clearly stated: CBR values measured from as-compacted samples at optimum moisture and density conditions can be significantly greater than CBR values measured from similar samples after soaking, for example. For field measurement, care should be taken to make certain that the deflection dial is anchored well outside the loaded area. Field measurement is made at the field moisture content while laboratory testing is typically performed for soaked conditions, so soil-specific correlations between field and laboratory CBR values are often required.
Typical Values	See Table 5-28. For AASHO Road Test, CBR ≅ 100 for the granular base layer and about 30 for the granular subbase.

Figure 5-12. California Bearing Ratio test device (http://www.ele.com/geot/cali.htm).

Table 5-28. Typical CBR values (after U.S. Army Corps of Engineers, 1953).

USCS Soil Class	Field CBR
GW	60 – 80
GP	35 – 60
GM	40 – 80
GC	20 – 40
SW	20 – 40
SP	15 – 25
SM	20 – 40
SC	10 – 20
ML	5 – 15
CL	5 – 15
OL	4 – 8
MH	4 – 8
CH	3 – 5
OH	3 – 5

5.4.2 Stabilometer (R-Value)

The Stabilometer or R-Value test (Table 5-29) was developed by the California Division of Highways for use in their in-house empirical pavement design method. The R-value measured in this test is a measure of the resistance to deformation and is expressed as a function of the ratio of the induced lateral pressure to the applied vertical pressure as measured in a triaxial-type loading device (Figure 5-13):

$$R = 100 - \frac{100}{(2.5/D_2)\left[(P_v/P_h) - 1\right] + 1} \tag{5.7}$$

in which R = resistance value
P_v = applied vertical pressure (160 psi)
P_h = transmitted horizontal pressure
D_2 = displacement of stabilometer fluid necessary to increase horizontal pressure from 5 to 100 psi, measured in revolutions of a calibrated pump handle

A kneading compactor is used to prepare the test samples, as specimens fabricated by this method are thought to develop internal structures most similar to those in actual field compacted materials.

The R-Value is used either directly or translated into more common factors (*i.e.,* CBR) through correlation charts to be used with other more common design methods (*i.e.,* AASHTO). Like CBR, however, it is not a fundamental material property and thus is unsuitable for use in mechanistic and mechanistic-empirical design procedures.

Table 5-29. Stabilometer or R-Value.

Description	The R-value is a measure of the ability of a soil to resist lateral deformation under vertical load.
Uses in Pavements	• Direct input to some empirical pavement design methods • Correlations with other properties (e.g., CBR, resilient modulus)
Laboratory Determination	Measurement of the R-value of a soil is done with a stabilometer (AASHTO T 190 or ASTM D 2844). A stabilometer (Figure 5-13) is similar to a triaxial device consisting of a metal cylinder in which there is a rubber membrane; the annular space between the two is filled with oil that transmits lateral pressure to the specimen. Compacted, unstabilized, or stabilized soils and aggregates can be tested. Samples are compacted using a special kneading compaction device. When the specimen is vertically loaded, a lateral pressure is transmitted to the soil, which can be measured on a pressure gage. The R-value is determined for the vertical to lateral pressure ratio and the displacement.
Field Measurement	Not applicable.
Commentary	The test also allows the measurement of swell pressure of expansive soils (see Section 5.3.3). The swell pressure or expansion pressure data is used in determining the suitability of expansive soils for use under pavements and the magnitude of overburden pressure needed to control the expansion of these soils.
Typical Values	Dense graded crushed stone: 80+ High compressibility silts: 15 – 30 For the AASHO Road Test, R ≅ 85 for the granular base layer and about 60 for the granular subbase.

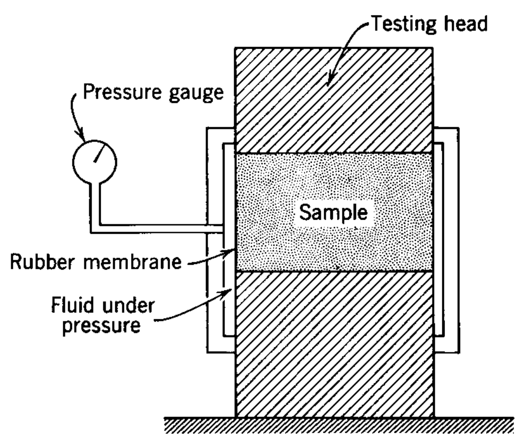


Figure 5-13. Schematic of stabilometer test setup for measuring R-value (Yoder and Witczak, 1975).

5.4.3 Elastic (Resilient) Modulus

Pavement thickness design prior to the 1986 AASHTO Design Guide was based on experience, soil classification, and the plastic response of pavement materials to static load, *e.g.,* Marshall stability for asphalt concrete and CBR for unbound materials. The potential for fatigue cracking of asphalt concrete and the accumulation of permanent deformations in the unbound materials in flexible pavements under essentially elastic deformation conditions were not considered. Many expressed concerns about this approach, including Professor A. Casagrande (Burmeister, 1943):

> "Irrespective of the theoretical method of evaluation of load tests, there remains the important question as to what extent individual static load tests reflect the results of thousands of dynamic load repetitions under actual traffic. Experience and large-scale traffic tests have already indicated that various types of soils react differently, and that the results of static load tests by no means bear a simple relation to pavement behavior."

Investigators in the 1950s began using repeated load triaxial tests in the laboratory to evaluate the stiffness and other behavior of pavement materials under conditions that more closely simulated real traffic loadings in the field. Substantial pioneering contributions in this area were made by Seed, Chan, and Monismith (1955), Seed and McNeill (1956), and Seed, Chan, and Lee (1963) in their work on the deformation characteristics and resilient modulus of compacted subgrades. They found significant differences between values of initial tangent modulus measured from single-cycle unconfined compression tests as compared to values of resilient modulus as determined from repeated cyclic unconfined compression loading. The conclusion from this work was that the behavior of soils under traffic loading should be obtained from repeated load tests whenever possible. This conclusion was substantiated by field data obtained by the California Department of Highways that showed the marked difference in pavement deflections occurring under standing and slowly moving wheel loads.

The culmination of this work was the adoption of resilient modulus testing by AASHTO in 1982. The AASHTO T274 standard was the first modern test protocol for resilient modulus. The concept of resilient modulus was subsequently incorporated into the 1986 and AASHTO Guide for Design of Pavement Structures.

Unbound Materials

The elastic modulus for unbound pavement materials is most commonly characterized in terms of the resilient modulus, M_R. The resilient modulus is defined as the ratio of the applied cyclic stress to the recoverable (elastic) strain after many cycles of repeated loading (Figure 5-14) and thus is a direct measure of stiffness for unbound materials in pavement systems. It is the single most important unbound material property input in most current pavement design procedures. Beginning in 1986, the AASTHO Design Guides have recommended use of resilient modulus for characterizing subgrade support for flexible and rigid pavements and for determining structural layer coefficients for flexible pavements. The resilient modulus is also the primary material property input for unbound materials in the NCHRP 1-37A Design Guide for both flexible and rigid pavements. It is an essential input to mechanistic pavement response models used to compute stresses, strains, and deformations induced in the pavement structure by the applied traffic loads.

The definition of the resilient modulus as measured in a standard resilient modulus cyclic triaxial test is shown in Figure 5-15, in which σ_a and ε_a are the stress and strain in the axial (*i.e.,* cyclic loading) direction. The sample is initially subjected to a hydrostatic confining pressure (σ_c), which induces an initial strain (ε_c). This initial strain is unmeasured in the test, but it is assumed the same in all directions for isotropic material behavior. The axial stress is then cycled at a constant magnitude ($\Delta\sigma$), which during unloading induces the cyclic resilient axial strain ($\Delta\varepsilon$). The resilient modulus (M_R) is defined simply as the ratio of the cyclic axial stress to resilient axial strain:

$$M_R = \frac{\Delta\sigma}{\Delta\varepsilon_a} \tag{5.8}$$

Although resilient modulus of unbound pavement materials is most commonly evaluated in the laboratory using a conventional triaxial cell, other test equipment/methods include the simple shear test, torsional resonant column testing, hollow cylinders, and true (cubical) triaxial tests. The pros and cons of these less-commonly employed testing procedures are described in Barksdale *et al.* (1996) and in LTPP (2003). The reasons that the triaxial device is most commonly used for resilient modulus testing include the following:

- *Equipment availability*. Resilient modulus testing can be performed using triaxial testing equipment commonly found in many pavement materials laboratory. This equipment is virtually identical to that found in most geotechnical laboratories except for the requirement of larger specimen sizes (up to 6 in./150 mm diameter by 12 in./300 mm tall) for coarse-grained base and subbase materials.

- *Stress state*. The stress conditions within the specimen on any plane are defined throughout the triaxial test. The stress conditions applied in resilient modulus testing are similar in magnitude to those that occur when an isolated wheel loading is applied to the pavement directly above the element of material simulated in the test.

- *Specimen drainage*. The triaxial test permits relatively simple, controlled drainage of the specimen in the axial and/or radial directions. Pore pressures can also be easily measured at the ends of the specimen, or, with more difficulty, within the specimen.

- *Strain measurement*. Axial, radial, and volumetric strains can all be measured relatively easily in the triaxial test.

- *Availability and robustness of test protocols*. The testing protocols for triaxial resilient modulus have been improved steadily over the years. Good summaries of the evolution of the various protocols and their advantages and disadvantages can be found in Andrei (1999) and Witczak (2004).

In addition to the above advantages, undisturbed tube samples of the subgrade obtained from the field can be extruded and tested with a minimum amount of specimen preparation. Finally, the triaxial cell used for the repeated load triaxial test can also be employed in static testing.

The most severe limitation of the triaxial cell is its ability to simulate rotation of the principal stress axes and shear stress reversal. Both of these phenomena apply when a wheel load moves across the pavement. Additionally, the intermediate principal stress applied to a specimen cannot be controlled in the triaxial test.

The laboratory-measured resilient modulus for most unbound pavement materials is stress dependent. The dominant effect for coarse-grained materials is an increase in M_R with increasing confining stress, while the dominant effect for fine-grained soils is a decrease in M_R with increasing shear stress. Many nonlinear M_R models have been proposed over the years for incorporating the effects of stress level on the resilient modulus (Andrei, 1999; Witczak, 2004). The stress-dependent M_R model implicitly included in the 1993 AASHTO Guide for granular base and subbase materials is (see Section 5.4.5 for more details)

$$M_R = k_1 \theta^{k_2} \tag{5.9}$$

in which

θ = bulk stress = $\sigma_1+\sigma_2+\sigma_3$ (psi)
k_1,k_2 = material properties

Guidance is provided in the 1993 AASHTO Guide for estimating the values of k_1 and k_2 for unbound base and subbase layers. Typical ranges of k_1 and k_2 are given in Table 5-30.

The more general stress-dependent M_R model adopted in the NCHRP 1-37A Design Guide is

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \tag{5.10}$$

in which

θ = bulk stress = $\sigma_1+\sigma_2+\sigma_3$ (same units as p_a)

τ_{oct} = octahedral shear stress = $\frac{1}{3}\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_1-\sigma_3)^2}$ (same units as p_a)

p_a = atmospheric pressure (to make equation dimensionless)

k_1,k_2,k_3 = material properties with constraints $k_1>0$, $k_2\geq 0$, $k_3\leq 0$

Equation (5.10) combines both the stiffening effect of the confinement or bulk stress (the term under the k_2 exponent) for coarse-grained materials and the softening effect of shear stress (the term under the k_3 exponent) for fine-grained soils.

The seasonal variation of unbound material properties is often significant, particularly for moisture-sensitive fine-grained soils or for locations with significant freeze-thaw cycles. Both the 1993 AASHTO Guide and the NCHRP 1-37A design procedures include provisions for including seasonal variations of unbound material properties in the design. The procedure in the 1993 AASHTO Guide for incorporating seasonal variations into the effective subgrade (M_R) can be briefly summarized as follows:

- Determine an M_R value for each time interval during a year. Typically, time intervals of two weeks or one month duration are used for this analysis. Methods for determining the M_R value for each time interval include
    - laboratory measurement at the estimated in-situ water content for the time interval.
    - backcalculation from FWD tests performed during each season. Mohammad *et al*. (2002) and Andrei (2003) provide some useful correlations between M_R, moisture content, and other soil parameters.

- Estimate a relative damage u_f corresponding to each seasonal modulus value using the empirical relationship

$$u_f = 1.18 \times 10^8 \left(M_R\right)^{-2.32} \tag{5.11}$$

- Compute the average relative damage $\bar{u}_f$ as the sum of the relative damage values for each season divided by the number of seasons.

- Determine the effective subgrade M_R from using the inverse of Eq. (5.11):

$$\boldsymbol{M_R = 3015(u_f)^{-0.431}} \tag{5.12}$$

This procedure can also be used to incorporate seasonal variations into the effective base and subbase M_R values used to estimate structural layer coefficients in the 1993 AASHTO Guide (see Section 5.4.5).

There are two options for incorporating the seasonal variation of unbound material properties in the NCHRP 1-37A design procedure. The first is the direct input of monthly M_R values. The second method combines moisture and freeze/thaw predictions from the Enhanced Integrated Climate Model (EICM) with models relating M_R to environmental conditions. The EICM and M_R environment models are built into the NCHRP 1-37A Design Guide software; details are provided in Appendix D.

Details of the procedures for determining M_R for unbound paving materials are given in Table 5-31. Laboratory determination of M_R is recommended for new construction and reconstruction projects. For rehabilitation projects, backcalculation of layer and subgrade M_R from FWD testing is the preferred approach (see Section 4.5.4), although calibrating backcalculated estimates with laboratory-measured values is a good practice (see Table 5-32).

Table 5-30. Typical ranges for k_1 and k_2 coefficients in Eq. (5.9) (AASHTO, 1993).

Material	k_1 (psi)	k_2
Granular base	3000 – 8000	0.5 – 0.7
Granular subbase	2500 - 7000	0.4 – 0.6

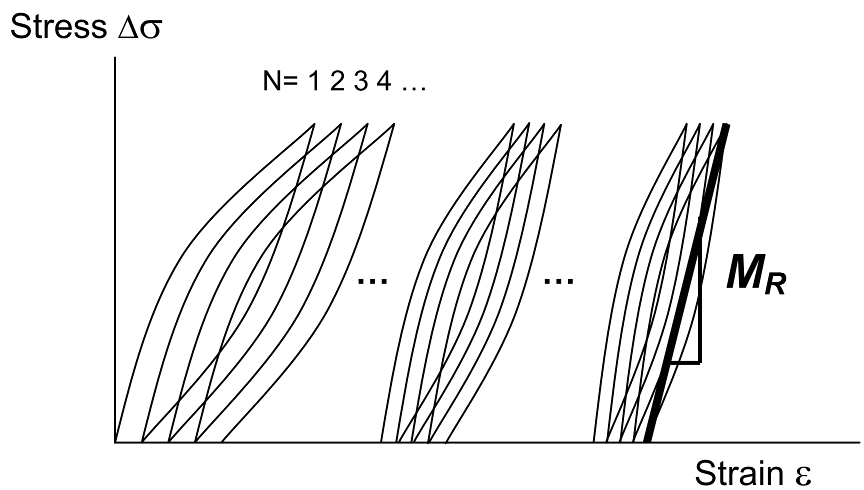


Figure 5-14. Resilient modulus under cyclic loading.

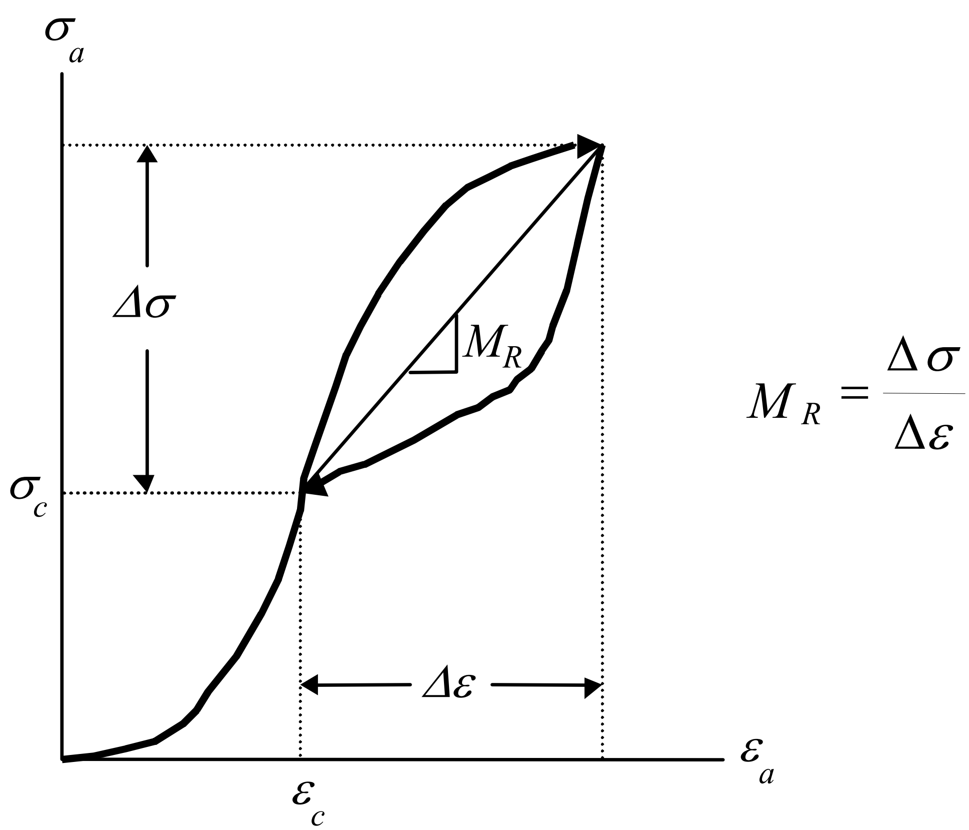


Figure 5-15. Definition of resilient modulus M_R for cyclic triaxial loading.

Table 5-31. Resilient modulus (M_R).

Description	The resilient modulus (M_R) is the elastic unloading modulus after many cycles of cyclic loading.
Uses in Pavements	• Characterization of subgrade stiffness for flexible and rigid pavements (AASHTO 1986/1993; NCHRP 1-37A) • Determination of structural layer coefficients in flexible pavements (AASHTO 1986/1993) • Characterization of unbound layer stiffness (NCHRP 1-37A)
Laboratory Determination	There currently are five test protocols in use for resilient modulus testing in the laboratory: • AASHTO T 292-91 • AASHTO T 294-92 • AASHTO T 307-99 (supersedes AASHTO 7 247) • AASHTO T P46-94 • NCHRP 1-28 Appendix E • NCHRP 1-28A ("harmonized" protocol) The harmonized protocol developed in NCHRP Project 1-28A attempts to combine the best features from all of the earlier test methods with a new loading sequence that minimizes the potential for premature failure of the test specimen. All of the test procedures employ a closed loop electro-hydraulic testing machine to apply repeated cycles of a haversine shaped load-pulse. Load pulses are typically a 0.1 second loading time followed by a 0.9 second rest time for base/subbase materials, and a 0.2 second loading time followed by an 0.8 second rest time for subgrade materials. A triaxial set-up for the resilient modulus test is shown in Figure 5-16. Axial deformation is best measured on the sample using clamps positioned one quarter and three quarters from the base of the test specimen. For very soft specimens, the displacement may be measured between the top and bottom plates. Different specimen sizes, compaction procedures, and loading conditions are usually recommended for granular base/subbase materials, coarse-grained subgrades, and fine-grained subgrades. These different procedures reflect the different particle sizes of the materials, the state of stress specific to each layer in the pavement structure, and the mechanical behavior of the material type. Detailed comparisons of the various resilient modulus test protocols is presented in Witczak (2004).
Field Measurement	In-situ resilient modulus values can be estimated from backcalculation of falling weight deflectometer (FWD) test results (Section 4.5.4) or correlations with Dynamic Cone Penetrometer (DCP) values (Section 4.5.5; see also Table 5-34).
Commentary	No definitive studies have been conducted to date to provide guidance on differences between measured M_R from the various laboratory test protocols. Field M_R values determined from FWD backcalculation are often significantly higher than design M_R values measured from laboratory tests because of differences in stress states. The 1993 AASHTO Guide recommends for

subgrade soils that field M_R values be multiplied by a factor of up to 0.33 for flexible pavements and up to 0.25 for rigid pavements to adjust to design M_R values. NCHRP 1-37A recommends adjustment factors of 0.40 for subgrade soils and 0.67 for granular bases and subbases under flexible pavements. More detailed guidance for adjusting backcalculated modulus values to design M_R values is given in Table 5-32.

The 1993 AASHTO Guide includes procedures for incorporating seasonal variations into an effective M_R for the subgrade. Seasonal variations of material properties are included directly in the NCHRP 1-37A M-E design methodology.

The Level 1, 2, and 3 M_R inputs in the NCHRP 1-37A design methodology are functions of pavement and construction type, as summarized in Table 5-33.

Typical Values

Correlations between M_R and other soil properties include the following:

AASHTO 1993 Guide

- Granular base and subbase layers:

θ (psi)	M_R (psi)
100	100 740 × *CBR* 1000 + 780 × *R*
30	440 × *CBR* 1000 + 450 × *R*
20	340 × *CBR* 1000 + 350 × *R*
10	250 × *CBR* 1000 + 250 × *R*

- Subgrade (roadbed) soils

$$M_R \text{ (psi)} = 1500 \times CBR \quad \text{for } CBR \leq 10 \tag{5.13}$$

(Heukelom and Klomp, 1962)

$$M_R \text{ (psi)} = A + B \times (\text{R-value}) \tag{5.14}$$

with A= 772 to 1,155; B = 369 to 555 (Asphalt Institute, 1982)

$$M_R \text{ (psi)} = 1000 + 555 \times (\text{R-value}) \quad \text{(recommended values)} \tag{5.15}$$

Additional useful correlations for subgrade M_R are provided in Figure 5-17.

NCHRP 1-37A (Level 2 Inputs)

See Table 5-34 for correlations between M_R and various material strength and index properties. The correlations in Table 5-34 are in rough order of preference; correlations of M_R with CBR have the longest history and most supporting data and thus are most preferable.

NCHRP 1-37A (Level 3 Inputs)

See Table 5-35 for typical ranges and default values as functions of AASHTO and USCS soil class. Note that these values are for soils compacted at optimum moisture and density conditions; the NCHRP 1-37A analysis software adjusts these for in-situ moisture and density conditions.

Figure 5-16. Triaxial cell set-up for resilient modulus test.

Table 5-32. Average backcalculated to laboratory-determined elastic modulus ratios (Von Quintus and Killingsworth, 1997a; 1997b; 1998).

Layer Type and Location		Mean E_R/M_R Ratio
Unbound Granular Base and Subbase Layers	Granular base/subbase between two stabilized layers (cementitious or asphalt stabilized materials).	1.43
	Granular base/subbase under a PCC layer.	1.32
	Granular base/subbase under an HMA surface or base layer.	0.62
Embankment and Subgrade Soils	Embankment or subgrade soil below a stabilized subbase layer or stabilized soil.	0.75
	Embankment or subgrade soil below a flexible or rigid pavement without a granular base/subbase layer.	0.52
	Embankment or subgrade soil below a flexible or rigid pavement with a granular base or subbase layer.	0.35

E_R = Elastic modulus backcalculated from deflection basin measurements.
M_R = Elastic modulus of the in-place materials determined from laboratory repeated load resilient modulus test.

Table 5-33. Hierarchical input levels for unbound material stiffness in the NCHRP 1-37A Design Guide (NCHRP 1-37A, 2004).

Project Type	Level 1	Level 2	*Level 3*
Flexible Pavements			
New/reconstruction	Laboratory-measured M_R with stress dependence—Eq. (5.10)	M_R correlations with other properties (Table 5-34)	Default M_R based on soil type (Table 5-35)
Rehabilitation	Backcalculated M_R from FWD deflections	M_R correlations with other properties (Table 5-34)	Default M_R based on soil type (Table 5-35)
Rigid Pavements			
New/reconstruction	Not available	M_R correlations with other properties (Table 5-34)	Default M_R based on soil type (Table 5-35)
Rehabilitation	Backcalculated modulus of subgrade reaction (k) from FWD deflections (see Section 5.4.6)	M_R correlations with other properties (Table 5-34)	Default M_R based on soil type (Table 5-35)

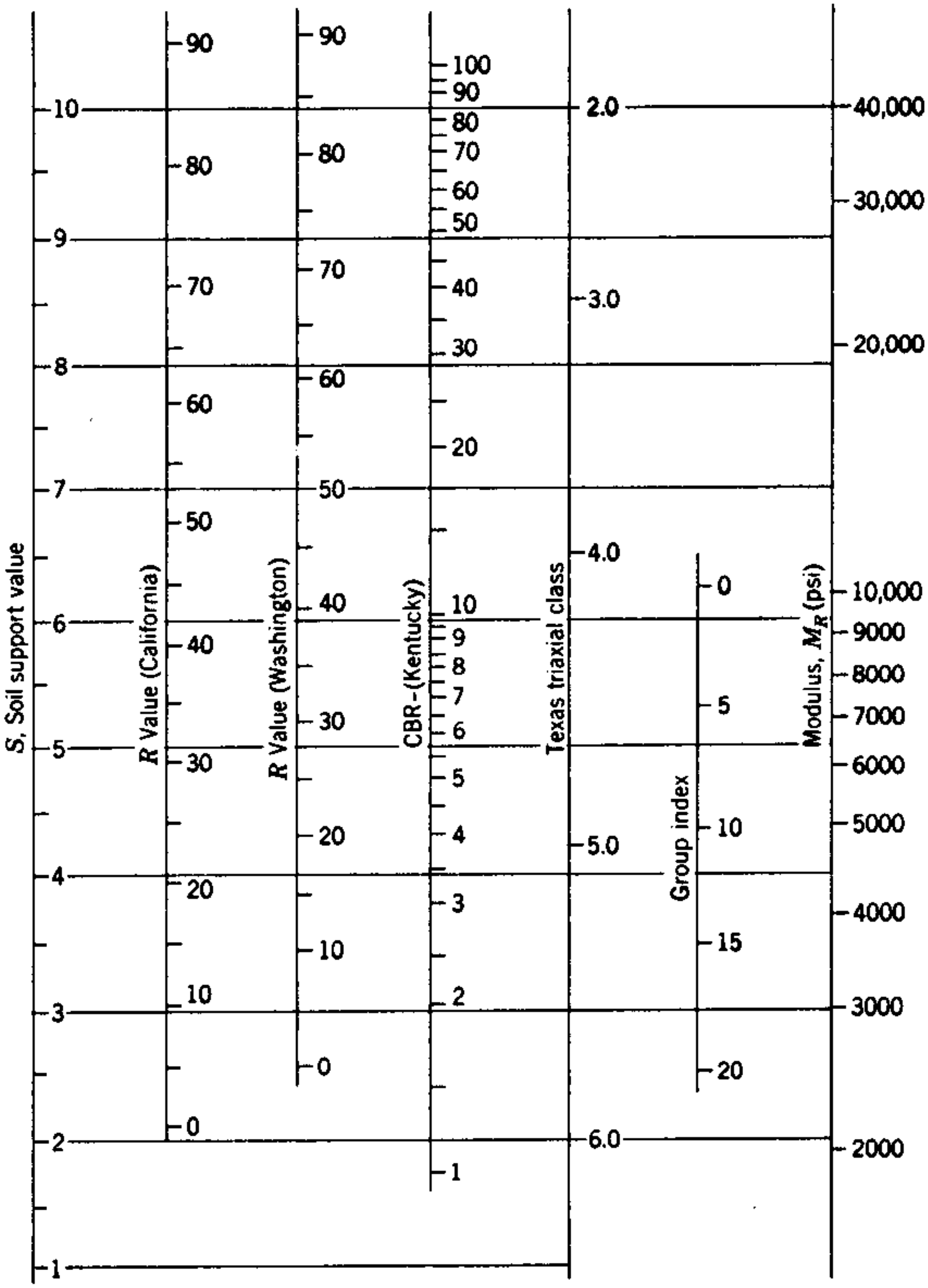


Figure 5-17. Correlations between subgrade resilient modulus and other soil properties (1 psi = 6.9 kPa; from Huang, 1993, after Van Til *et al.*, 1972).

Table 5-34. Correlations between resilient modulus and various material strength and index properties (NCHRP 1-37A, 2004).

Strength/Index Property	Model[a]	Comments	Test Standard
California Bearing Ratio[b]	M_R (psi) = $2555(CBR)^{0.64}$ M_R (MPa) = $17.6(CBR)^{0.64}$	CBR = California Bearing Ratio (%)	AASHTO T193—The California Bearing Ratio
Stabilometer R-value	M_R (psi) = $1155 + 555R$ M_R (MPa) = $8.0 + 3.8R$	R = R-value	AASHTO T190—Resistance R-Value and Expansion Pressure of Compacted Soils
AASHTO layer coefficient	M_R (psi) = $30{,}000\ (a_i/0.14)^3$ M_R (MPa) = $207\ (a_i/0.14)^3$	a_i = AASHTO layer coefficient	AASHTO Guide for the Design of Pavement Structures (1993)
Plasticity index and gradation	$CBR = \dfrac{75}{1+0.728(wPI)}$	$wPI = P200*PI$ $P200$ = % passing No. 200 sieve size PI = plasticity index (%)	AASHTO T27—Sieve Analysis of Coarse and Fine Aggregates AASHTO T90—Determining the Plastic Limit and Plasticity Index of Soils
Dynamic Cone Penetration[c]	$CBR = 292/(DCP^{1.12})$	CBR = California Bearing Ratio (%) DCP =Penetration index, in./blow	ASTM D6951—Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications

[a]Correlations should be applied to similar conditions – *i.e., CBR* measured at optimum moisture and density vs. soaked conditions correlates to M_R at corresponding moisture and density conditions.
[b]NCHRP 1-37A strongly recommends against use of the older Heukelom and Klomp (1962) correlation Eq. (5.13) between M_R and CBR specified in the 1993 AASHTO Design Guide.
[c]Estimates of CBR are used to estimate M_R.

Table 5-35. Default M_R values for unbound granular and subgrade materials at unsoaked optimum moisture content and density conditions (NCHRP 1-37A, 2004).

Material Classification	M_R Range (psi)*	Typical M_R (psi)*
AASHTO Soil Class		
A-1-a	38,500 – 42,000	40,000
A-1-b	35,500 – 40,000	38,000
A-2-4	28,000 – 37,500	32,000
A-2-5	24,000 – 33,000	28,000
A-2-6	21,500 – 31,000	26,000
A-2-7	21,500 – 28,000	24,000
A-3	24,500 – 35,500	29,000
A-4	21,500 – 29,000	24,000
A-5	17,000 – 25,500	20,000
A-6	13,500 – 24,000	17,000
A-7-5	8,000 – 17,500	12,000
A-7-6	5,000 – 13,500	8,000
USCS Soil Class		
GW	39,500 – 42,000	41,000
GP	35,500 – 40,000	38,000
GM	33,000 – 42,000	38,500
GC	24,000 – 37,500	31,000
GW-GM	35,500 – 40,500	38,500
GP-GM	31,000 – 40,000	36,000
GW-GC	28,000 – 40,000	34,500
GP-GC	28,000 – 39,000	34,000
SW	28,000 – 37,500	32,000
SP	24,000 – 33,000	28,000
SM	28,000 – 37,500	32,000
SC	21,500 – 28,000	24,000
SW-SM	24,000 – 33,000	28,000
SP-SM	24,000 – 33,000	28,000
SW-SC	21,500 – 31,000	25,500
SP-SC	21,500 – 31,000	25,500
ML	17,000 – 25,500	20,000
CL	13,500 – 24,000	17,000
MH	8,000 – 17,500	11,500
CH	5,000 – 13,500	8,000

*Multiply by 0.069 to convert to MPa.

Fractured PCC Slabs

Rehabilitation designs for AC overlays over badly damaged PCC existing pavement frequently require fracturing (crack and seat, etc.) or rubblizing of the existing concrete slabs. The net effect of the fracturing or rubblization process is to turn the slabs into a very coarse unbound granular material. Table 5-36 summarizes recommended design values for the modulus of the fractured slab, E_{fs}, for Level 1 characterization in the NCHRP 1-37A Design Guide. These recommended design values, which are functions of the anticipated variability of the slab fracturing process, were developed based on NDT data on fractured slab projects contained in NAPA IS-117 (NAPA, 1994). When using these design values, NDT of the fractured slab must be performed to ensure that not more than 5 percent of the in-situ fractured slab modulus values exceed 1000 ksi. The Level 1 design values may be used for all methods of fracture (crack and seat or rubblize for JPCP, break and seat or rubblize for JRCP, or rubblize for CRCP).

Table 5-37 summarizes recommended design values for the modulus of the fractured slab, E_{fs}, for Level 3 characterization in the NCHRP 1-37A Design Guide. These values, which are functions of the fracture method used and the nominal fragment size, were developed by applying conservatism to the relationship of E_{fs} versus nominal fragment size published in the 1986 AASHTO Design Guide and NAPA IS-117. Level 3 should not be used with JRCP unless it is certain that full debonding of the steel and concrete occurs.

Table 5-36. Recommended fractured slab design modulus values for Level 1 characterization (NCHRP 1-37A, 2004).

Expected Control on Slab Fracture Process	Anticipated Coefficient of Variation for the Fractured Slab Modulus, %	Design Modulus
Good to Excellent	25	600 ksi (4.1 GPa)
Fair to Good	40	450 ksi (3.1 GPa)
Poor to Fair	60	300 (2.1 GPa)

Table 5-37. Recommended fractured slab design modulus values for Level 3 characterization (NCHRP 1-37A, 2004).

Type of Fracture	Design Modulus
Rubbilization	150 ksi (1.0 GPa)
Crack and Seat	
12-in crack spacing	200 ksi (1.4 GPa)
24-in crack spacing	250 ksi (1.7 GPa)
36-in crack spacing	300 ksi (2.1 GPa)

Note: For JRCP, Level 1 should be used unless agency experience dictates otherwise.

Bedrock

Shallow bedrock under an alignment can have a significant impact on the pavement's mechanical responses and thus needs to be considered in mechanistic-empirical design. Shallow bedrock is also an important factor in the backcalculation of layer moduli for rehabilitation design. While a precise value of bedrock stiffness is seldom required, the effect of high bedrock stiffness must nonetheless be incorporated into the analysis. Recommended values from NCHRP 1-37A for the elastic modulus of bedrock are as follows:

- Solid, massive bedrock:
  E = 750 – 2,000 ksi (5.2 – 13.8 GPa)
  Default = 1,000 ksi (6.9 GPa)

- Highly fractured/weathered bedrock:
  E = 250 – 1,000 ksi (1.7 – 6.9 GPa)
  Default = 500 ksi (3.4 GPa)

5.4.4 Poisson's Ratio

Description	Poisson's ratio ν is defined as the ratio of the lateral strain ε_x to the axial strain ε_y due to an axial loading (Figure 5-18).
Uses in Pavements	• Direct input to pavement response models in M-E design procedure. • Estimation of in-situ lateral stresses (see Section 5.4.9).
Laboratory Determination	Determined as part of resilient modulus test (see Section 5.4.3.).
Field Measurement	Not applicable.
Commentary	The influence of ν on computed pavement response is normally quite small. Consequently, use of assumed values for ν often gives satisfactory results, and direct measurement in the laboratory is usually unnecessary.
Typical Values	Poisson's ratio for isotropic elastic materials must be between 0 and 0.5. Typical values of ν for pavement geomaterials are given in Table 5-30.

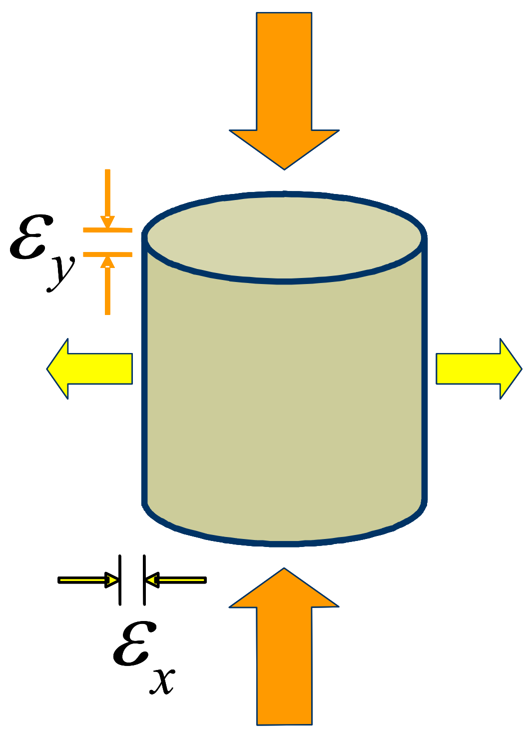


Figure 5-18. Illustration of Poisson's ratio.

Table 5-38. Typical Poisson's ratio values for geomaterials in pavements (NCHRP 1-37A, 2004).

Material Description	ν Range	ν Typical
Clay (saturated)	0.4 – 0.5	0.45
Clay (unsaturated)	0.1 – 0.3	0.2
Sandy clay	0.2 – 0.3	0.25
Silt	0.3 – 0.35	0.325
Dense sand	0.2 – 0.4	0.3
Coarse-grained sand	0.15	0.15
Fine-grained sand	0.25	0.25
Bedrock	0.1 – 0.4	0.25

5.4.5 Structural Layer Coefficients

The material quality of granular base and subbase layers is characterized in the AASHTO flexible pavement design procedures in terms of structural layer coefficients a_i (see Section 3.5.2). These coefficients were entirely empirical through the 1972 version of the Guide. Beginning with the 1986 Guide, the recommended procedure for estimating structural layer coefficients is through correlations with resilient modulus.

It must be emphasized that structural layer coefficients are not fundamental engineering properties for a material. There are no laboratory or field procedures for measuring structural layer coefficients directly. The structural layer coefficients were originally defined as simple substitution ratios – *i.e.,* how much additional thickness of granular base at a given reference stiffness must be added if a unit thickness of asphalt concrete of a given stiffness is removed in order to maintain the same surface deflection under a standardized load? These substitution ratios were evaluated in the 1986 AASHTO Guide[1] via a parametric analytical study for a limited range of flexible pavement geometries and layer stiffnesses. In this approach, the value of the structural layer coefficient for a given material also depends not only on its inherent stiffness, but also upon the material's location within the pavement structure (*e.g.,* the a_2 value for a given material when used in a base layer is different from the a_3 value for that same material when used as a subbase). Subsequent correlations between structural layer coefficients and other engineering properties such as resilient modulus and CBR are entirely empirical. Structural layer coefficients are not used in mechanistic-empirical design procedures like the NCHRP 1-37A Design Guide.

[1] See Appendix GG in Volume 2 of AASHTO (1986).

New Construction/Reconstruction

The relationship in the 1993 AASHTO Guide between the structural layer coefficient a_2 and resilient modulus E_{BS} (in psi) for granular base materials is given as

$$a_2 = 0.249 \log_{10} E_{BS} - 0.977 \tag{5.16}$$

The value for E_{BS} in Eq. (5.16) will be a function of the stress state within the layer. The relationship suggested in the 1993 AASHTO Guide is

$$E_S = k_1 \theta^{k_2} \tag{5.17}$$

in which

θ = sum of principal stresses = $\sigma_1+\sigma_2+\sigma_3$ (psi)
k_1,k_2 = material properties

Typical values for the material properties are (see also Table 5-39)

k_1 = 3000 to 8000 psi
k_2 = 0.5 to 0.7

The values of E_{BS} from the base layers in the original AASHO Road Test are summarized in Table 5-40. Note that the E_{BS} values are not only functions of moisture, but also of stress state θ, which in turn is a function of the pavement structure – *i.e.,* subgrade modulus and thickness of the surface layer. Typical values of θ recommended in the 1993 AASHTO Guide for use in base design are summarized in Table 5-41.

Figure 5-19 summarizes correlations between the a_2 structural layer coefficient for nonstabilized granular base layers and corresponding values of CBR, R-Value, Texas triaxial strength, and resilient modulus. Similar correlations between a_2 and various strength and stiffness measures for cement- and bituminous-treated granular bases are given in Figure 5-20 and Figure 5-21.

Table 5-39. Typical values for k_1 and k_2 for use in Eq. (5.17) for unbound base and subbase materials (AASHTO, 1993).

Moisture Condition	k_1* (psi)**	k_2*
(a) Base		
Dry	6,000 – 10,000	0.5 – 0.7
Damp	4,000 – 6,000	0.5 – 0.7
Wet	2,000 – 4,000	0.5 – 0.7
(b) Subbase		
Dry	6,000 – 8,000	0.4 – 0.6
Damp	4,000 – 6,000	0.4 – 0.6
Wet	1,500 – 4,000	0.4 – 0.6

*Range in k_1 and k_2 is a function of the material quality
**1 psi = 6.9 kPa

Table 5-40. Granular base resilient modulus E_{SB} values (psi) from AASHO Road Test (AASHTO, 1993).

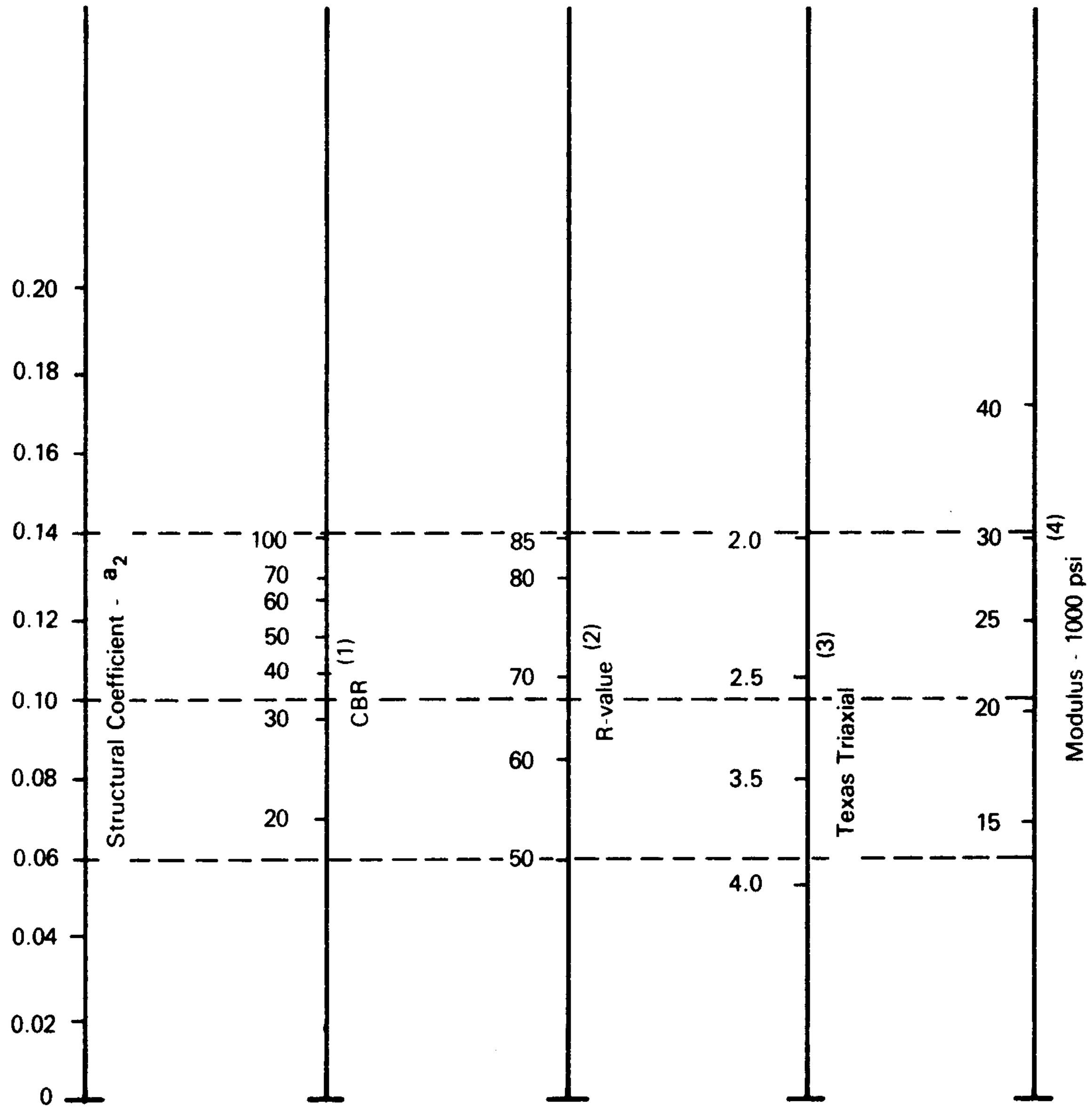
Moisture State	Equation	Stress State (psi*)			
		$\theta = 5$	$\theta = 10$	$\theta = 20$	$\theta = 30$
Dry	$8{,}000\theta^{0.6}$	21,012	31,848	48,273	61,569
Damp	$4{,}000\theta^{0.6}$	10,506	15,924	24,136	30,784
Wet	$3{,}200\theta^{0.6}$	8,404	12,739	19,309	24,627

*1 psi = 6.9 kPa

Table 5-41. Suggested bulk stress θ (psi) values for use in design of granular base layers (AASHTO, 1993).

Asphalt Concrete Thickness (inches*)	Roadbed Soil Resilient Modulus (psi*)		
	3,000	7,500	15,000
< 2	20	25	30
2 – 4	10	15	20
4 – 6	5	10	15
> 6	5	5	5

*1 inch = 25.4 mm; 1 psi = 6.9 kPa



(1) Scale derived by averaging correlations obtained from Illinois.
(2) Scale derived by averaging correlations obtained from California, New Mexico and Wyoming.
(3) Scale derived by averaging correlations obtained from Texas.
(4) Scale derived on NCHRP project *(3)*.

Figure 5-19. Correlations between structural layer coefficient a_2 and various strength and stiffness parameters for unbound granular bases (AASHTO, 1993).

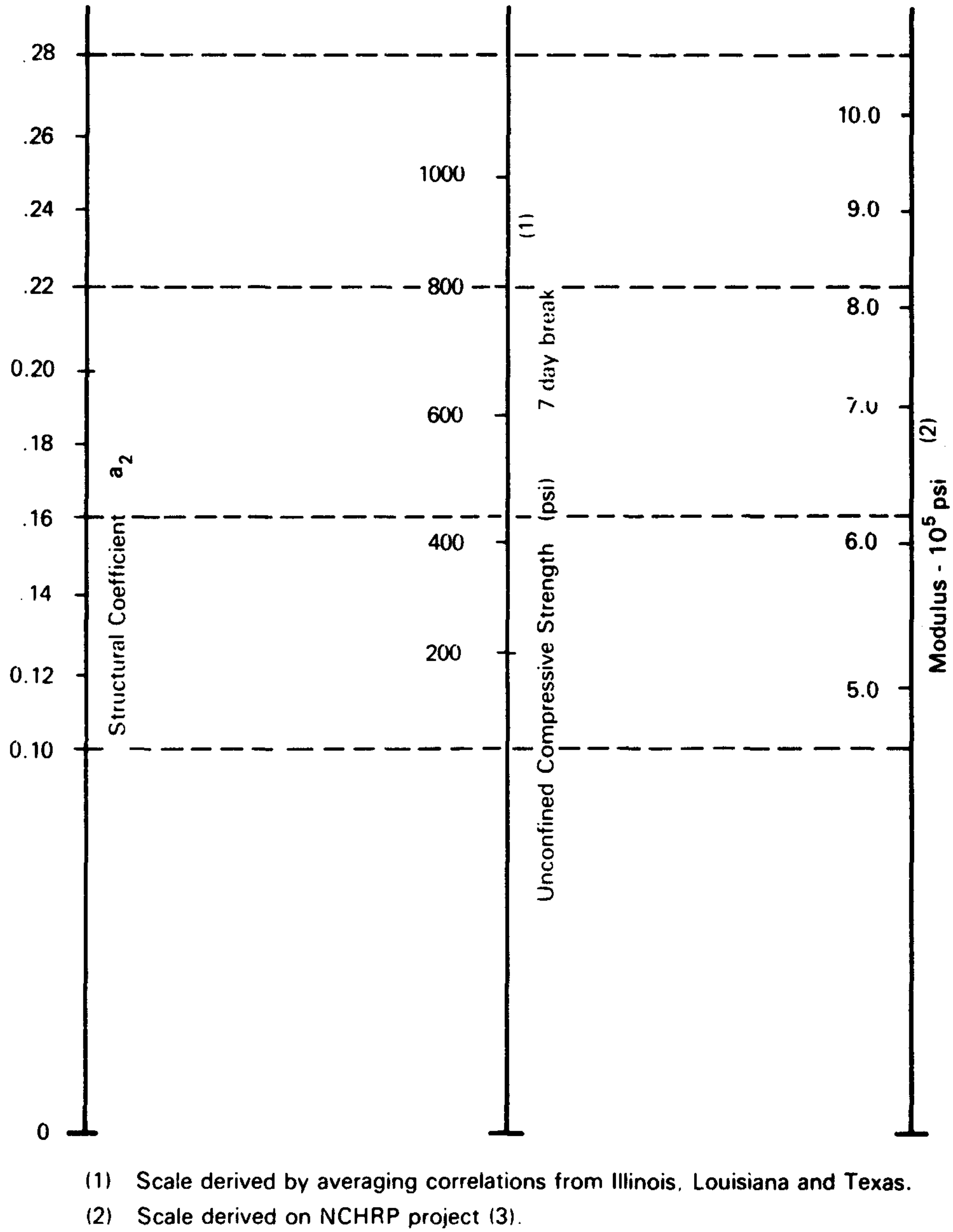


Figure 5-20. Correlations between structural layer coefficient a_2 and various strength and stiffness parameters for cement-treated granular bases (AASHTO, 1993).

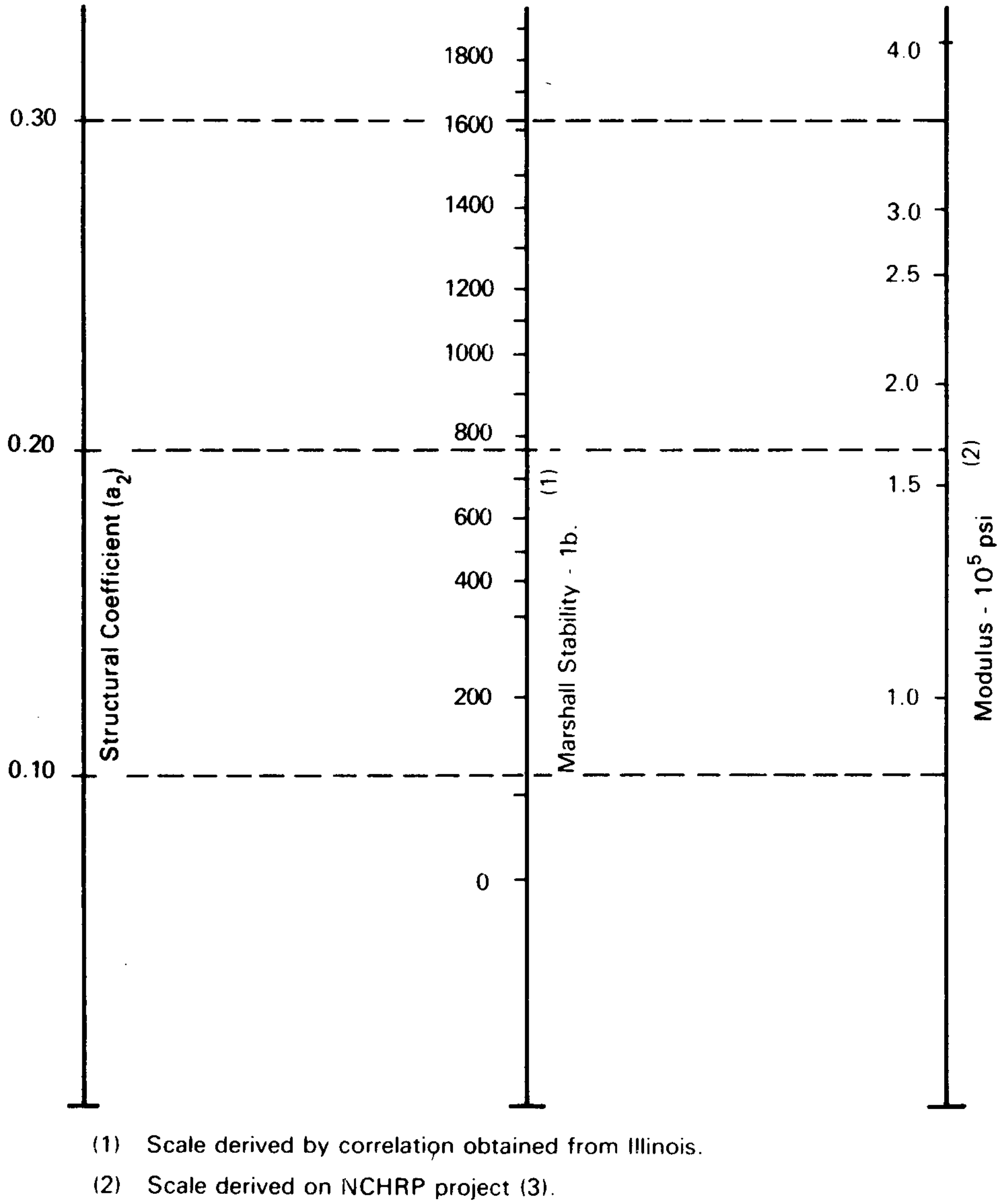


Figure 5-21. Correlations between structural layer coefficient a_2 and various strength and stiffness parameters for bituminous-treated granular bases (AASHTO, 1993).

The relationship in the 1993 AASHTO Guide between the structural layer coefficient a_3 and resilient modulus E_{SB} (in psi) for granular subbase materials is given as

$$a_3 = 0.227\log_{10} E_{SB} - 0.839 \tag{5.18}$$

The resilient modulus E_{SB} for granular subbase layers is influenced by stress state in a manner similar to that for the base layer, as given in Eq. (5.17). Typical values for the k_1 and k_2 material properties for granular subbases are

k_1 = 1500 to 6000

k_2 = 0.4 to 0.6

The values of E_{SB} from subbase layers in the original AASHO Road Test are summarized in Table 5-42. Note that the E_{SB} values are not only functions of moisture, but also of stress state θ, which in turn is a function of the pavement structure – *i.e.,* thickness of the asphalt concrete surface layer. Typical values of θ recommended in the 1993 AASHTO Guide for use in subbase design are summarized in Table 5-43. Figure 5-22 summarizes relationships between the a_3 structural layer coefficient for granular subbase layers and corresponding values of CBR, R-Value, Texas Triaxial strength, and resilient modulus.

Table 5-42. Granular subbase resilient modulus E_{SB} values (psi) from AASHO Road Test (AASHTO, 1993).

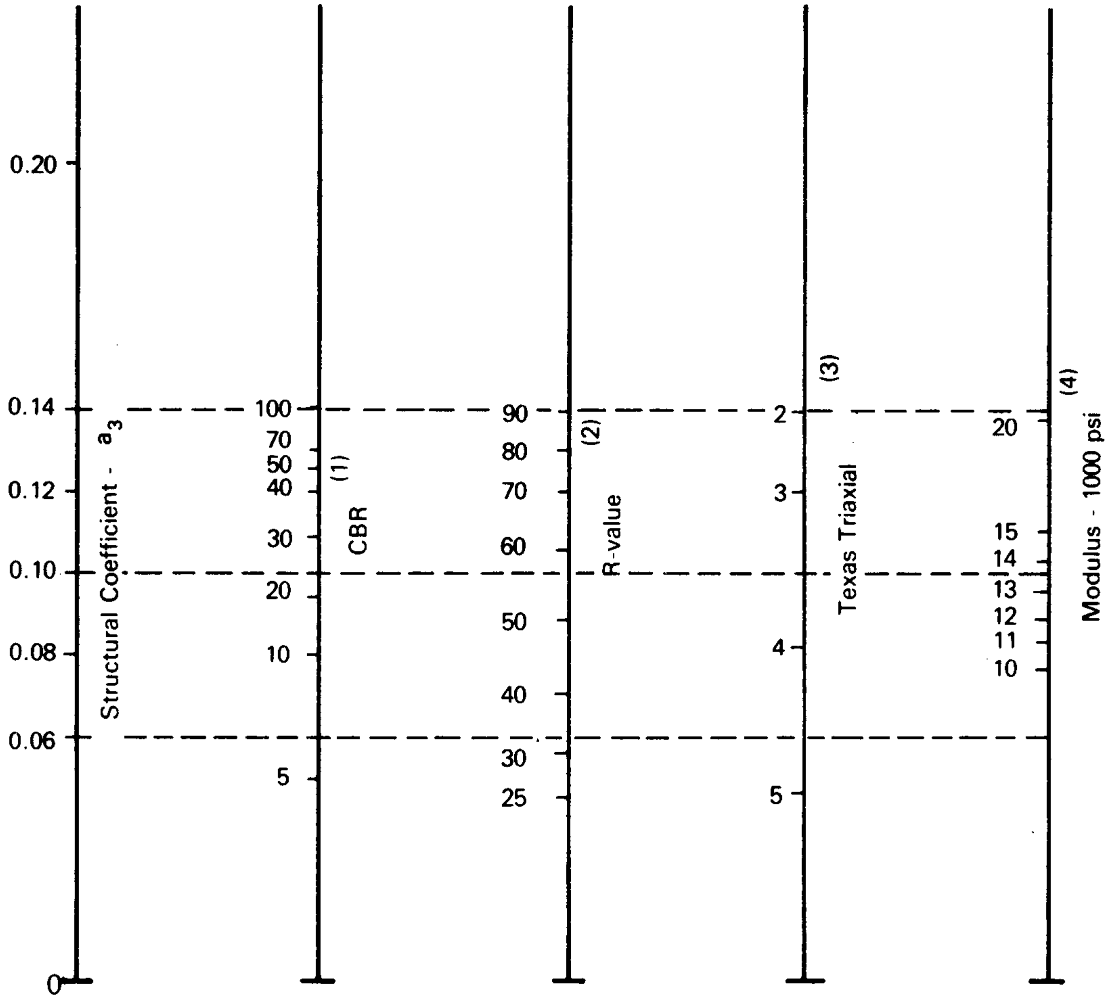
Moisture State	Equation	Stress State (psi*)		
		$\theta = 5$	$\theta = 7.5$	$\theta = 10$
Damp	$5{,}400\theta^{0.6}$	14,183	18,090	21,497
Wet	$4{,}600\theta^{0.6}$	12,082	15,410	18,312

*1 psi = 6.9 kPa

Table 5-43. Suggested bulk stress θ (psi) values for use in design of granular subbase layers (AASHTO, 1993).

Asphalt Concrete Thickness (inches*)	Stress State (psi*)
< 2	10.0
2 – 4	7.5
> 4	4.0

*1 inch = 25.4 mm; 1 psi = 6.9 kPa



(1) Scale derived from correlations from Illinois.

(2) Scale derived from correlations obtained from The Asphalt Institute, California, New Mexico and Wyoming.

(3) Scale derived from correlations obtained from Texas.

(4) Scale derived on NCHRP project *(3)*.

Figure 5-22. Correlations between structural layer coefficient a_3 and various strength and stiffness parameters for unbound granular subbases (AASHTO, 1993).

Rehabilitation

Depending on the types and amounts of deterioration present, the layer coefficient values assigned to materials in in-service existing pavements should in most cases be less than the values that would be assigned to the same materials for new construction. Exceptions to this general rule would include unbound granular materials that show no sign of degradation or contamination.

Limited guidance is available for the selection of layer coefficients for in-service pavement materials. Recommendations from the 1993 AASHTO Pavement Design Guide are provided in Table 5-44. In addition to evidence of pumping noted during a visual condition survey, samples of base and subbase materials should be obtained and examined for evidence of erosion, degradation, and contamination by fines, as well as evaluated for drainability, and layer coefficients should be reduced accordingly. Coring and testing are recommended for evaluation of all materials and are strongly recommended for evaluation of stabilized layers.

Table 5-44. Suggested layer coefficients for existing flexible pavement layer materials (AASHTO, 1993).

Material	Surface Condition	Coefficient
AC Surface	Little or no alligator cracking and/or only low-severity transverse cracking	0.35 – 0.40
	<10% low-severity alligator cracking and/or <5% medium- and high-severity transverse cracking	0.25 – 0.35
	>10% low-severity alligator cracking and/or <10 percent medium-severity alligator cracking and/or >5-10% medium- and high-severity transverse cracking	0.20 – 0.30
	>10% medium-severity alligator cracking and/or <10% high-severity alligator cracking and/or >10% medium- and high-severity transverse cracking	0.14 – 0.20
	>10% high-severity alligator cracking and/or >10% high-severity transverse cracking	0.08 – 0.15
Stabilized Base	Little or no alligator cracking and/or only low-severity transverse cracking	0.20 – 0.35
	<10% low-severity alligator cracking and/or <5% medium- and high-severity transverse cracking	0.15 – 0.25
	>10% low-severity alligator cracking and/or <10% medium-severity alligator cracking and/or >5-10% medium- and high-severity transverse cracking	0.15 – 0.20
	>10% medium-severity alligator cracking and/or <10% high-severity alligator cracking and/or >10% medium- and high-severity transverse cracking	0.10 – 0.20
	>10% high-severity alligator cracking and/or >10% high-severity transverse cracking	0.08 – 0.15
Granular Base/Subbase	No evidence of pumping, degradation, or contamination by fines	0.10 – 0.14
	Some evidence of pumping, degradation, or contamination by fines	0.00 – 0.10

5.4.6 Modulus of Subgrade Reaction

Mechanistic solutions for the stresses and strains in rigid pavements have historically characterized the stiffness of the foundation soil in terms of the modulus of subgrade reaction *k* (Figure 5-23). However, the modulus of subgrade reaction is not a true engineering property for the foundation soil because it depends not only upon the soil stiffness, but also upon the slab (or footing) size and stiffness. For an example of a square footing on a homogeneous isotropic elastic foundation soil, *k* can be expressed as

$$k = \frac{0.65E}{B\left(1-\nu^2\right)}\sqrt[12]{\frac{EB^4}{E_f I}} \tag{5.19}$$

in which

B	=	width of footing
E	=	elastic modulus of soil
ν	=	Poisson's ratio of soil
E_f	=	elastic modulus of footing
I	=	moment of inertia of footing = $\frac{Bt^3}{12}$, t = footing thickness

For a given slab/footing size and stiffness, *k* is directly proportional to the effective elastic modulus of the foundation soil in Eq. (5.19).

The effective modulus of subgrade reaction is a direct input in the AASHTO design procedures for rigid pavements (see Section 3.5.2). The modulus of subgrade reaction was first introduced in the 1972 version of the Guide, with the recommendation that its value be determined from plate loading tests. Beginning with the 1986 Guide, the recommended procedure for estimating *k* for new/reconstruction designs is through correlations with subgrade M_R plus various adjustments for base layer stiffness and thickness, presence of shallow rock, potential loss of slab support due to erosion, and seasonal variations.[2] The recommended procedure for determining *k* for rehabilitation designs is backcalculation from FWD test results.

[2] The 1998 supplement to the 1993 AASHTO Guide provides an alternate approach for determining the effective modulus of subgrade reaction for rigid pavements.

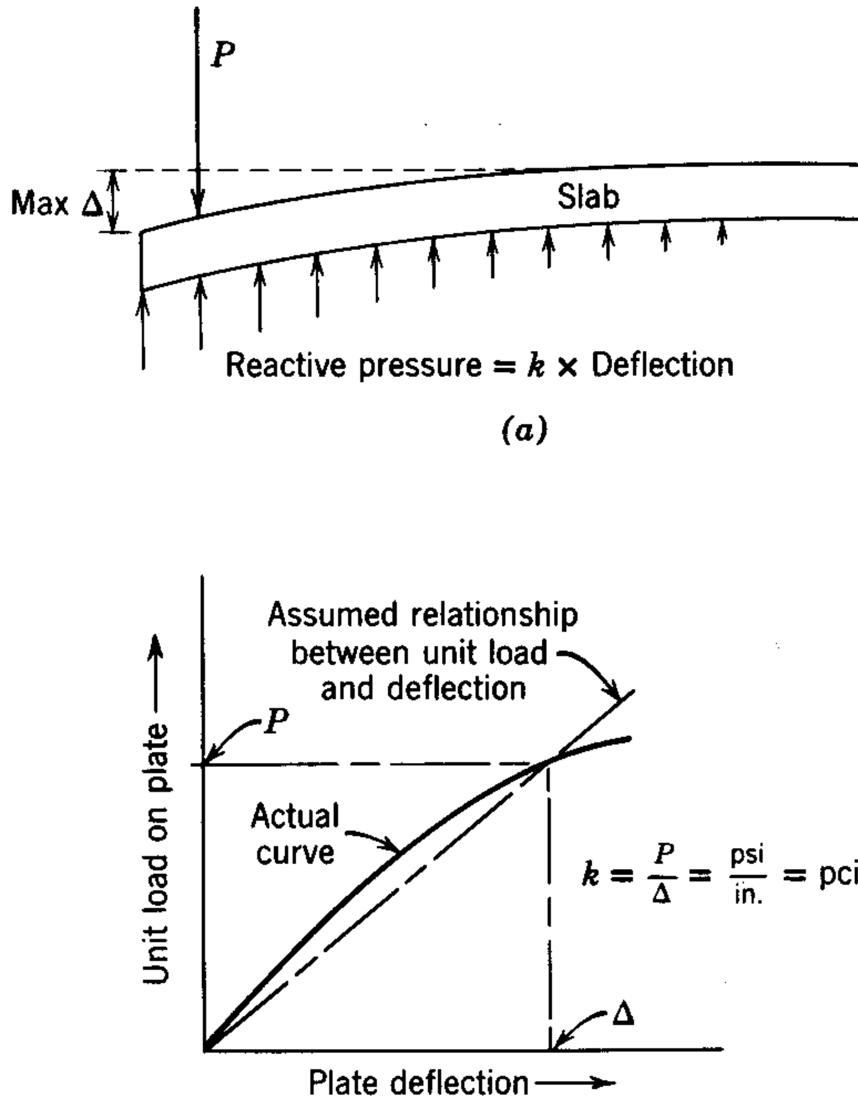


Figure 5-23. Coefficient of subgrade reaction *k* (Yoder and Witczak, 1975).

The subgrade, base, and subbase resilient moduli values are the direct inputs in the NCHRP 1-37A design methodology. These values are adjusted internally within the NCHRP 1-37A Design Guide software for environmental effects and then converted into an average monthly effective k-value for structural response calculation and damage analysis.

The detailed procedures used in the 1993 AASHTO and NCHRP 1-37A Design Guides to determine *k* for new/reconstruction and rehabilitation designs are described in the following subsections.

1993 AASHTO Guide

New Construction/Reconstruction

The steps recommended in the 1993 AASHTO Design Guide for determining the effective modulus of subgrade reaction for new/reconstruction designs are as follows:

1. Identify the subgrade and subbase type(s), thicknesses, and other properties.

2. Determine the subgrade resilient modulus M_R values for each season. Appropriate techniques for this are similar to those described earlier in Section 5.4.3.

3. Determine the subbase[3] resilient modulus E_{SB} for each season (similar to step 2). The 1993 AASHTO Guide recommends the following limits for the subbase resilient modulus:

$$15{,}000 \text{ (spring thaw)} < E_{SB} \text{ (psi)} < 50{,}000 \text{ (winter freeze)} \tag{5.20}$$

$$E_{SB} < 4M_R \text{ (psi)} \tag{5.21}$$

4. Using Figure 5-24, determine a composite k value for each season that represents the combined stiffness of the subgrade and subbase. Figure 5-24 is based on the following model (see Appendix LL in Volume 2 of AASHTO, 1986):

$$\begin{aligned}\ln k_\infty = -2.807 + 0.1253(\ln D_{SB})^2 + 1.062\left(\ln M_R\right) + 0.1282\left(\ln D_{SB}\right)\left(\ln E_{SB}\right) \\ - 0.4114\left(\ln D_{SB}\right) - 0.0581\left(\ln E_{SB}\right) - 0.1317\left(\ln D_{SB}\right)\left(\ln M_R\right)\end{aligned} \tag{5.22}$$

in which

k_∞ = composite modulus of subgrade reaction (pci) assuming a semi-infinite roadbed soil
D_{SB} = subbase thickness (inches)
E_{SB} = subbase elastic modulus (psi)
M_R = subgrade resilient modulus (psi)

5. Using Figure 5-25, correct the composite k values from step 4 for any effects of shallow bedrock. Figure 5-25 is based on the following model (see Appendix LL in Volume 2 of AASHTO, 1986):

[3] In the 1993 AASHTO Guide terminology, the subbase is defined as the granular layer between the PCC slab and the roadbed (subgrade) soil. This layer is termed the base layer in the 1998 supplement to the 1993 Guide.

$$\ln k_{rf} = 5.303 + 0.0710(\ln D_{SB})(\ln M_R) + 1.366(\ln k_\infty) - 0.9187(\ln D_{SG}) - 0.6837(\ln M_R) \quad (5.23)$$

in which

k_{rf} = composite modulus of subgrade reaction (pci) considering the effect of a rigid foundation near the surface

D_{SG} = depth to rigid foundation (inches)

and the other terms are as defined previously in Eq. (5.22).

6. Determine the seasonal average composite k value using the following procedure:
   - Estimate the design thickness of the slab and use Figure 5-26 to determine the relative damage u_{ri} for each season. Figure 5-26 is based on the following simplified damage model (see Appendix HH in Volume 2 of AASHTO, 1986):

     $$u_{ri} = \left[D^{0.75} - 0.39k_i^{0.25}\right]^{3.42} \quad (5.24)$$

     in which D is the slab thickness (inches) and k_i is the modulus of subgrade reaction for each season.
   - Compute the average relative damage $\overline{u}_r$ as the sum of the relative damage values for each season divided by the number of seasons.
   - Determine the seasonally averaged composite k from Figure 5-26 using $\overline{u}_r$ and the estimated slab thickness. This seasonally averaged composite k is termed the effective modulus of subgrade reaction k_{eff}.

7. Using Figure 5-27 and Table 5-45, correct the effective modulus of subgrade reaction k_{eff} for loss of support due to subbase erosion. This corrected k_{eff} is the value to be used for design. Table 5-46 summarizes the recommended design values for the modulus of subgrade reaction from the low-volume road section of the 1993 Design Guide.

Example:

D_{SB} = 6 inches

E_{SB} = 20,000 psi

M_R = 7,000 psi

Solution: k_∞ = 400 pci

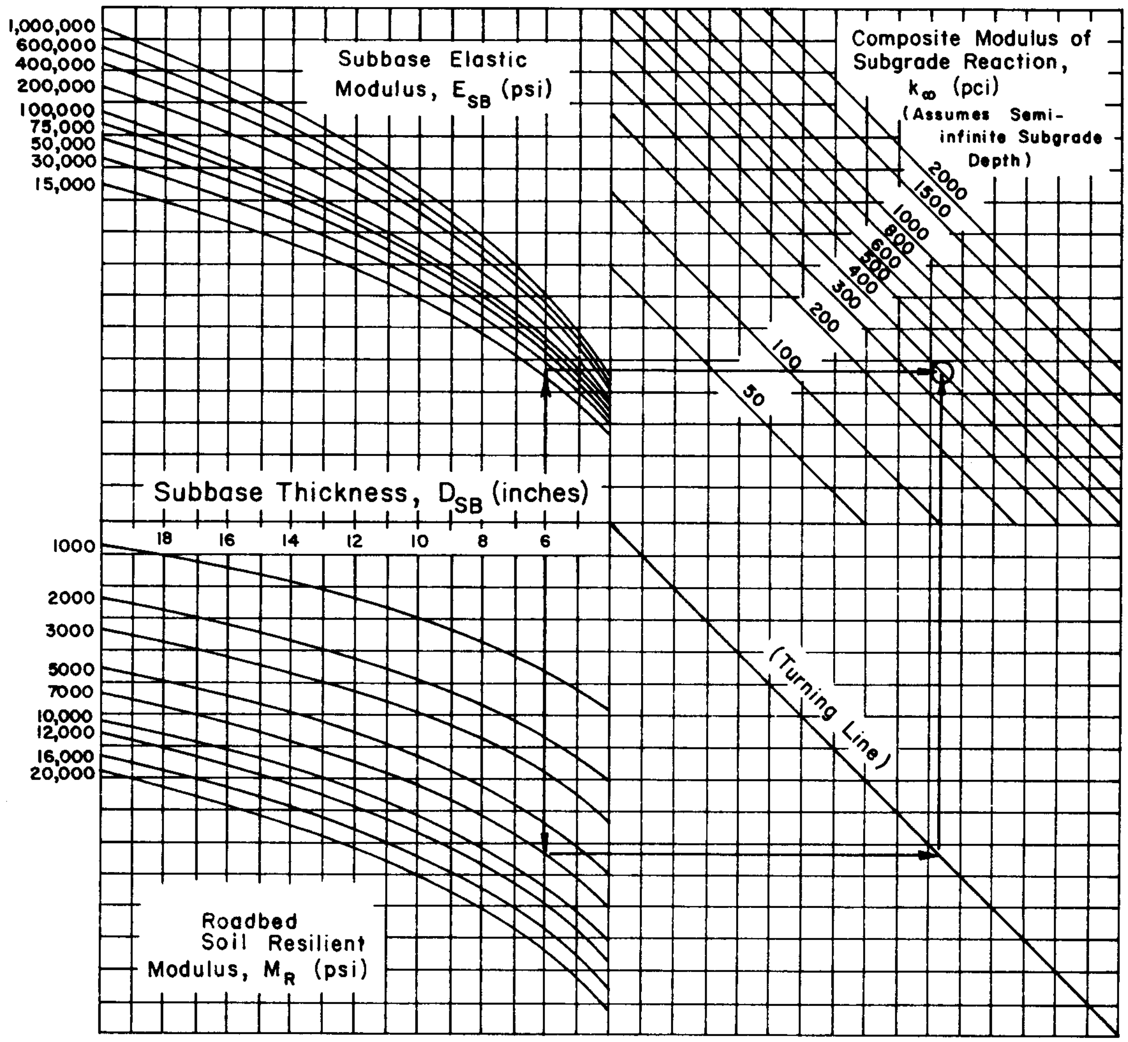


Figure 5-24. Chart for estimating composite modulus of subgrade reaction k_∞, assuming a semi-infinite subgrade depth (AASHTO, 1993).

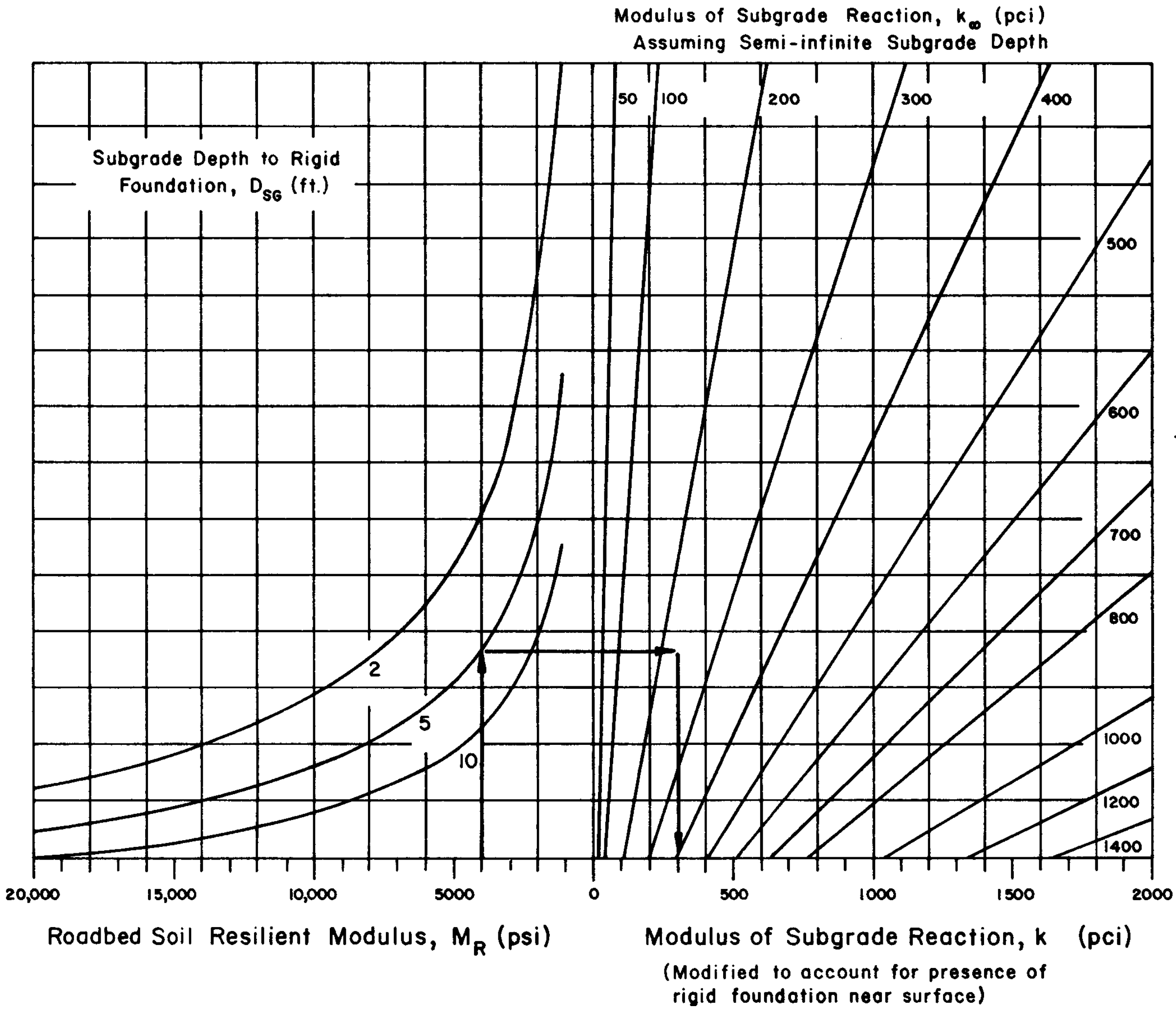


Figure 5-25. Chart to modify modulus of subgrade reaction to consider effects of rigid foundation near surface (within 10 ft) (AASHTO, 1993).

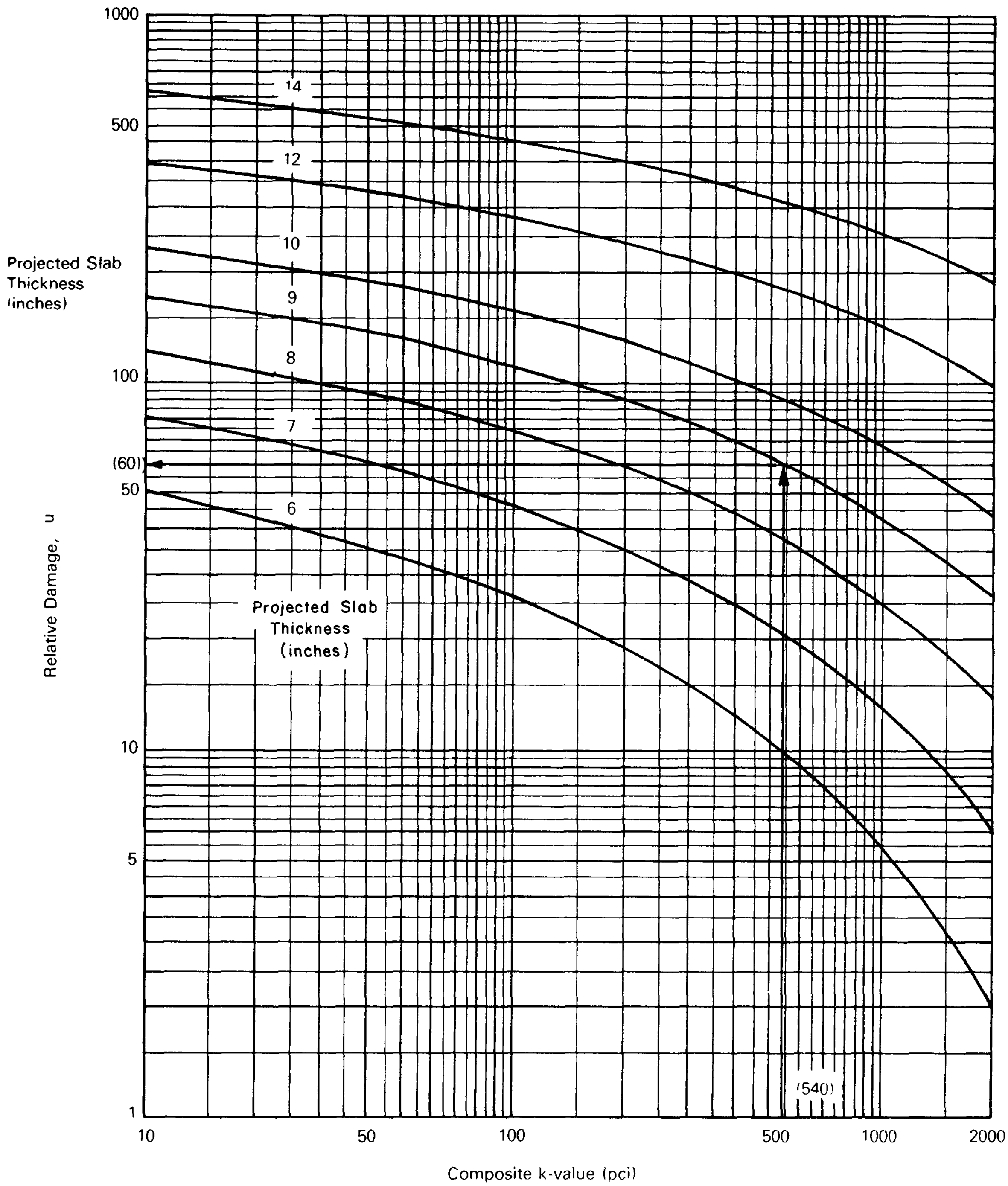


Figure 5-26. Chart for estimating relative damage to rigid pavements based on slab thickness and underlying support (AASHTO, 1993).

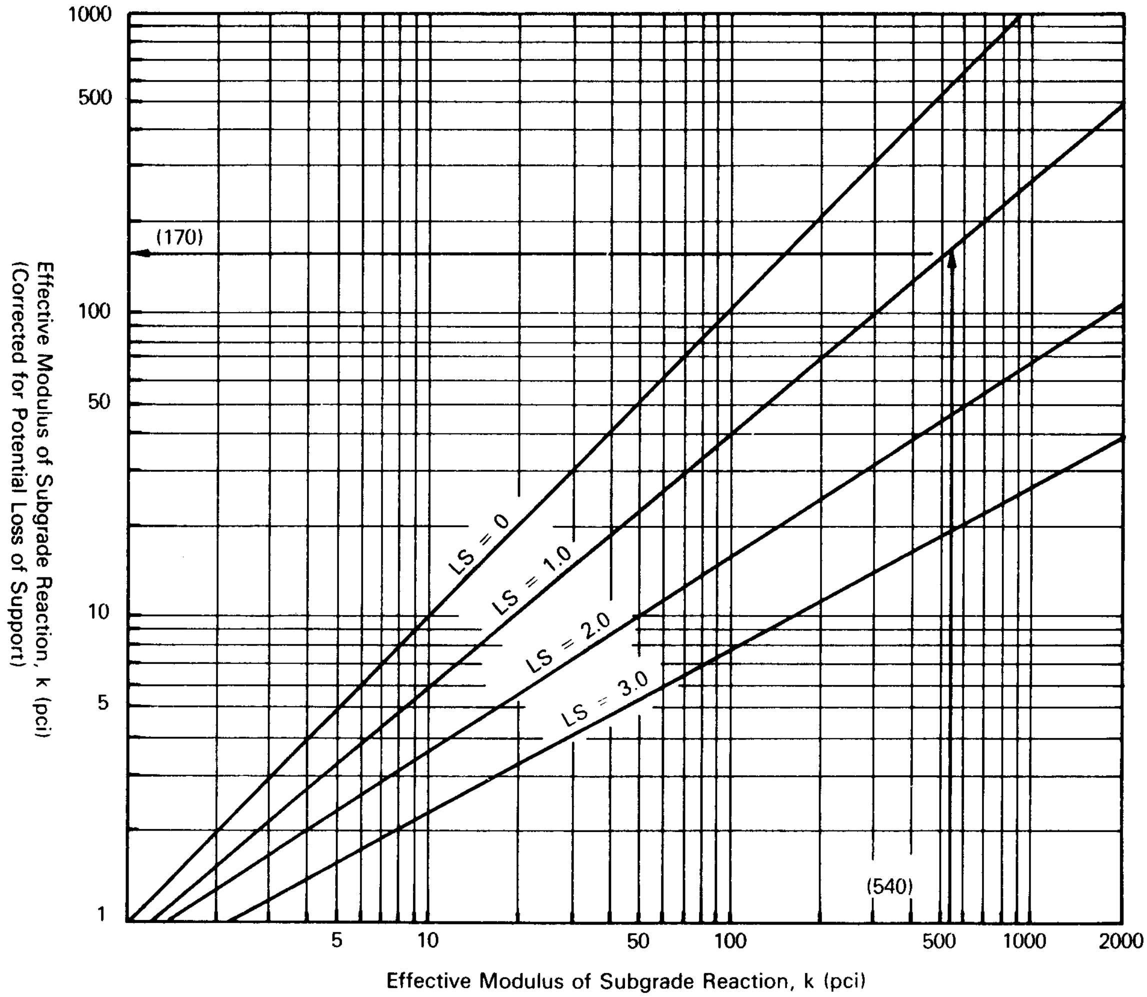


Figure 5-27. Correction of effective modulus of subgrade reaction for potential loss of subbase support (AASHTO, 1993).

Table 5-45. Typical ranges of loss of support *LS* factors for various types of materials (AASHTO, 1993).

Type of Material	Loss of Support (LS)
Cement treated granular base (E = 1,000,000 to 2,000,000 psi)	0.0 to 1.0
Cement aggregate mixtures (E = 500,000 to 1,000,000 psi)	0.0 to 1.0
Asphalt treated base (E = 350,000 to 1,000,000 psi)	0.0 to 1.0
Bituminous stabilized mixtures (E = 40,000 to 300,000 psi)	0.0 to 1.0
Lime stabilized (E = 20,000 to 70,000)	1.0 to 3.0
Unbound granular materials (E = 15,000 to 45,000 psi)	1.0 to 3.0
Fine grained or natural subgrade materials (E = 3,000 to 40,000 psi)	2.0 to 3.0

Table 5-46. Suggested ranges for modulus of subgrade reaction for design (AASHTO, 1993).

Roadbed Soil Quality	Range for k_{eff} (pci)
Very Good	> 550
Good	400 – 500
Fair	250 – 350
Poor	150 – 250
Very Poor	< 150

Rehabilitation

For rehabilitation projects, the modulus of subgrade reaction *k* can be determined from FWD deflection testing of the existing PCC pavement. An FWD with a load plate radius of 5.9 inches and a load magnitude of 9000 pounds is recommended, with deflections measured at sensors located at 0, 12, 24, and 36 inches from the center of the load along the outer wheel path. For each slab tested, a dynamic $k_{dynamic}$ value (pci) can be determined from Figure 5-28 based on the deflection at the center of the loading plate, d_0 (inches) and the AREA of the deflection basin as computed by[4]

$$AREA = 6\left[1 + 2\left(\frac{d_{12}}{d_0}\right) + 2\left(\frac{d_{24}}{d_0}\right) + \left(\frac{d_{36}}{d_0}\right)\right] \tag{5.25}$$

in which the d_i values are the deflections at i inches from the plate center. The static k_{eff} value for design is then determined as:

$$k_{eff} = \frac{k_{dynamic}}{2} \tag{5.26}$$

As is the case for new/reconstruction, this k_{eff} value may need to be adjusted for seasonal effects.

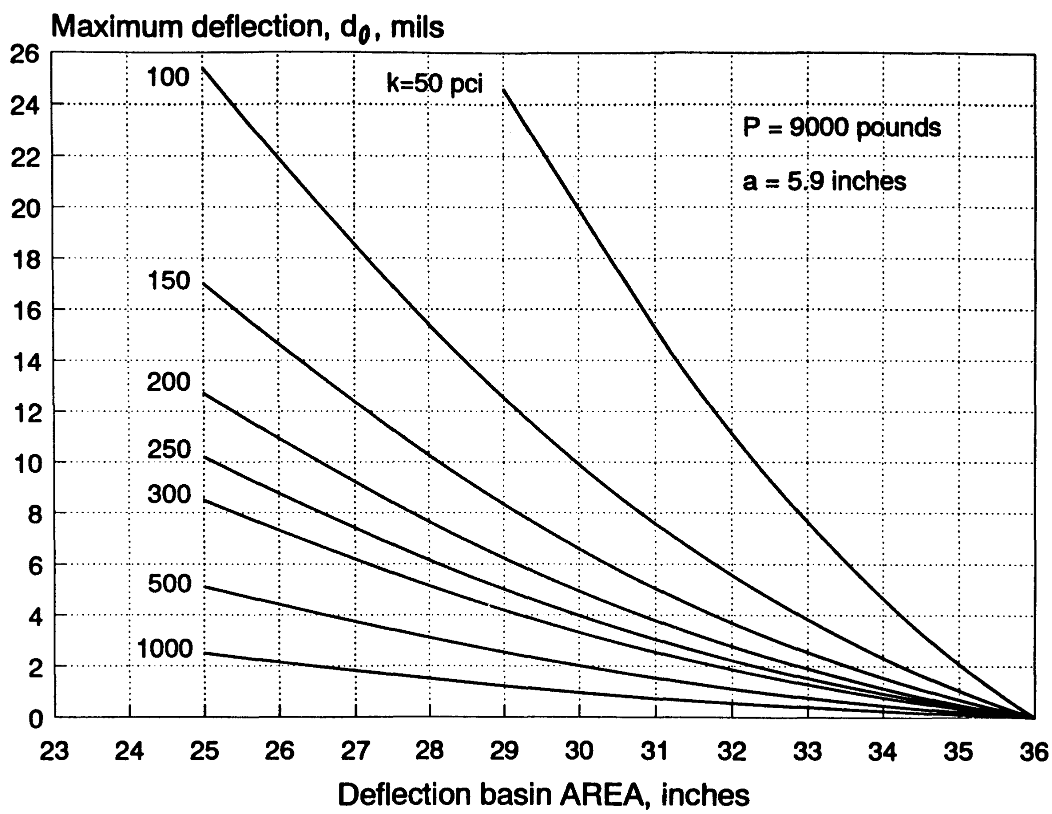


Figure 5-28. Effective dynamic *k* value determination from d_0 and *AREA* (AASHTO, 1993).

[4] For loads within 2000 pounds more or less of the 9000 pound desired value, deflections may be linearly scaled to equivalent 9000-pound deflections.

NCHRP 1-37A Design Guide

<u>New Construction/Reconstruction</u>

All subgrade and unbound pavement layers for all pavement types are characterized using M_R in the NCHRP 1-37A design methodology. The pavement response model for rigid pavement design, however, is based on a Winkler-spring foundation model that requires a value for the modulus of subgrade reaction $k_{dynamic}$ (see Appendix D for more details on the rigid pavement response model). The modulus of subgrade reaction is obtained from the subgrade and subbase M_R values and the subbase thickness through a conversion process that transforms the actual multilayer pavement structure into an equivalent three-layer structure consisting of the PCC slab, base, and an effective dynamic k, as shown in Figure 5-29. This conversion is performed internally in the NCHRP 1-37A Design Guide software as a part of input processing.

The procedure to obtain the effective value of $k_{dynamic}$ for each time increment in the analysis can be summarized in the following steps:

1. Assign initial estimates of the stiffness parameters M_R and ν to each unbound layer in the pavement structure.
2. Using multilayer elastic theory, simulate an FWD load and compute the stresses in the subgrade and subbase.
3. Adjust the subgrade and subbase M_R values to account for the stress states determined in Step 2.
4. Using multilayer elastic theory, again simulate an FWD load using the updated subgrade and subbase M_R values from Step 3. Calculate the PCC surface deflections at specified radii from the center of the load plate.
5. Using the rigid pavement response model, determine the $k_{dynamic}$ value that gives the best fit to the PCC surface deflections from Step 4.

The $k_{dynamic}$ value represents the compressibility of all layers beneath the base layer.
It is a computed quantity and not a direct input to the NCHRP 1-37A design procedure for new/reconstruction. Note also that $k_{dynamic}$ is a dynamic value, which should be distinguished from the traditional static k values used in previous design procedures.

The $k_{dynamic}$ value is calculated for each month of the year. It is used directly in the computation of the critical stresses, strains, and deflections for the incremental damage accumulation algorithms in the NCHRP 1-37A performance forecasting procedure. Environmental factors like water table depth, depth to bedrock, and freeze/thaw that can significantly affect the value for $k_{dynamic}$ are all considered in the NCHRP 1-37A calculations via the Enhanced Integrated Climate Model (EICM). Additional details of these algorithms are provided in Appendix D.

Rehabilitation

The modulus of subgrade reaction is a direct input for rigid pavement rehabilitation designs in the NCHRP 1-37A procedure. Measured surface deflections from FWD testing are used to backcalculate a $k_{dynamic}$ for design. The mean backcalculated $k_{dynamic}$ for a given month is input to the NCHRP 1-37A Design Guide software, and the $k_{dynamic}$ values for the remaining months of the year are seasonal adjustment factors computed by the EICM.

5.4.7 Interface Friction

1993 AASHTO Guide

The reinforcement design of jointed reinforced concrete pavements (JCRP) is dependent upon the frictional resistance between the bottom of the slab and the top of the underlying subbase or subgrade. This frictional resistance is characterized in the 1993 AASHTO Guide by a friction factor F that is related (but not equal) to the coefficient of friction between the slab and the underlying material. Recommended values for natural subgrade and a variety of subbase materials are presented in Table 5-47. The friction factor is required only for JCRP design.

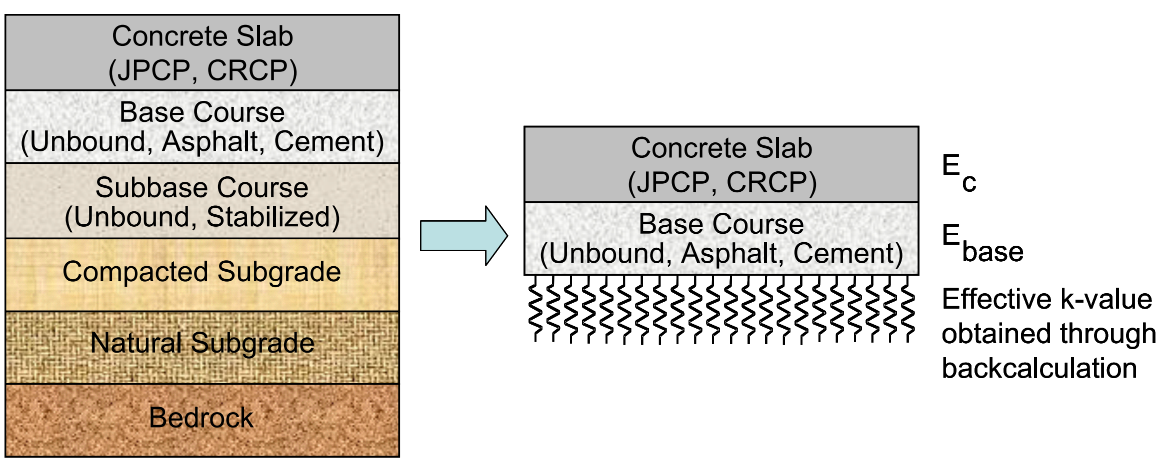


Figure 5-29. Structural model for rigid pavement structural response computations.

Table 5-47. Recommended friction factor values (AASHTO, 1993).

Type of Material Beneath Slab	Friction Factor *F*
Surface treatment	2.2
Lime stabilization	1.8
Asphalt stabilization	1.8
Cement stabilization	1.8
River gravel	1.5
Crushed stone	1.5
Sandstone	1.2
Natural subgrade	0.9

<u>NCHRP 1-37A Procedure</u>

The NCHRP 1-37A procedure for flexible pavements permits specification of the degree of bonding between each layer and the layer immediately beneath. The degree of bonding is characterized by an interface coefficient that varies between the limits of 1 for fully bonded conditions and 0 for a full slip interface. No guidance is provided at present in the NCHRP 1-37A procedure for specifying intermediate values for the interface coefficient to represent partial bond conditions between layers in flexible pavements.

The NCHRP 1-37A procedure for jointed plain concrete pavements (JPCP) requires specification of fully bonded for fully unbonded interface conditions between the bottom of the slab and the underlying layer. No provision is provided for intermediate bond conditions. The friction conditions at the bottom of continuously reinforced concrete pavements (CRCP) are specified in terms of a base/slab friction coefficient. Guidelines for specifying this coefficient are provided in Table 5-48. Jointed reinforced concrete pavement design is not included in the NCHRP 1-37A design procedures.

Table 5-48. Typical values of base/slab friction coefficient recommended for CRCP design in the NCHRP 1-37A procedure (NCHRP 1-37A, 2004).

Subbase/Base Type	Friction Coefficient (low – medium – high)
Fine grained soil	0.5 – 1.1 – 2.0
Sand*	0.5 – 0.8 – 1.0
Aggregate	0.5 – 2.5 – 4.0
Lime stabilized clay*	3.0 – 4.1 – 5.3
Asphalt treated base	2.5 – 7.5 – 15
Cement treated base	3.5 – 8.9 – 13
Soil cement	6.0 – 7.9 – 23
Lime-cement-flyash	3.0 – 8.5 - 20
Lime-cement-flyash, not cured*	> 36

*Base type did not exist or was not considered in the NCHRP 1-37A calibration process.

5.4.8 Permanent Deformation Characteristics

The permanent deformation characteristics of unbound materials are used in the empirical rutting distress models in the NCHRP 1-37A design methodology. This information is not required for rigid pavement design in the NCHRP 1-37A Design Guide or at all in the 1993 AASHTO design procedure. Permanent deformation characteristics are measured via triaxial repeated load tests conducted for many cycles of loading; Figure 5-30 shows schematically the typical behavior measured in this type of test. The repeated load permanent deformation tests are very similar to the cyclic triaxial tests used to measure resilient modulus (see 5.4.3), except that the cyclic deviator stress magnitude is kept constant throughout the test. There are at present no ASTM or AASHTO test specifications for repeated load permanent deformation testing. However, the first 1000 conditioning cycles of the AASHTO T307-99 resilient modulus testing procedure are often used for permanent deformation modeling.

The NCHRP 1-37A design methodology characterizes the permanent deformation behavior of unbound base, subbase, and subgrade materials using a model based on work by Tseng and Lytton (1989):

$$\delta_a(N) = \xi_1 \left(\frac{\varepsilon_o}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{N}\right)^{\xi_2 \beta}} \varepsilon_v h \tag{5.27}$$

in which

δ_a = Permanent deformation for the layer/sublayer

N = Number of traffic repetitions

ε_o, β, ρ = Material properties

ε_r = Resilient strain imposed in laboratory test to obtain material properties ε_o, β, and ρ

ε_v = Average vertical resilient strain in the layer/sublayer as obtained from the primary response model

h = Thickness of the layer/sublayer

ξ_1, ξ_2 = Field calibration functions

Tseng and Lytton provide regression equations for the $\varepsilon_o/\varepsilon_r$, ρ, and β terms. These regression equations were substantially revised during development of the NCHRP 1-37A design methodology. The revised equations implemented in the NCHRP 1-37A procedure are as follows:

$$\log\left(\frac{\varepsilon_o}{\varepsilon_r}\right) = 0.74168 + 0.08109 W_c - 0.000012157 M_R \tag{5.28}$$

$$\log \beta = -0.61119 - 0.017638 W_c \tag{5.29}$$

$$\log \rho = 0.622685 + 0.541524 W_c \tag{5.30}$$

In Eq. (5.28) through Eq. (5.30), M_R is the resilient modulus in psi, and W_c is an estimate of the average in-situ gravimetric water content in percent. The NCHRP 1-37A procedure proposes the following equation for determining W_c in the absence of measured values:

$$W_c = 51.712\left(CBR\right)^{-0.3586(GWT)^{0.1192}} \qquad W_c \le W_{sat} \tag{5.31}$$

In Eq. (5.31), GWT is the depth to the groundwater table in feet, and CBR can be estimated from resilient modulus using

$$CBR = \left(\frac{M_R}{2555}\right)^{(1/0.64)} \tag{5.32}$$

The W_{sat} limit in Eq. (5.31) can be determined from

$$W_{sat} = \left(\frac{2.75}{SPG} - 1 \right) * 100 / 2.75 \tag{5.33}$$

where *SPG* is the saturated specific gravity of the soil. For laboratory test conditions, W_c is presumably equal to the tested water content.

Although fine-tuning of the calibration is still underway and therefore the expressions for ξ_1, ξ_2 may yet change, the current best estimates are as follows:

$$\xi_1 = 1.2 - 1.39 e^{-0.058(M_R/1000)} \qquad \leq 1 \times 10^{-7} \tag{5.34}$$

$$\xi_2 = 0.7 \tag{5.35}$$

In Eq. (5.34), a lower bound of 2.6 is set for $M_R/1000$.

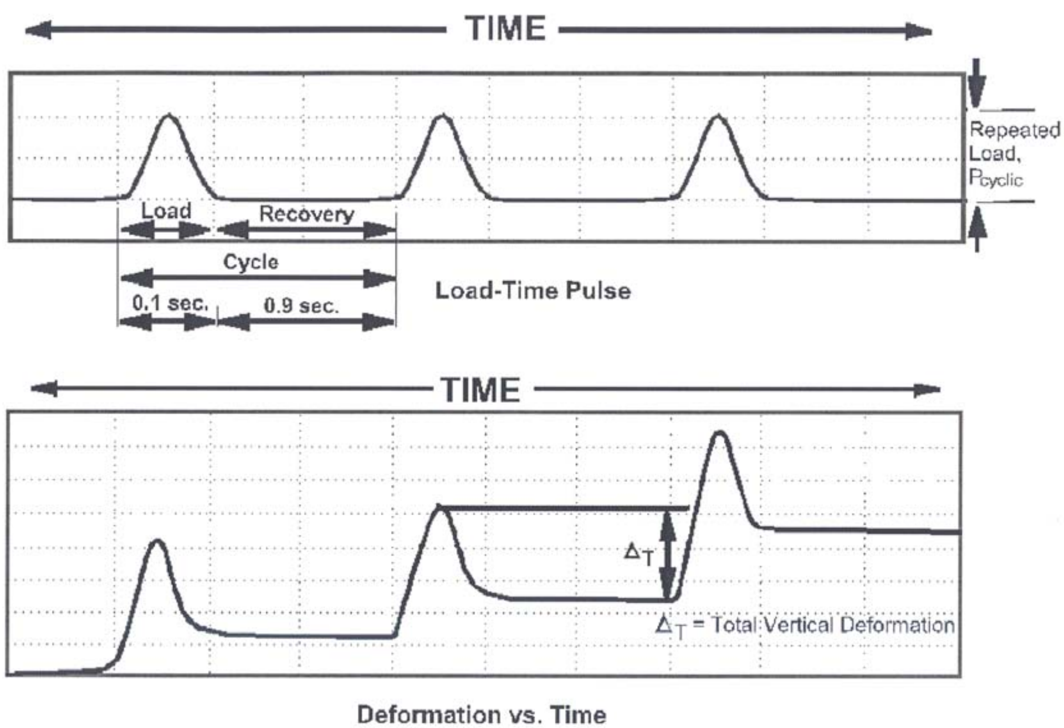


Figure 5-30. Accumulation of permanent deformations with repeated cyclic loading (LTPP, 2003).

5.4.9 Coefficient of Lateral Pressure

The coefficient of lateral earth pressure K_0 is defined as the ratio of the horizontal to vertical in-situ effective stress:

$$K_0 = \frac{\bar{\sigma}_{ho}}{\bar{\sigma}_{vo}} \tag{5.36}$$

The coefficient of lateral earth pressure is an input in the NCHRP 1-37A design procedure. It is used to compute the combined in-situ and induced stress states within the pavement system.

Elasticity theory can be used to estimate K_0 based on the confined Poisson expansion:

$$K_0 = \frac{\nu}{1-\nu} \tag{5.37}$$

in which ν is Poisson's ratio. Values of K_0 predicted by Eq. (5.37) for typical geomaterials range between 0.4 and 0.6.

A common empirical correlation for K_0 for cohesionless and normally consolidated cohesive soils is the Jaky relationship:

$$K_0 = 1 - \sin\phi \tag{5.38}$$

in which ϕ is the friction angle. Overconsolidation in cohesive soils will increase the value for K_0 above that given in Eq. (5.38). Figure 5-31 shows the typical relationship between K_0, the overconsolidation ratio OCR, and the plasticity index *PI*.

Loading followed by unloading and reloading, such as occurs during compaction of unbound materials in pavements, often results in an increase in K_0. The relative magnitudes of horizontal and vertical stress during a load-unload-reload path are shown schematically in Figure 5-32. Mayne and Kulhawy (1982) proposed the following model for K_0 after loading-unloading-reloading:

$$K_0 = \left(1 - \sin\phi\right)\left[\frac{OCR}{OCR_{\max}^{(1-\sin\phi)}} + \left(\frac{3}{4}\right)\frac{OCR}{OCR_{\max}}\right] \tag{5.39}$$

in which $OCR_{\max}$ is the maximum overconsolidation ratio achieved in the load path and the other terms are as defined previously.

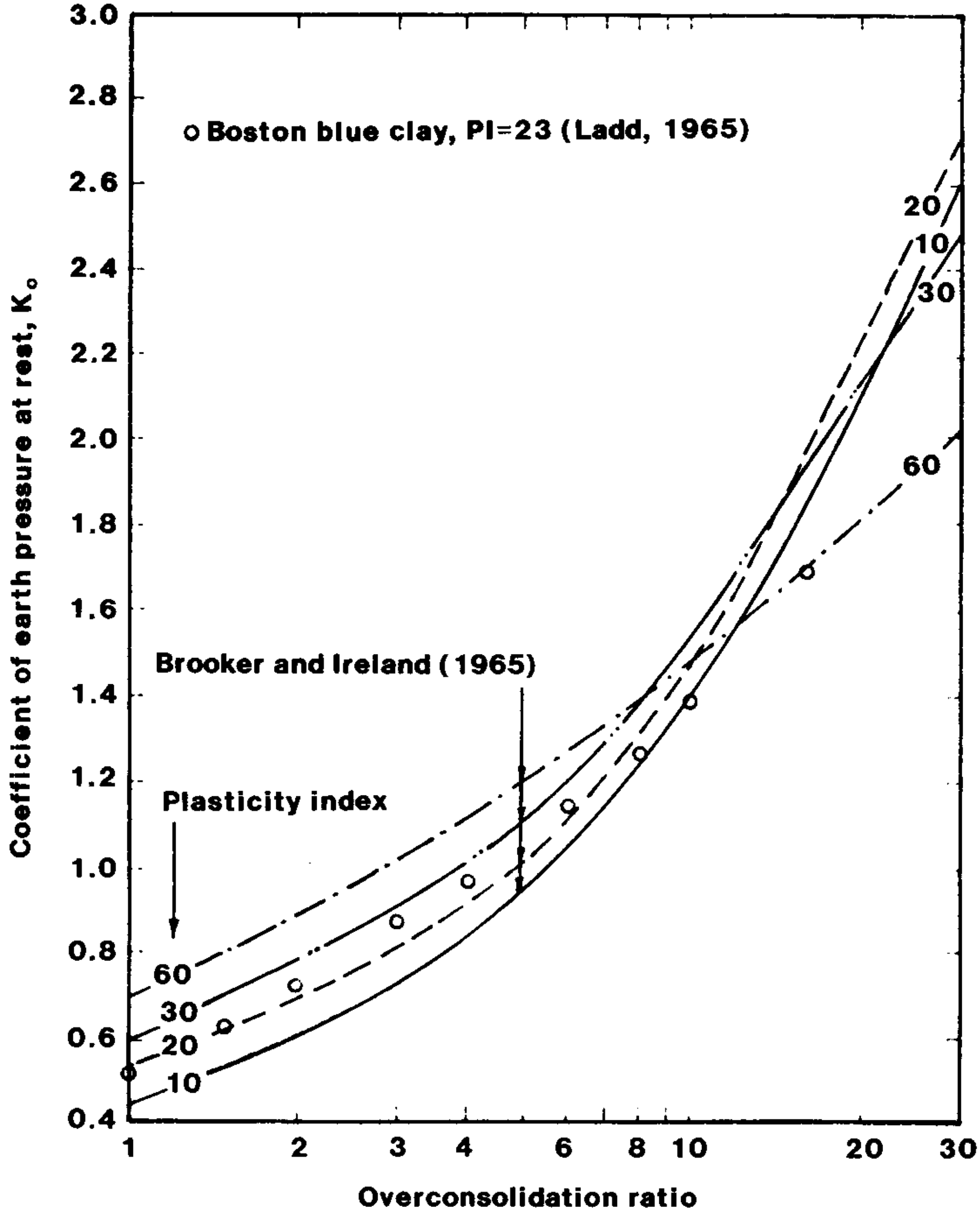


Figure 5-31. Correlation between coefficient of lateral earth pressure and overconsolidation ratio for clays of various plasticity indices (Carter and Bentley, 1991).

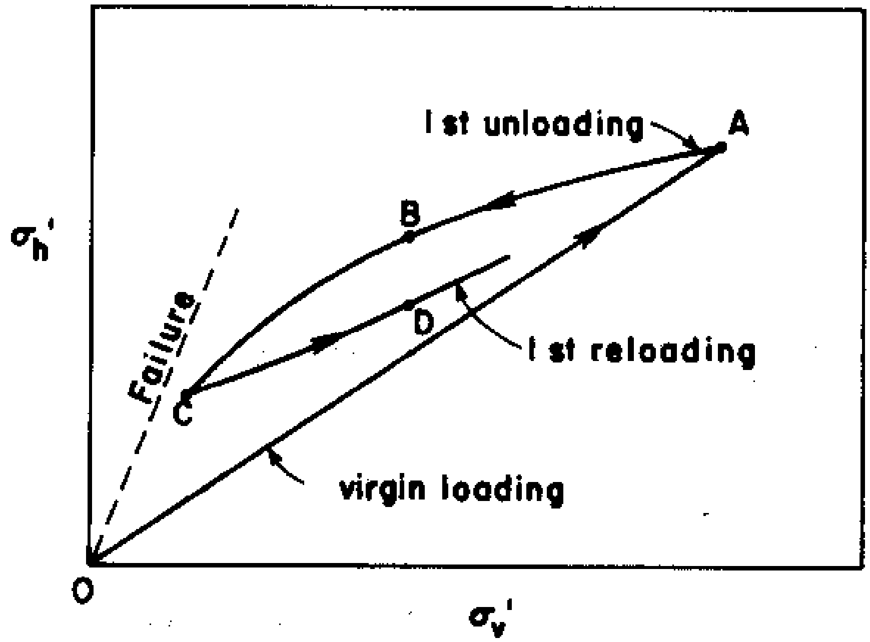


Figure 5-32. Horizontal and vertical in-situ stresses during a load-unload-reload path (Mayne and Kulhawy, 1982).

5.5 THERMO-HYDRAULIC PROPERTIES

Thermo-hydraulic material properties are required to evaluate the temperature and moisture conditions in a pavement system and their effects on the material behavior. Temperature has significant effects on the stiffness of asphalt concrete, and temperature gradients can induce thermal curling and stresses in rigid pavement slabs. Moisture content influences the stiffness and strength of unbound materials, and moisture gradients can induce warping of rigid pavement slabs. Combined temperature and moisture effects can cause detrimental freeze/thaw cycles in unbound materials.[5]

The empirical 1993 AASHTO Design Guide and the mechanistic-empirical NCHRP 1-37A design procedure have drastically different input requirements for thermo-hydraulic properties. The thermo-hydraulic design inputs in the 1993 AASHTO Guide are largely empirical coefficients grouped in the following categories:

- Drainage coefficients (for unbound layers)
- Swelling parameters (for expansive subgrade soils)
- Frost heave parameters (for frost-susceptible subgrade soils)

[5] Moisture and freeze/thaw are also important factors behind stripping of asphalt concrete, but this material phenomenon is beyond our scope.

These empirical properties in the 1993 Guide often mix material property and climate factors. For example, drainage coefficients are functions of both climate-determined moisture conditions and material-related drainage quality.

The thermo-hydraulic properties required as input to the NCHRP 1-37A Design Guide tend to be more fundamental material properties. These include

- Groundwater table depth
- Infiltration and drainage properties
- Physical properties
- Soil water characteristic curve
- Hydraulic conductivity (permeability)
- Thermal conductivity
- Heat capacity

These thermo-hydraulic properties are used in the mechanistic Enhanced Integrated Climate Model (EICM) along with climate inputs (discussed separately in Section 5.6) to predict temperature and moisture distributions in the pavement as functions of depth and time. Appendix D provides details on algorithms embedded in the EICM.

Because of the substantial differences in these thermo-hydraulic inputs to the two design methods, each design method is discussed separately in the following subsections.

5.5.1 1993 AASHTO Guide

The environment-related aspects in the 1993 AASHTO Design Guide are grouped into two general categories: drainage and subgrade swelling/frost heave. As described in Section 3.5.2 in Chapter 3, drainage is incorporated via adjustment to the unbound structural layer coefficients for flexible pavements or via a drainage factor in the design equation for rigid pavements. Swelling and/or frost heave, on the other hand, is incorporated via a partitioning of the total allowable serviceability loss ΔPSI; part of ΔPSI is allocated to environment-induced deterioration due to swelling and/or frost heave, and the remainder of ΔPSI is allocated to traffic-induced deterioration.

<u>Drainage Coefficients</u>

The 1993 AASHTO Guide provides guidance for the design of subsurface drainage systems and modifications to the flexible and rigid pavement design procedure to take advantage of improvements in performance due to good drainage. For flexible pavements, the benefits of drainage are incorporated into the structural number via empirical drainage coefficients:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \qquad (5.40)$$

in which m_2 and m_3 are the drainage coefficients for the base and subbase layers, respectively, and all other terms are as defined previously. Table 5-49 summarizes the recommended values for m_i in the 1993 AASHTO Guide as functions of qualitative descriptions of drainage quality and climate conditions.

For rigid pavements, the benefits of drainage are incorporated via an empirical drainage coefficient C_d in the rigid pavement design equation. Table 5-50 summarizes the recommended values for C_d in the 1993 AASHTO Guide as a function of qualitative descriptions of drainage quality and climate conditions.

Table 5-49. Recommended m_i values for modifying structural layer coefficients of untreated base and subbase materials in flexible pavements (AASHTO, 1993).

Quality of Drainage	Water Removed Within	Percent of Time Pavement is Exposed to Moisture Levels Approaching Saturation			
		<1%	1-5%	5-25%	>25%
Excellent	2 hours	1.40-1.35	1.35-1.30	1.30-1.20	1.20
Good	1 day	1.35-1.25	1.25-1.15	1.15-1.00	1.00
Fair	1 week	1.25-1.15	1.15-1.05	1.00-0.80	0.80
Poor	1 month	1.05-0.80	1.05-0.80	0.80-0.60	0.60
Very Poor	no drainage	0.95-0.75	0.95-0.75	0.75-0.40	0.40

Table 5-50. Recommended values of drainage coefficient C_d values for rigid pavement design (AASHTO, 1993).

Quality of Drainage	Water Removed Within	Percent of Time Pavement is Exposed to Moisture Levels Approaching Saturation			
		<1%	1-5%	5-25%	>25%
Excellent	2 hours	1.25-1.20	1.20-1.15	1.15-1.10	1.10
Good	1 day	1.20-1.15	1.15-1.10	1.10-1.00	1.00
Fair	1 week	1.15-1.10	1.10-1.00	1.00-0.90	0.90
Poor	1 month	1.10-1.00	1.00-0.90	0.90-0.80	0.80
Very Poor	no drainage	1.00-0.90	0.90-0.80	0.80-0.70	0.70

Swelling Parameters

The 1993 AASHTO Guide includes three empirical parameters for estimating potential serviceability loss due to swelling:

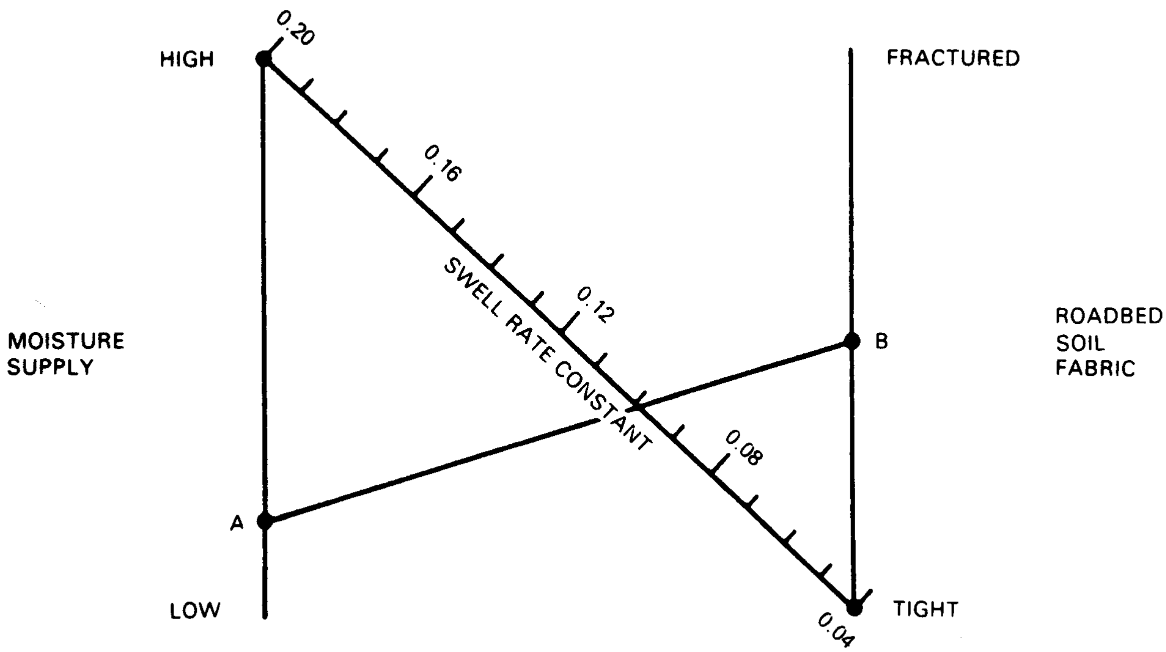
- Swell rate constant θ
- Potential vertical rise V_R
- Swell probability P_S

The swell rate constant θ is used to estimate the rate at which swelling will take place. It varies between 0.04 and 0.20, with higher values appropriate for soils exposed to a large moisture supply either due to high rainfall, poor drainage, or some other source. Figure 5-33 provides a nomograph for subjectively estimating the rate of subgrade soil swelling based upon qualitative descriptions of moisture supply and soil fabric. Little guidance beyond that in Figure 5-33 is provided in the 1993 Guide for estimating the values for moisture supply and soil fabric.

The potential vertical rise V_R is a measure of the vertical expansion that may occur in the subgrade soil under extreme swell conditions. Although it is possible to measure V_R from laboratory swell tests, this is not commonly done in practice. Instead, V_R is estimated using the chart in Figure 5-34 based on the soil's plasticity index, moisture condition, and overall thickness of the layer. The moisture condition is a subjective estimate of the difference between the in-situ moisture conditions during construction and moisture conditions at a later date.

The swell probability (P_S) is a measure of the proportion (percent) of the project length that is subject to swell. The probability of swelling at a given location is assumed to be 100% if the subgrade soil plasticity index is greater than 30 and the layer thickness is greater than 2 feet (or if V_R is greater than 0.20 inches). These criteria can be used to separate the project length into swelling and nonswelling sections, from which a length-averaged estimate of P_S can be determined.

These three swelling parameters are used in a nomograph (see Appendix C) along with the design life to determine the expected serviceability loss due to swelling ΔPSI_{SW}. However, it should be clear from the empirical and highly subjective procedures used to determine the input parameters that the predicted ΔPSI_{SW} will be only a very approximate estimate.



NOTES: a) LOW MOISTURE SUPPLY:

Low rainfall
Good drainage

b) HIGH MOISTURE SUPPLY

High rainfall
Poor drainage
Vicinity of culverts, bridge abutments, inlet leads

c) SOIL FABRIC CONDITIONS (self explanatory)

d) USE OF THE NONOGRAPH

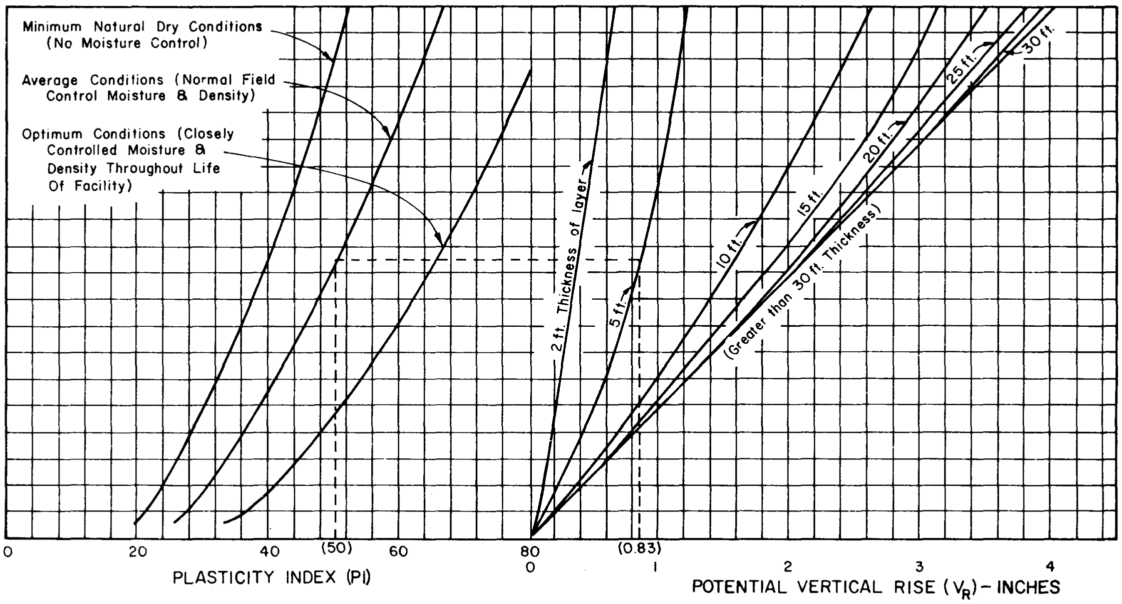
1) Select the appropriate moisture supply condition which may be somewhere between low and high (such as A).

2) Select the appropriate soil fabric (such as B). This scale must be developed by each individual agency.

3) Draw a straight line between the selected points (A to B).

4) Read swell rate constant from the diagonal axis (read 0.10).

Figure 5-33. Nomograph for estimating swell rate constant (AASHTO, 1993).



NOTES:
1. This figure is predicated upon the following assumptions:
   a. The subgrade soils for the thickness shown all are passing the No. 40 mesh sieve.
   b. The subgrade soil has a uniform moisture content and plasticity index throughout the layer thickness for the conditions shown.
   c. A surcharge pressure from 20 inches of overburden (± 10 inches will have no material effect).
2. Calculations are required to determine V_R for other surcharge pressures.

Figure 5-34. Chart for estimating potential vertical rise of natural soils (AASHTO, 1993).

<u>Frost Heave Parameters</u>

The 1993 AASHTO Guide includes three empirical parameters for estimating potential serviceability loss due to frost heave:

- Frost heave rate ϕ
- Maximum potential serviceability loss ΔPSI_{MAX}
- Frost heave probability P_F

The frost heave rate ϕ is a measure of the rate of increase of frost heave in millimeters per day. The rate of frost heave depends on the type of subgrade material, in particular the percentage of fine-grained material. Figure 5-35 can be used to estimate the rate of frost heave based on the USCS class for the subgrade and the percentage of material finer than 0.02 mm.

The maximum potential serviceability loss ΔPSI_{MAX} due to frost heave is dependent on the quality of drainage and the depth of frost penetration. Figure 5-36 can be used to estimate the maximum potential serviceability loss due to these two factors. The drainage quality parameter in Figure 5-36 is the same as that used to define the drainage coefficients in Table

5-49 and Table 5-50. See Yoder and Witczak (1975) for methods for determining the depth of frost penetration.

The frost heave probability P_F is the designer's estimate of the percentage length of the project that will experience frost heave. This estimate will depend upon the extent of frost-susceptible subgrade material, moisture availability, drainage quality, number of freeze-thaw cycles during the year, and the depth of frost penetration. Past experience is valuable here, as there is no clear method for approximating the frost heave probability.

These three frost heave parameters are used in a nomograph (see Appendix C) along with the design life to determine the expected serviceability loss due to frost heave ΔPSI_{FH}. However, it should be clear from the empirical and highly subjective procedures used to determine the input parameters that the predicted ΔPSI_{FH} will be only a very approximate estimate.

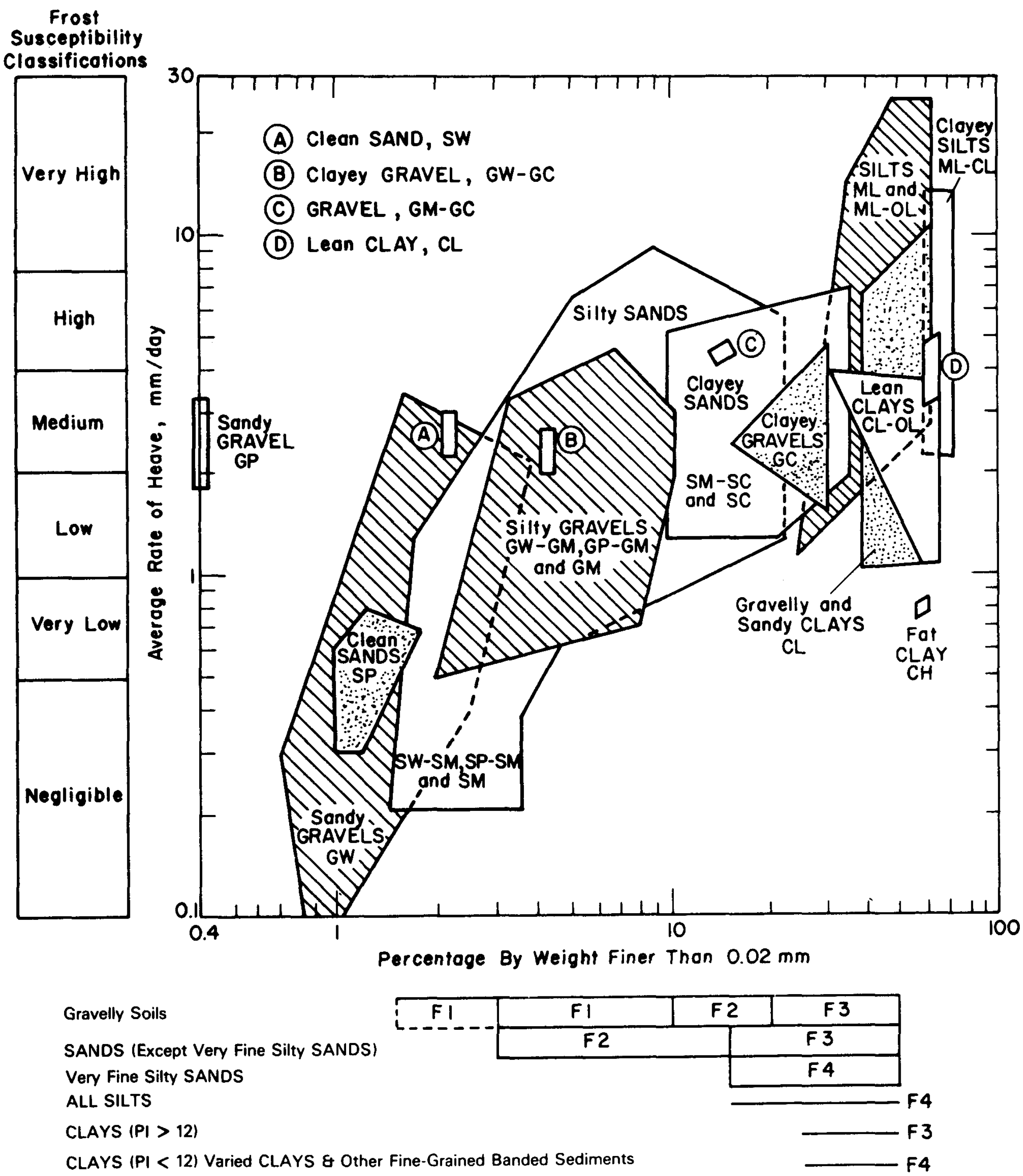


Figure 5-35. Chart for estimating frost heave rate for subgrade soil (AASHTO, 1993).

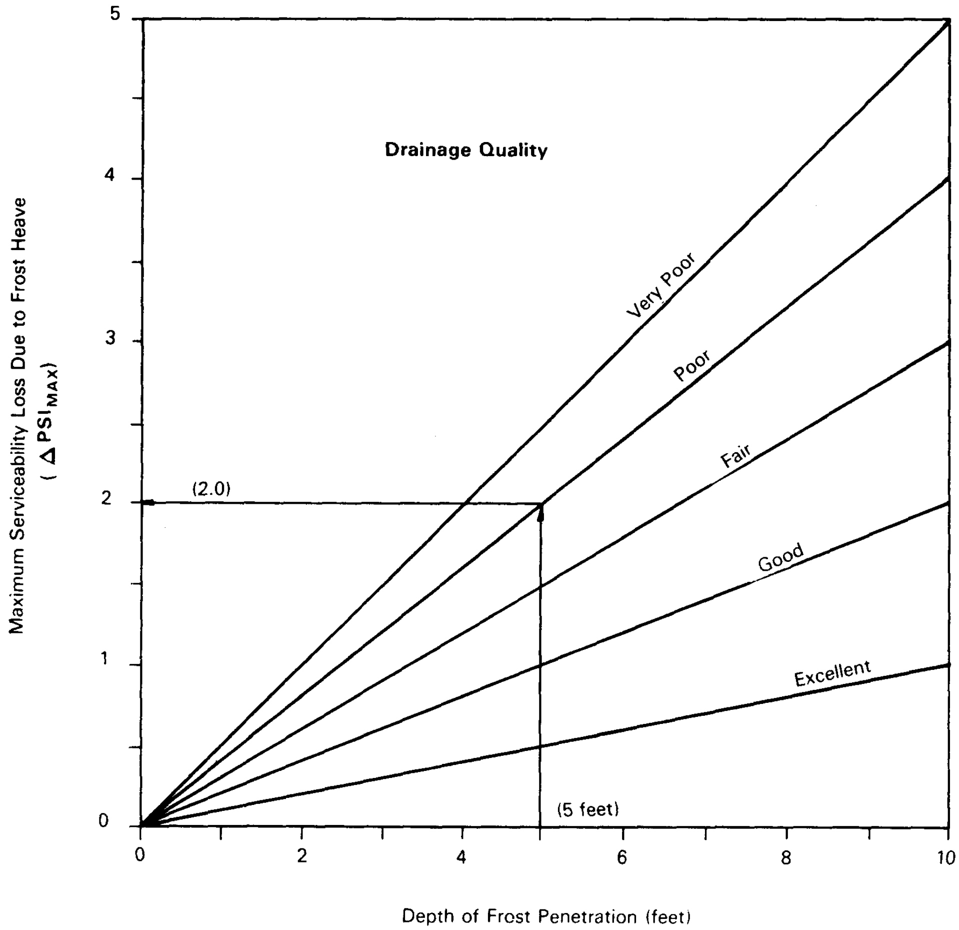


Figure 5-36. Graph for estimating maximum serviceability loss due to frost heave (AASHTO, 1993).

5.5.2 NCHRP 1-37A Design Guide

The thermo-hydraulic properties required as input to the NCHRP 1-37A Design Guide can be grouped into the following categories:

- Groundwater depth
- Infiltration and drainage properties
- Physical/index properties
- Soil water characteristic curve
- Hydraulic conductivity (permeability)
- Thermal conductivity
- Heat capacity

Methods for determining the design inputs in each of these categories are described in the following subsections. In some cases, the design inputs are determined by direct measurement in the laboratory or the field. However, other design inputs (*e.g.,* soil water characteristic curve) are much less commonly measured in geotechnical practice. Recognizing this, the NCHRP 1-37A project team expended substantial effort to develop robust correlations between these properties and other more conventional soil properties (*e.g.,* gradation and plasticity). These correlations are also detailed in the following subsections as appropriate.

Groundwater Depth

The groundwater depth plays a significant role in the NCHRP 1-37A Design Guide predictions of moisture content distributions in the unbound pavement materials and thus on the seasonal resilient modulus values. The input value is intended to be the best estimate of the annual average groundwater depth. Groundwater depth can be determined from profile characterization borings during design (see Section 4.7.1) or estimated. The county soil reports produced by the National Resources Conservation Service can often be used to develop estimates of groundwater depth.

Infiltration and Drainage

Three input parameters related to infiltration and drainage are required in the NCHRP 1-37A design methodology:

- Amount of infiltration
- Pavement cross slope
- Drainage path length

<u>Amount of Infiltration</u>

The amount of infiltration will be a function of rainfall intensity and duration (determined from the climate inputs, see Section 5.6), pavement condition, shoulder type, and drainage features. The NCHRP 1-37A Design Guide qualitatively divides infiltration into four categories, as summarized in Table 5-51. These categories are used at all hierarchical input levels. The infiltration category is based upon shoulder type, generally the largest single source of moisture entry into the pavement structure, and edge drains, since these shorten the drainage path and provide a positive drainage outlet. Note that if a drainage layer is present in addition to edge drains, its influence is automatically accounted for within the EICM moisture calculations.

Table 5-51. Infiltration categories in the NCHRP 1-37A Design Guide (NCHRP 1-37A, 2004).

Infiltration Category	Conditions	% Precipitation Entering Pavement
None		0
Minor	This option is valid when tied and sealed concrete shoulders (rigid pavements), widened PCC lanes, or full-width AC paving (monolithic main lane and shoulder) are used or when an aggressive policy is pursued to keep the lane-shoulder joint sealed. This option is also applicable when edge drains are used.	10
Moderate	This option is valid for all other shoulder types, PCC restoration, and AC overlays over old and cracked existing pavements where reflection cracking will likely occur.	50
Extreme	Generally not used for new or reconstructed pavement levels.	100

Most designs and maintenance activities, especially for higher functional class pavements, should strive to achieve zero infiltration or reduce it to a minimum value. This can be done by proper design of surface drainage elements (cross slopes, side ditches, etc.), adopting construction practices that reduce infiltration (*e.g.,* eliminating cold lane/shoulder joints, use of tied joints for PCC pavements, etc.), proactive routine maintenance activities (*e.g.,* crack and joint sealing, surface treatments, etc.), and providing adequate subsurface drainage (*e.g.,* drainage layers, edge drains). Chapter 7 provides more information on pavement drainage systems.

<u>Pavement Cross Slope</u>

The pavement cross slope is the slope of the surface perpendicular to the direction of traffic. This input is used in computing the drainage path length, as described in the next subsection.

Drainage Path Length

The drainage path length is the resultant of the cross and longitudinal slopes of the pavement. It is measured from the highest point in the pavement cross section to the drainage outlet. This input is used in the EICM's infiltration and drainage model to compute the time required to drain an unbound base or subbase layer from an initially wet condition.

The DRIP computer program (Mallela *et al*., 2002) can be used to compute the drainage path length based on pavement cross and longitudinal slopes, lane widths, edge drain trench widths (if applicable, and cross section crown and superelevation). The DRIP program is provided as part of the NCHRP 1-37A Design Guide software.

Physical Properties

Several physical properties are required for the internal calculations in the EICM. For unbound materials, these are

- Specific gravity of solids G_s (see Table 5-10)
- Maximum dry unit weight $\gamma_{d\ max}$ (see Table 5-13)
- Optimum gravimetric moisture content w_{opt} (see Table 5-13)

Table 5-52 describes the procedures to obtain these physical property inputs for hierarchical input levels 1 and 2 (level 3 inputs are not applicable for this input category). From these properties, all other necessary weight and volume properties required in the EICM can be computed. These include

- Degree of saturation at optimum compaction (S_{opt})
- Optimum volumetric moisture content (θ_{opt})
- Saturated volumetric water content (θ_{sat})

For rehabilitation designs only, the equilibrium or in-situ gravimetric water content is also a required input. NCHRP 1-37A recommends that this value be estimated from direct testing of bulk samples retrieved from the site, or through other appropriate means.

Although the material properties of the lower natural subgrade layers are important to the overall response of the pavement, a lower level of effort is generally sufficient to characterize these deeper layers as compared to the overlying compacted materials. Level 1 inputs are thus generally not necessary for in-situ subgrade materials. NCHRP 1-37A recommends that only gradation properties and Atterberg limits be measured for the in-situ subgrade materials.

Table 5-52. Physical properties for unbound materials required for EICM calculations (NCHRP 1-37A, 2004).

Material Property	Input Level	Description
Specific gravity, G_s	1	A direct measurement using AASHTO T100 (performed in conjunction with consolidation tests – T180 for bases or T 99 for other layers). See Table 5-10.
	2	Determined from P_{200}[1] and PI[2] of the layer as below: 1. Determine P_{200} and PI. 2. Estimate G_s: $G_s = 0.041(P_{200} * PI)^{0.29} + 2.65$ (5.41)
	3	Not applicable.
Optimum gravimetric water content, w_{opt}, and maximum dry unit weight of solids, $(\gamma_d)_{max}$	1	Typically, AASHTO T180 compaction test for base layers and AASHTO T99 compaction test for other layers. See Table 5-13.
	2	Estimated from D_{60}[1], P_{200}[1] and PI[2] of the layer following these steps: 1. Determine PI, P_{200}, and D_{60}. 2. Estimate S_{opt}: $S_{opt} = 6.752\ (P_{200} * PI)^{0.147} + 78$ (5.42) 3. Estimate w_{opt}: If $P_{200} * PI > 0$ $w_{opt} = 1.3\ (P_{200} * PI)^{0.73} + 11$ (5.43) If $P_{200} * PI = 0$ $w_{opt\ (T99)} = 8.6425\ (D_{60})^{-0.1038}$ (5.44) If layer is not a base course $w_{opt} = w_{opt\ (T99)}$ (5.45) If layer is a base course $\Delta w_{opt} = 0.0156[w_{opt(T99)}]^2 - 0.1465 w_{opt(T99)} + 0.9$ (5.46) $w_{opt} = w_{opt\ (T99)} - \Delta w_{opt}$ (5.47) 4. Determine G_s using the level 2 procedure described in this table above. 5. Compute $(\gamma_d)_{max\ comp}$ at optimum moisture and maximum compacted density: $\gamma_{d\ max\ comp} = \dfrac{G_s \gamma_{water}}{1 + \dfrac{w_{opt} G_s}{S_{opt}}}$ (5.48) 6. Determine $(\gamma_d)_{max}$: If layer is a compacted material: $\gamma_{d\ max} = \gamma_{d\ max\ comp}$ (5.49) If layer is a natural in-situ material: $\gamma_{d\ max} = 0.9 \gamma_{d\ max\ comp}$ (5.50)
	3	Not applicable.

1 P_{200} and D_{60} can be obtained from a grain-size distribution test (AASHTO T 27)—see Table 5-19.

2 PI can be determined from an Atterberg limit test (AASHTO T 90)—see Table 5-21.

Soil Water Characteristic Curve

The soil water characteristic curve (SWCC) defines the relationship between water content and matric suction h for a given soil. Matric suction is defined as the difference between the pore air pressure u_a and pore water pressure u_w in a partially saturated soil:

$$h = \left(u_a - u_w\right) \tag{5.51}$$

This relationship is usually plotted as the variation of water content (gravimetric w, volumetric θ, or degree of saturation S) vs. soil suction (Figure 5-37). The SWCC is one of the primary material inputs used in the EICM to compute moisture distributions with depth and time. Although the SWCC can be measured in the laboratory (e.g., see Fredlund and Rahardjo, 1993), this is quite uncommon and rather difficult. Instead, empirical models are used to express the SWCC in terms of other, more easily measurable parameters. The EICM algorithms in the NCHRP 1-37A analysis procedure are based on a SWCC model proposed by Fredlund and Xing (1994):

$$\theta_w = C(h) \times \left[\frac{\theta_{sat}}{\left[\ln\left[EXP(1) + \left(\frac{h}{a_f}\right)^{b_f} \right] \right]^{c_f}} \right] \tag{5.52}$$

$$C(h) = \left[1 - \frac{\ln\left(1 + \frac{h}{h_r}\right)}{\ln\left(1 + \frac{1.45 \times 10^5}{h_r}\right)} \right] \tag{5.53}$$

in which	h	= matric suction (units of stress)
	θ_{sat}	= volumetric moisture content at saturation
	a_f, b_f, c_f, and h_r	= model parameters (a_f, h_r in units of stress)

Table 5-53 summarizes the NCHRP 1-37A recommended approach for estimating the parameters of the SWCC at each of the three hierarchical input levels.

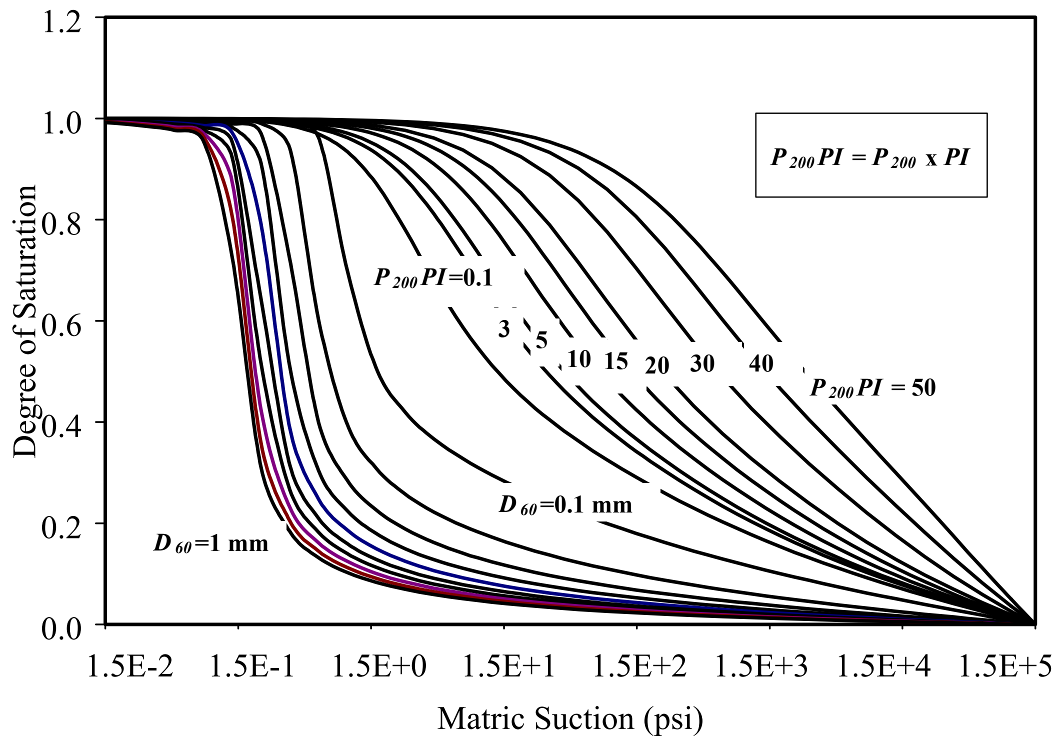


Figure 5-37. Soil water characteristic curves (NCHRP 1-37A, 2004).

Hydraulic Conductivity (Permeability)

Hydraulic conductivity (or permeability) k describes the ability of a material to conduct fluid (water). It is defined as the quantity of fluid flow through a unit area of soil under a unit pressure gradient. Hydraulic conductivity is one of the primary material inputs to the environment model in the NCHRP 1-37A analysis procedure, where it is used to determine the transient moisture profiles in unbound materials and to estimate their drainage characteristics.

The unsaturated flow algorithms in the EICM require a complete specification of the unsaturated hydraulic conductivity as a function of matric suction h. Although the unsaturated hydraulic conductivity vs. matric suction relationship can be measured in the laboratory (*e.g.,* see Fredlund and Rahardjo, 1993), this is uncommon and difficult. At best, only the saturated hydraulic conductivity k_{sat} is measured in practice. Consequently, within the EICM an empirical model proposed by Fredlund *et al.* (1994) is used to express the unsaturated hydraulic conductivity $k(h)$ vs. matric suction h relationship in terms of the Fredlund and Xing (1994) SWCC model, Eqs. (5.52) and (5.53). The $k(h)$ model is expressed in terms of a relative hydraulic conductivity:

$$k_r(h) \equiv k(h) / k_{sat} \qquad \textbf{(5.54)}$$

Table 5-53. Options for estimating the SWCC parameters (NCHRP 1-37A, 2004).

Input Level	Procedure to Determine SWCC parameters	Required Testing
1	1) Direct measurement of suction (h) in psi, and volumetric water content (θ_w) pairs of values. 2) Direct measurement of optimum gravimetric water content, w_{opt} and maximum dry unit weight, $\gamma_{d\,max}$. 3) Direct measurement of the specific gravity of the solids, G_s. 4) Compute $\theta_{opt} = \frac{w_{opt}\gamma_{d\,max}}{\gamma_{water}}$ (5.55) 5) Compute $S_{opt} = \frac{\theta_{opt}}{1 - \frac{\gamma_{d\,max}}{\gamma_{water} G_s}}$ (5.56) 6) Compute $\theta_{sat} = \frac{\theta_{opt}}{S_{opt}}$ (5.57) 7) Using non-linear regression analysis, compute the SWCC model parameters a_f, b_f, c_f, and h_r from Eqs. (5.52) and (5.53) and the (h, θ_w) pairs of values obtained in Step 1.	Pressure plate, filter paper, and/or Tempe cell testing. AASHTO T180 or T99 for $\gamma_{d\,max}$ (see Table 5-13). AASHTO T100 for G_s (see Table 5-10).
2	1) Direct measurement of optimum gravimetric water content, w_{opt} and maximum dry unit weight, $\gamma_{d\,max}$. 2) Direct measurement of the specific gravity of the solids, G_s. 3) Direct measurement of P_{200}, D_{60}, and PI. The EICM will then internally do the following: a) Calculate $P_{200} * PI$. b) Calculate θ_{opt}, S_{opt}, and θ_{sat}, as described for level 1. c) Determine the SWCC model parameters a_f, b_f, c_f, and h_r in Eqs. (5.52) and (5.53) via correlations with $P_{200}PI$ and D_{60}. If $P_{200}\,PI > 0$ $a_f = \frac{0.00364(P_{200}PI)^{3.35} + 4(P_{200}PI) + 11}{6.895}$ (psi) (5.58) $\frac{b_f}{c_f} = -2.313(P_{200}PI)^{0.14} + 5$ (5.59) $c_f = 0.0514(P_{200}PI)^{0.465} + 0.5$ (5.60) $\frac{h_r}{a_f} = 32.44e^{0.0186(P_{200}PI)}$ (5.61) If $P_{200}\,PI = 0$ $a_f = \frac{0.8627(D_{60})^{-0.751}}{6.895}$ (psi) (5.62) $b_f = 7.5$ (5.63)	AASHTO T180 or T99 for $\gamma_{d\,max}$ (see Table 5-13). T100 for G_s (see Table 5-10). AASHTO T88 for P_{200} and D_{60} (see Table 5-19). AASTHO T90 for PI (see Table 5-21).

	$c_f = 0.1772\ln(D_{60}) + 0.7734$ (5.64) $\frac{h_r}{a_f} = \frac{1}{D_{60} + 9.7e^{-4}}$ (5.65)	
3	Direct measurement and input of P_{200}, PI, and D_{60}, after which the EICM uses correlations with $P_{200}PI$ and D_{60} to automatically generate the SWCC parameters for each soil as follows: 1) Compute G_s, as outlined in Table 5-52 for level 2. 2) Compute $P_{200} * PI$ 3) Estimate S_{opt}, w_{opt}, and $\gamma_{d\,max}$, as shown Table 5-52 for level 2. 4) Determine the SWCC model parameters a_f, b_f, c_f, and h_r via correlations with $P_{200}PI$ and D_{60}, as shown in this table for level 2.	AASHTO T88 for P_{200} and D_{60} (see Table 5-19). AASHTO T90 for PI (see Table 5-21).

Recommendations from NCHRP 1-37A for determining the k_{sat} value needed in Eq. (5.54) are summarized in Table 5-54. The Fredlund *et al.* (1994) model for $k_r(h)$ is then expressed in integral form as

$$k_r(h) = \frac{\displaystyle\int_h^{h_r} \frac{\theta(x) - \theta(h)}{x^2}\theta'(x)dx}{\displaystyle\int_{h_{aev}}^{h_r} \frac{\theta(x) - \theta_s}{x^2}\theta'(x)dx} \tag{5.66}$$

in which:

- $\theta(h)$ = volumetric water content as a function of matric suction, from the SWCC Eqs. (5.52) and (5.53)
- θ_s = saturated volumetric water content
- $\theta'(h)$ = derivative of the SWCC
- x = dummy integration variable corresponding to water content
- h_r = matric suction corresponding to the residual water content (i.e., the water content below which a large increase in suction is required to remove additional water)
- h_{ave} = the air-entry matric suction (i.e., the suction where air starts to enter the largest pores in the soil)

The procedures described in Table 5-53 are used in the EICM to determine the SWCC via Eqs. (5.52) and (5.53), which in turn is then used to determine the unsaturated hydraulic conductivity via Eq. (5.66). These calculations are performed internally within the EICM software.

Table 5-54. Options for determining the saturated hydraulic conductivity for unbound materials (NCHRP 1-37A, 2004).

Material Property	Input Level	Description
Saturated hydraulic conductivity, k_{sat}	1	Direct measurement using a permeability test (AASHTO T215)—see Table 5-55.
	2	Determined from P_{200}[1], D_{60}[1], and PI[2] of the layer as below: 1. Determine $P_{200}PI = P_{200} * PI$ 2. If $0 \le P_{200}PI < 1$ $k_{sat} = 118.11 \times 10^{\left[-1.1275(\log D_{60}+2)^2 + 7.2816(\log D_{60}+2) - 11.2891\right]}$ (5.67) Units: ft/hr Valid for $D_{60} < 0.75$ in If $D_{60} > 0.75$ in, set $D_{60} = 0.75$ mm 3. If $P_{200}PI \ge 1$ $k_{sat} = 118.11 \times 10^{\left[0.0004(P_{200}PI)^2 - 0.0929(P_{200}PI) - 6.56\right]}$ (ft/hr) (5.68)
	3	Not applicable.

[1] P_{200} and D_{60} can be obtained from a grain-size distribution test (Table 5-19)
[2] *PI* can be determined from an Atterberg limit test (Table 5-21).

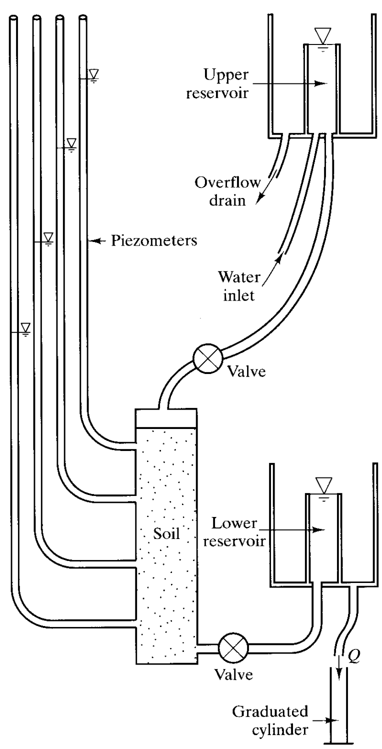


Figure 5-38. Schematic of a constant head permeameter (Coduto, 1999).

Table 5-55. Saturated hydraulic conductivity.

Description	Quantity of fluid flow through a unit area of soil under a unit pressure gradient.
Uses in Pavements	Used in the EICM for predicting distributions of moisture with depth and time in the NCHRP 1-37A Design Guide.
Laboratory Determination	AASHTO T 215; ASTM D 2434 (Granular Soils), ASTM D 5084 (All Soils). There are two basic standard types of test procedures to directly determine permeability: (1) the constant-head test, normally used for coarse materials (Figure 5-38); and (2) the falling-head test, normally used for clays (Figure 5-39). Undisturbed, remolded, or compacted samples can be used in both procedures.
Field Measurement	Pumping tests can be used to measure hydraulic conductivity in-situ.
Commentary	Both test procedures determine permeability of soils under specified conditions. The geotechnical engineer must establish which test conditions are representative of the problem under consideration. As with all other laboratory tests, the geotechnical engineer has to be aware of the limitations of this test. The process is sensitive to the presence of air or gases in the voids and in the permeant or water. Prior to the test, distilled, de-aired water should be run through the specimen to remove as much of the air and gas as practical. It is a good practice to use de-aired or distilled water at temperatures slightly higher than the temperature of the specimen. As the water permeates through the voids and cools, it will have a tendency to dissolve the air and some of the gases, thus removing them during this process. The result will be a more representative, albeit idealized, permeability value. The type of permeameter, (*i.e.,* flexible wall - ASTM D 5084 -versus rigid - ASTM D 2434 and AASHTO T215) may also affect the final results. For testing of fine-grained low-permeability soils, the use of flexible-wall permeameters is recommended, which are essentially very similar to the triaxial test apparatus. When rigid wall units are used, the permeant may find a route at the sample-permeameter interface. This will produce erroneous results. It should be emphasized that permeability is sensitive to viscosity. In computing permeability, correction factors for viscosity and temperatures must be applied. The temperature of the permeant and the laboratory should be kept constant during testing. Laboratory permeability tests produce reliable results under ideal conditions. Permeability of fine-grained soils can also be computed from one-dimensional consolidation test results, although these results are not as accurate as direct k_{sat} measurements.
Typical Values	See Table 5-56 and Table 5-57. Saturated hydraulic conductivity for loose clean sands can also be estimated using the Hazen relationship: $$k_{sat} = C * D_{10}^2 \quad (5.69)$$ in which k_{sat} is the saturated hydraulic conductivity in cm/sec; C is Hazen's coefficient ranging between 0.8 and 1.2 (a value of 1.0 is commonly used); and D_{10} is the effective particle size, defined as the largest particle diameter in the finest 10% fraction of the soil.

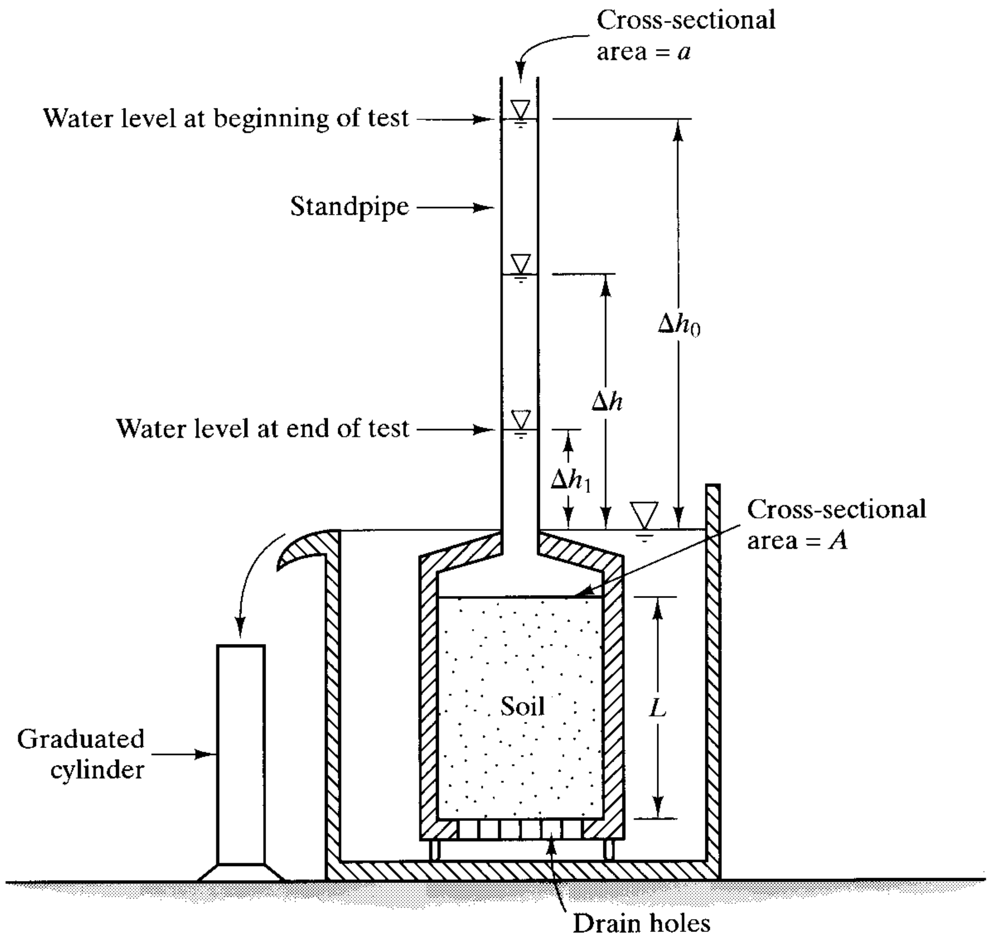


Figure 5-39. Schematic of a falling head permeameter (Coduto, 1999).

Table 5-56. Typical values of saturated hydraulic conductivity for soils (Coduto, 1999).

Soil Description	Hydraulic Conductivity k	
	(cm/s)	(ft/s)
Clean gravel	1 – 100	3×10^{-2} – 3
Sand-gravel mixtures	10^{-2} – 10	3×10^{-4} – 0.3
Clean coarse sand	10^{-2} – 1	3×10^{-4} – 3×10^{-2}
Fine sand	10^{-3} – 10^{-1}	3×10^{-5} – 3×10^{-3}
Silty sand	10^{-3} – 10^{-2}	3×10^{-5} – 3×10^{-4}
Clayey sand	10^{-4} – 10^{-2}	3×10^{-6} – 3×10^{-4}
Silt	10^{-8} – 10^{-3}	3×10^{-10} – 3×10^{-5}
Clay	10^{-10} – 10^{-6}	3×10^{-12} – 3×10^{-8}

Table 5-57. Typical values of saturated hydraulic conductivity for highway materials (Carter and Bentley, 1991).

Material	Hydraulic Conductivity k (m/s)*
Uniformly graded coarse aggregate	$0.4 – 4x10^{-3}$
Well-graded aggregate without fines	$4x10^{-3} – 4x10^{-5}$
Concrete sand, low dust content	$7x10^{-4} – 7x10^{-6}$
Concrete sand, high dust content	$7x10^{-6} – 7x10^{-8}$
Silty and clayey sands	$10^{-7} – 10^{-9}$
Compacted silt	$7x10^{-8} – 7x10^{-10}$
Compacted clay	$< 10^{-9}$
Bituminous concrete**	$4x10^{-5} – 4x10^{-8}$
Portland cement concrete	$< 10^{-10}$

*1 m/s = 3.25 ft/s

**New pavements; values as low as 10^{-10} have been reported for sealed, traffic-compacted highway pavements.

Thermal Conductivity

Thermal conductivity K is defined as the ability of a material to conduct heat. Typical units are BTU/ft-hr-°F or W/m- °K. Thermal conductivity is used in the EICM algorithms for the computation of temperature distributions with depth and time in the NCHRP 1-37A analysis methodology.

Table 5-58 outlines the NCHRP 1-37A recommended approach for characterizing the dry thermal conductivity K for unbound materials. Note that thermal conductivity is not commonly measured for unbound pavement materials, and consequently the level 3 inputs will be used for nearly all designs. The EICM automatically adjusts the dry thermal conductivity for the influence of moisture during the calculations.

Heat Capacity

Heat capacity Q is defined as the amount of heat required to raise by one degree the temperature of a unit mass of soil. Typical units are BTU/lb-°F or J/kg-°K. Heat capacity is used in the EICM algorithms for the computation of temperature distributions with depth and time in the NCHRP 1-37A analysis methodology.

Table 5-58 outlines the NCHRP 1-37A recommended approached for characterizing the dry heat capacity *Q* for unbound materials. Note that heat capacity is not commonly measured for unbound pavement materials, and consequently the level 3 inputs will be used for nearly all designs. The EICM automatically adjusts the dry heat capacity for the influence of moisture content during the calculations.

Table 5-58. Options for determining the dry thermal conductivity and heat capacity for unbound materials (NCHRP 1-37A, 2004).

Material Property	Input Level	Description
Dry Thermal Conductivity, K	1	Direct measurement (ASTM E 1952).
	2	Not applicable.
	3	<u>*Soil Type*</u> <u>*Range*</u> *Recommended* <u>*BTU/ft-hr-°F**</u> A-1-a 0.22 – 0.44 0.30 A-1-b 0.22 – 0.44 0.27 A-2-4 0.22 – 0.24 0.23 A-2-5 0.22 – 0.24 0.23 A-2-6 0.20 – 0.23 0.22 A-2-7 0.16 – 0.23 0.20 A-3 0.25 – 0.40 0.30 A-4 0.17 – 0.23 0.22 A-5 0.17 – 0.23 0.19 A-6 0.16 – 0.22 0.18 A-7-5 0.09 – 0.17 0.13 A-7-6 0.09 – 0.17 0.12 Additional typical values are given in Table 5-59.
Dry Heat Capacity, Q	1	Direct measurement (ASTM D 2766).
	2	Not applicable.
	3	Typical values range from 0.17 to 0.20 BTU/lb-°F. Additional typical values are given in Table 5-59.

*1 BTU/ft-hr-°F = 1.73 W/m-°K; 1 BTU/lb-°F = 4187 J/kg-°K

Table 5-59. Typical values for thermal conductivity and heat capacity of unbound materials (adapted from Sundberg, 1988).

Soil Type	Thermal Conductivity (W/m-°K)*	Heat Capacity (J/kg-°K)*
Clay with high clay content	0.85 - 1.1	1700 – 2050
Silty clay/silt	1.1 - 1.5	1650 – 1900
Silt	1.2 – 2.4	1400 – 1900
Sand, gravel below GWT	1.5 – 2.6 (1.6 – 2.0)	1450 – 1850 (1700)
Sand, gravel above GWT	0.4 – 1.1 (0.7 – 0.9)	700 – 1000 (800)
Till below GWT	1.5 – 2.5	1350 – 1700
Sandy till above GWT	0.6 – 1.8	750 – 1100
Peat below GWT	0.6	2300
Peat above GWT	0.2 – 0.5	400 - 1850

*1 W/m-°K = 0.578 BTU/ft-hr-°F; 1 J/kg-°K = 2.388E-4 BTU/lb-°F

5.6 ENVIRONMENT/CLIMATE INPUTS

5.6.1 1993 AASHTO Guide

There are only four environmental inputs in the 1993 AASHTO Guide:

- Estimated seasonal variation of the subgrade resilient modulus M_R (Section 5.4.3)
- The category for the percentage of time that the unbound pavement materials are exposed to moisture conditions near saturation (Section 5.5.1)
- The qualitative description of moisture supply for expansive subgrades (Section 5.5.1)
- The depth of frost penetration (Section 5.5.1)

These environmental factors are intertwined with their associated material property inputs and have already been described in this chapter in the sections noted above.

5.6.2 NCHRP 1-37A Design Guide

Three sets of environmental inputs are required in the NCHRP 1-37A design methodology:

- Climate, defined in terms of histories of key weather parameters
- Groundwater depth

- Surface shortwave absorptivity

These parameters are the inputs/boundary conditions for the calculation of climate-specific temperature and moisture distributions with depth and time in the EICM (see Appendix D). These distributions, in turn, are used to determine seasonal moisture contents and freeze-thaw cycles for the unbound pavement materials.

Climate Inputs

The seasonal damage and distress accumulation algorithms in the NCHRP 1-37A design methodology require hourly history data for five weather parameters:

- Air temperature
- Precipitation
- Wind speed
- Percentage sunshine (used to define cloud cover)
- Relative humidity.

The NCHRP 1-37A Design Guide recommends that the weather inputs be obtained from weather stations located near the project site. At least 24 months of actual weather station data are required for the computations. The Design Guide software includes a database of appropriate weather histories from nearly 800 weather stations throughout the United States. This database is accessed by specifying the latitude, longitude, and elevation of the project site. The Design Guide software locates the six closest weather stations to the site; the user selects a subset of these to create a virtual project weather station via interpolation of the climatic data from the selected physical weather stations.

Specification of the weather inputs is identical at all the three hierarchical input levels in the NCHRP 1-37A Design Guide.

Groundwater Depth

The groundwater table depth is intended to be the best estimate of the annual average depth. Level 1 inputs are based on soil borings, while level 3 inputs are simple estimates of the annual or seasonal average values. A potential source for level 3 groundwater depth estimates is the county soil reports produced by the National Resources Conservation Service. There is no level 2 approach for this design input.

It is important to recognize that groundwater depth can play a significant role in the overall accuracy of the foundation/pavement moisture contents and, hence, the seasonal modulus values. This is explored further in Chapter 6. Every attempt should be made to characterize groundwater depth as accurately as possible.

Surface Shortwave Absorptivity

This last environmental input is a property of the AC or PCC surface layer. The dimensionless surface short wave absorptivity defines the fraction of available solar energy that is absorbed by the pavement surface. It depends on the composition, color, and texture of the surface layer. Generally speaking, lighter and more reflective surfaces tend to have lower short wave absorptivity.

The NCHRP 1-37A recommendations for estimating surface shortwave absorptivity at each hierarchical input level are as follows:

- Level 1—Determined via laboratory testing. However, although laboratory procedures exist for measuring shortwave absorptivity, there currently are no AASHTO protocols for this for paving materials.
- Level 2—Not applicable.
- Level 3—Default values as follows:
  - Weathered asphalt (gray) 0.80 – 0.90
  - Fresh asphalt (black) 0.90 – 0.98
  - Aged PCC layer 0.70 – 0.90

Given the lack of suitable laboratory testing standards, level 3 values will typically be used for this design input.

5.7 DEVELOPMENT OF DESIGN INPUTS

Myth has it that an unknown structural engineer offered the following definition of his profession (Coduto, 2001):

> *"Structural engineering is the art and science of molding materials we do not fully understand into shapes we cannot precisely analyze to resist forces we cannot accurately predict, all in such a way that the society at large is given no reason to suspect the extent of our ignorance."*

This definition applies even more emphatically to pavement engineering. In spite of our many technical advances, there are still great gaps in our understanding. Often the greatest uncertainties in an individual project are with site conditions and materials—the types and conditions of materials encountered along the highway alignment, their spatial, temporal, and inherent variability, and their complex behavior under repeated traffic loading and environmental cycles.

Site investigation and testing programs often generate large amounts of data that can be difficult to synthesize. Real soil profiles are nearly always very complex, so borings often do not correlate and results from different tests may differ enormously. The development of a simplified representation of the soils and geotechnical conditions at a project site requires much interpolation and extrapolation of data, combined with sound engineering judgment. But what is engineering judgment? Ralph Peck suggested several alternative definitions (Dunnicliff and Deere, 1984):

> *"To the engineering student, judgment often appears to be an ingredient said to be necessary for the solution of engineering problems, but one that the student can acquire only later in his career by some undefined process of absorption from his experience and his colleagues.*
>
> *"To the engineering scientist, engineering judgment may appear to be a crutch used by practicing engineers as a poor substitute for sophisticated analytical procedures.*
>
> *"To the practicing engineer, engineering judgment may too often be an impressive name for guessing rather than for rational thinking."*

Perhaps *Webster's New Collegiate Dictionary* offers the definitive statement:

> **Judgment:** The operation of the mind, involving comparison and discrimination, by which knowledge of values and relations is mentally formulated.

But when confronted with voluminous quantities of inconsistent—and often contradictory—information, how does the pavement engineer compare and discriminate? What tools (or tricks) of the trade are available? This is a difficult process to describe. However, some common techniques for determining design values from site exploration and other geotechnical data are as follows:

- Find and remove any obvious outliers in the data. Although there are statistical techniques for doing this (*e.g.,* McCuen, 1993), in practice, detailed knowledge of the data plus engineering reasoning is usually sufficient for removing data outliers for cause. Table 5-60 summarizes some typical ranges of variability for pavement design inputs; additional information on measured variability of geotechnical parameters can be found in Baecher and Christian (2003). However, it is important that outliers (*e.g.,* a single low stiffness value) not be arbitrarily removed without fully evaluating the data for an explanation. A local anomaly may exist in the field, for example, that requires remediation.

- Examine spatial (and in some cases, temporal) trends in the data. Look at both the subsurface stratagraphic profiles and plan view "map" of subsurface conditions. Refer to the 1993 AASHTO Design Guide for resolving spatial variations in pavement design data by defining homogeneous analysis units based on a "cumulative difference" approach (Figure 5-40). A separate set of design inputs can then be developed for each homogeneous analysis unit, reducing the variability of measured vs. design input values within each unit.

- Check whether the magnitudes and trends in the data pass the test of "engineering reasonableness" – *e.g.,* are the values of the right order of magnitude? Are the trends in the data in the intuitively correct directions?

- Examine the internal consistency of the data – *e.g.,* are the phase relationships by volume consistent with the phase relationships by weight?

- Use correlations among different types of data to strengthen data interpretation – *e.g.,* statistical correlations between resilient modulus and CBR can be used to supplement a limited set of measured M_R values (although today, laboratory resilient modulus tests can often be performed more quickly and less expensively than laboratory CBR tests—see Table 5-61).

- Be clear on what is needed for a design value. The value of a material property used for specification purposes may be different from the value of that same material property when used for design. For example, a conservative value (mean plus one or two standard deviations) may be specified for the minimum compressive strength of a lime stabilized subgrade for construction quality control specifications; the mean value would be more appropriate for design applications where overall reliability (*e.g.,* factor of safety) is considered explicitly, as is the case in both the AASHTO and NCHRP design procedures.

- <u>Evaluate the sensitivity of the design to the inputs</u>! This is perhaps the most important—and often the most overlooked—aspect of design. Evaluating sensitivity to design inputs can have several benefits. First, it will categorize which inputs are most important and which are less important to the design. There is no need to expend large effort determining the precise design values for inputs that have little impact on the final outcome. More resources can then be allocated to determining the inputs that have significant impact on the outcome once they have been identified. Second, design sensitivity analyses can indicate the potential consequences of incorrect judgments of the design inputs. For example, if the subgrade resilient modulus is underestimated by 50%, will this reduce the expected useful life of the pavement by 1 year or 10 years? How does the increased cost of reduced pavement life compare with the cost of additional exploration in order to establish the subgrade resilient modulus value more robustly?

- When in doubt run more tests (a single test is often worth a thousand guesses).

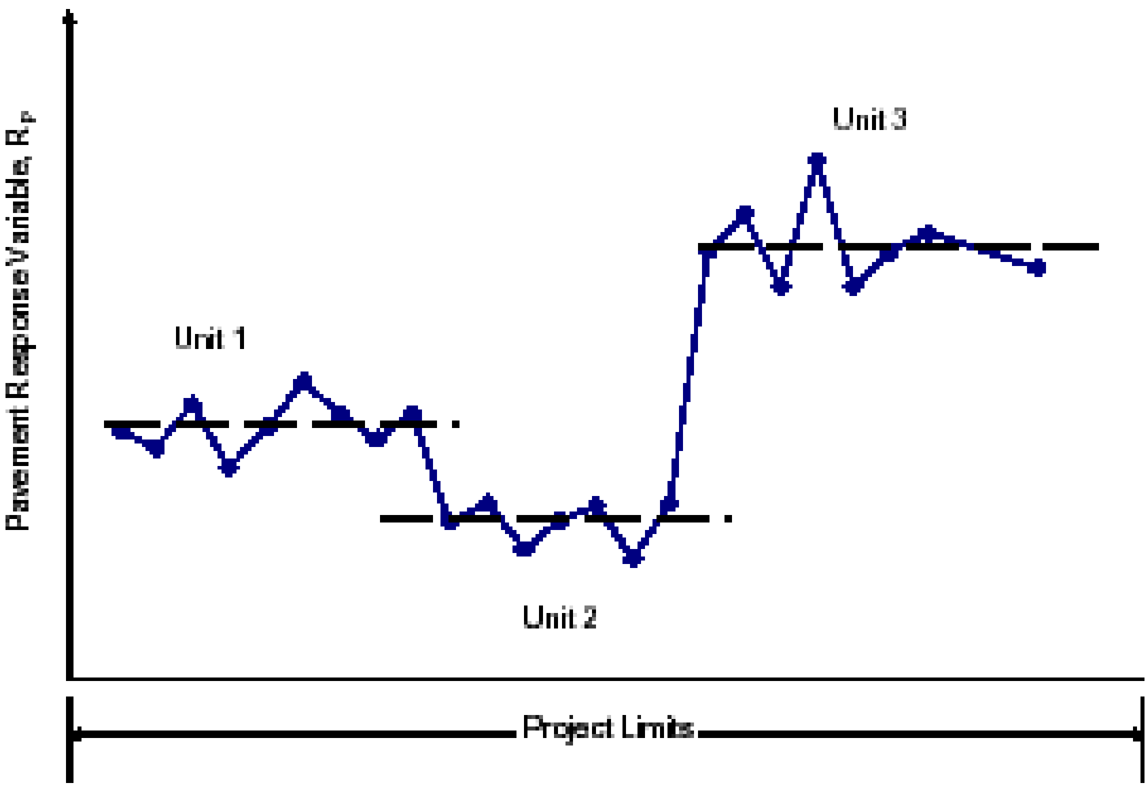


Figure 5-40. Variation of pavement response variable versus distance for given project (NCHRP 1-37A, 2004).

Table 5-60. Summary of typical pavement parameter variability (AASHTO, 1993).

Property	Standard Deviation		
	Low	Average	High
Thickness (inches)			
Portland cement concrete	0.1	0.3	0.5
Asphalt concrete	0.3	0.5	0.7
Cement treated base	0.5	0.6	0.7
Granular base	0.6	0.8	1.0
Granular subbase	1.0	1.2	1.5
Strength			
CBR (%)			
Subgrade (4 – 7)	0.5	1.0	2.0
Subgrade (7 – 13)	1.0	1.5	2.5
Subgrade (13 – 20)	2.5	4.0	6.0
Granular subbase (20 – 30)	5.0	8.0	12.0
Granular base (80+)	10.0	15.0	30.0
Percent compaction (%)			
Embankment/subgrade	2.0	4.5	7.0
Subbase/base	2.0	2.8	3.5
Portland cement concrete properties			
Air content (%)	0.6	1.0	1.5
Slump (inches)	0.6	1.0	1.4
28-day compressive strength (psi)	400	600	800
Asphalt concrete properties			
Gradation (%)			
3/4 or 1/2 inch	1.5	3.0	4.5
3/8 inch	2.5	4.0	6.0
No. 4	3.2	3.8	4.2
No. 40 or No. 50	1.3	1.5	1.7
No. 200	0.8	0.9	1.0
Asphalt content (%)	0.1	0.25	0.4
Percent compaction (%)	0.75	1.0	1.5
Marshall mix properties			
Stability (lbs)	200	300	400
Flow (in./in.)	1.0	1.3	2.0
Air voids (%)	0.8	1.0	1.4
AC consistency			
Pen @ 77°F	2	10	18
Viscosity @ 149°F (kilopoise)	2	25	100
	Coefficient of Variation (%)		
	Low	Average	High
Pavement deflection	15	30	45

Table 5-61. Resilient modulus versus CBR testing for fine grained subgrade soil (Boudreau Engineering, 2004, personal communication).

Property	CBR	Resilient Modulus
Sample size required	60 lbs (27 kg)	5 lbs (2.3 kg)
Turnaround time	10 days	4 days
Data value	Empirical	Mechanistic
In-situ testing	Field test	Shelby tube - lab
Unit price	\$365	\$300

5.8 EXERCISES

Depending upon the number of groups in the class, one or more of the following exercises may be assigned.

5.8.1 1993 AASHTO Design Guide—Flexible Pavements

Small group exercise: Given the pavement information for the Main Highway in Appendix B, estimate appropriate material property inputs for the unbound materials in a flexible pavement structure as required for the 1993 AASHTO Design Guide. (A worksheet will be distributed to guide this exercise.)

5.8.2 1993 AASHTO Design Guide—Rigid Pavements

Small group exercise: Given the pavement information for the Main Highway in Appendix B, estimate appropriate material property inputs for the unbound materials in a rigid pavement structure as required by the 1993 AASHTO Design Guide. (A worksheet will be distributed to guide this exercise.)

5.9 REFERENCES

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CHAPTER 6.0
PAVEMENT STRUCTURAL DESIGN AND PERFORMANCE

6.1 INTRODUCTION

Previous chapters have described in qualitative terms the many geotechnical factors influencing pavement design and performance, the wide range of geotechnical properties required as input to the design procedures, and the various methods for determining the values of these geotechnical inputs. Now it is time to evaluate quantitatively the importance of these factors and properties. These are the primary objectives of the present chapter:

1. To illustrate via examples how the geotechnical properties described in Chapter 5 are incorporated in the pavement design calculations; and
2. To highlight the effects of the geotechnical factors and inputs on pavement design and performance.

These objectives will be met through a series of design scenarios. First, a set of reference or baseline flexible and rigid pavement designs are developed for a hypothetical and simple project scenario. Then, the effects of various deviations from the baseline conditions will be investigated and quantified. These include

- Soft/weak subgrade conditions
- Subgrade stabilization
- Low quality base/subbase material
- Drainage and water conditions
- Shallow bedrock conditions

The design scenarios are intentionally highly idealized and simplified. Their point is to emphasize in quantitative terms how changes in geotechnical inputs affect the overall pavement design and performance. In a sense, these design scenarios are examples of the types of sensitivity studies one should perform during design to evaluate the importance of the various design inputs, especially with reference to the quality of the information used in their estimation.

All of the design scenarios described in this chapter are for new construction (or reconstruction). This is not to minimize the importance of rehabilitation design; as described in Chapter 1, most pavement design today is in fact for rehabilitation and not new construction. However, most structural rehabilitation designs focus on restoration of the surface layer, either through asphalt concrete overlays, concrete pavement restoration, or a combination of the two. For these types of scenarios, the geotechnical inputs are essentially the same as for new construction design – *e.g.,* a subgrade is a subgrade whether it is beneath a new or an existing pavement. The principal things that change are the methods by which the geotechnical inputs are determined – *e.g.,* M_R backcalculated from FWD tests instead of measured in the laboratory. The ways that these geotechnical inputs are used in the design calculations and the effects that they have on the design pavement structure are similar for new construction, rehabilitation, and reconstruction designs.

Both the current 1993 AASHTO Design Guide and the forthcoming new design guide from NCHRP Project 1-37A are applied to the scenarios in this chapter. Summaries of each of these design procedures are provided in Appendices C and D, respectively. Calculations for the 1993 AASHTO Guide designs are based on simple spreadsheet evaluation of the flexible and rigid pavement design equations. The calculations for the mechanistic-empirical designs were performed using Final Report Release version 0.700 (4/7/2004) of the NCHRP 1-37A software. This is the final version of the software as submitted to NCHRP at the conclusion of Project 1-37A.

It is important to keep in mind the significant differences between the two design procedures. The 1993 AASHTO Design Guide is an empirical methodology in which typically the design traffic, environmental conditions, and maximum serviceability (performance) loss are specified, and the corresponding required pavement structure—typically described just in terms of layer thicknesses—is determined. The NCHRP 1-37A procedure is a mechanistic-empirical methodology in which the design traffic, environmental conditions, and pavement structure are specified, and the corresponding pavement performance vs. time is predicted. In the NCHRP 1-37A procedure, several trial designs generally need to be evaluated in an iterative fashion in order to find the one (or more than one) that meets the design performance requirements.

Typically, there are multiple pavement designs that can provide the required performance for any scenario. This is true for both the 1993 AASHTO and NCHRP 1-37A design methodologies. The final selection of the "best" design should be based upon life-cycle costs, constructability, and other issues. Crude economic evaluations can be made in terms of initial construction costs, although even this is difficult because of large region-to-region variations in unit costs.

One final note regarding units of measure in this chapter: FHWA policy is to report values in SI units in all reports, with the corresponding U.S. Customary equivalent in parentheses. This is not done here. All values for the design examples in this chapter are reported in U.S. Customary units. There are two important practical reasons that dictate this choice. First, the structural layer coefficients, empirical correlations, and other data in the 1993 AASHTO Design Guide were developed and are presented in U.S. Customary units only. Although many of these could be converted to SI units, the consequences would be confusing. For example, expressing flexible pavement layer thicknesses in millimeters would require changing the values of the structural layer coefficients to quantities that would be unfamiliar to pavement engineers (*e.g.,* the a_1 value for asphalt concrete would change from 0.44 to 0.017 if asphalt thickness were expressed in millimeters). Second, although an SI version of the NCHRP 1-37A software is planned, at the time of this writing, only the U.S. Customary version is available. All of the outputs from the software are expressed in U.S. Customary units, and, consequently, the inputs described in this report are left in U.S. Customary units as well for consistency. The general SI-U.S. Customary conversion table included at the beginning of this reference manual can be used, if necessary, for converting units in this chapter.

6.2 BASELINE DESIGNS

The baseline designs for flexible and rigid pavements are intended to provide very simple and ordinary reference cases that can be used as the basis for subsequent exploration of the effects of various geotechnical inputs. The design scenario is based on the following assumptions:

- New construction
- Simple pavement structure
    - Hot mix asphalt concrete (HMA) over crushed stone graded aggregate base (GAB) over subgrade (SG) for flexible pavements
    - Jointed plain concrete pavement (JPCP) over graded aggregate base (GAB) over subgrade (SG) for rigid pavements
- Excellent, non-erodable base material (AASHTO A-1-a crushed stone)
- Nonexpansive subgrade
- Benign environmental conditions – *e.g.,* no frost heave/thaw or expansive soils
- Good drainage
- Simple traffic conditions – *e.g.,* no traffic growth over design period

Reference pavement designs consistent with these assumptions have been developed for a hypothetical new arterial highway outside of College Park, MD. The roadway is assumed to have two lanes in each direction and significant truck traffic consistenting primarily of Class 9 5-axle tractor-trailer units. The subgrade conditions are a non-expansive silty clay subgrade (AASHTO A-7-5/USCS MH material), a deep groundwater table, and no shallow bedrock. Environmental conditions in the Mid-Atlantic region are mild, so frost heave/thaw is not a design issue, and seasonal variations of the unbound material properties are expected to be minor.

Table 6-1 provides some typical in-place initial construction unit costs for paving materials in Maryland. These costs will be used for rough economic evaluations of the designs developed in this chapter.

Table 6-1. Typical in-place unit material costs for use in example design problems (MDSHA, 2002).

Material	Reasonable Range	Typical Unit Price	Typical Unit Price
Hot mix asphalt concrete (12.5 mm PG 64-22)	\$30-\$50/ton	\$36/ton	\$14,250/lane-mi-in
Portland cement concrete (PCC) without steel	\$110-\$180/cy	\$144/cy	\$28,200/lane-mi-in
Graded aggregate base	\$24-\$60/cy	\$42/cy	\$8,200/lane-mi-in

6.2.1 1993 AASHTO Design

Flexible Pavement

The baseline flexible pavement design is a three-layer system consisting of an asphalt concrete (AC) surface layer over a nonstabilized graded aggregate base (GAB) layer over subgrade (SG). The input parameters for the baseline design using the 1993 AASHTO flexible procedure for new pavements are summarized in Table 6-2. Refer to Chapter 5 for detailed explanations of all input parameters and the methods available for their determination.

The methodology by which the input parameters in Table 6-2 are used to determine the final structural design in the 1993 AASHTO Guide is described in Appendix C. The calculations are sufficiently straightforward that they can be easily performed using a spreadsheet. The key output from the 1993 AASHTO design methodology is the required pavement structure, which is determined as follows:

- Required overall structural number $SN = 4.61$
- Required structural number for asphalt concrete surface layer $SN_1 = 2.35$

- Required minimum thickness of asphalt $D_1 = \frac{SN_1}{a_1} = 5.3$ inches[1]
- Remaining structural number required for granular base layer $SN_2 = SN - D_1 a_1 = 2.28$
- Required thickness of granular base $D_2 = \frac{SN_2}{m_2 a_2} = 12.7$ inches[1]

Since the ratio of the layer coefficients $\left(a_1 / a_2 = 0.44/0.18 = 2.44\right)$ is greater than the ratio of the associated in-place unit costs per lane-mile-inch of thickness in Table 6-1 $\left(\$14,250/\$8,200 = 1.74\right)$, there is an economic benefit from substituting granular base thickness with additional asphalt in this design – *i.e.,* replacing 2.4 inches of granular base with an additional 1.0 inch of asphalt concrete is both structurally feasible (at least in terms of the 1993 AASHTO Guide) and economically beneficial (at least in terms of initial construction costs) since it would result in a savings of about $5400 per lane mile at the same *SN* value. However, in order to avoid complicating comparisons between the various design scenarios later in this chapter, the baseline flexible pavement structure will be kept at <u>5.3 inches of AC over 12.7 inches of GAB</u> (or 5.5 inches of AC over 13 inches of GAB after rounding).

Although the 15-year initial service life specified for this scenario is typical for flexible pavements and equal to the values used in the design examples in the 1993 AASHTO Guide, current trends are toward longer life or "perpetual" pavement designs. The required pavement section for a 30-year initial service life based on the 1993 AASHTO Guide is 6.0 inches of AC over 13.7 inches of GAB. The "premium" for an additional 15 years of pavement life is thus only about three-quarters of an inch of asphalt and one inch of crushed stone base.

[1]The 1993 AASHTO Design Guide recommends rounding the asphalt layer thickness to the nearest half inch and unbound layer thicknesses to the nearest inch. However, all layer thicknesses are rounded to the nearest 0.1 inch in this chapter to make the comparisons between the various scenarios more meaningful.

Table 6-2. Input parameters for 1993 AASHTO flexible pavement baseline design.

Input Parameter	Design Value	Notes
Initial service life	15 years	1
Traffic (W_{18})	6.1×10^6 ESALs	2
Reliability	90%	3
Reliability factor (Z_R)	-1.282	
Overall standard error (S_o)	0.45	1
Allowable serviceability deterioration (ΔPSI)	1.7	4
Subgrade resilient modulus (M_R)	7,500 psi	5
Granular base type	AASHTO A-1-a	1
Granular base layer coefficient (a_2)	0.18	6
Granular base drainage coefficient (m_2)	1.0	7
Asphalt concrete layer coefficient (a_1)	0.44	1

Notes:

1. Typical value for flexible pavement design.
2. Consistent with more detailed traffic input in the NCHRP 1-37A design (Section 6.2.2).
3. Typical value for a principal arterial (AASHTO, 1993).
4. Typical value for flexible pavements. No serviceability reduction for swelling or frost heave.
5. Consistent with the Level 3 default input for this soil class in the NCHRP 1-37A design after adjustment for seasonal effects (Section 6.2.2).
6. Corresponds to an M_R value of 40,000 psi, which is consistent with the Level 3 default input for this soil class in the NCHRP 1-37A design (Section 6.2.2).
7. Representative of good drainage and moderate (5-25%) saturation conditions; matches value typically used by the Maryland State Highway Administration for design.

Rigid Pavement

The baseline rigid pavement design is a three layer JPCP system consisting of a Portland cement concrete (PCC) slab over a nonstabilized graded aggregate base (GAB) layer over subgrade (SG). The input parameters for the baseline design using the 1993 AASHTO rigid procedure for new pavements are summarized in Table 6-3. Refer to Chapter 5 for detailed explanations of all input parameters and the methods available for their determination. The rigid pavement design inputs are consistent with those used for the baseline flexible pavement design.

The methodology by which the input parameters in Table 6-3 are used to determine the final structural design in the 1993 AASHTO Guide is described in Appendix C. The calculations are sufficiently straightforward that they can be easily performed using a spreadsheet. The key output from the 1993 AASHTO design methodology is the required pavement structure.

This is determined from the input parameters in Table 6-3 and the following additional intermediate steps:

- Assume a 6-inch design thickness for the granular subbase. This value is typical for rigid pavements on reasonably competent subgrades.
- Determine the composite modulus of subgrade reaction k_∞ representing the combined stiffness of the subgrade and the subgrade layer. For a 6-inch subbase thickness and unbound moduli as given in Table 6-3, $k_\infty = 423$ pci.
- Correct k_∞ for loss of support to determine the design modulus of subgrade reaction k_{eff}. (NOTE: No shallow bedrock correction is required for the assumed subgrade conditions.) For $k_\infty = 423$ pci and LS = 2, $k_{eff} = 38$ pci.
- Determine the required slab thickness D = 10.4 inches from the rigid pavement design equation.[2]

The final design selection based upon the design inputs in Table 6-3 is therefore a <u>10.4 inch PCC slab over 6 inches of GAB</u> (or 10.5 inches of PCC over 6 inches of GAB after rounding).

[2] The 1993 AASHTO Design Guide recommends rounding the slab thickness to the nearest inch (nearest half inch if controlled grade slip form pavers are used). However, all slab thicknesses are rounded to the nearest 0.1 inch in this chapter to make the comparisons between the various scenarios more meaningful.

Table 6-3. Input parameters for 1993 AASHTO rigid pavement baseline design.

Input Parameter	Design Value	Notes
Initial service life	25 years	1
Traffic (W_{18})	16.4×10^6 ESALs	2
Reliability	90%	3
Reliability factor (Z_R)	-1.282	
Overall standard error (S_o)	0.35	1
Allowable serviceability deterioration (ΔPSI)	1.9	4
Terminal serviceability level (p_t)	2.5	1
Subgrade resilient modulus (M_R)	7,500 psi	5
Granular subbase type	AASHTO A-1-a	1
Granular subbase resilient modulus (E_{SB})	40,000 psi	6
Drainage coefficient (C_d)	1.0	7
Loss of Support (LS)	2.0	8
PCC modulus of rupture (S_c')	690 psi	1
PCC modulus of elasticity (E_c)	4.4×10^6 psi	1
Joint load transfer coefficient (J)	2.8	9

Notes:

1. Typical value for rigid pavement design.
2. Consistent with more detailed traffic input in the NCHRP 1-37A design (Section 6.2.2).
3. Typical value for a principal arterial (AASHTO, 1993).
4. Typical value for rigid pavements. No serviceability reduction for swelling or frost heave.
5. Consistent with the Level 3 default input for this soil class in the NCHRP 1-37A design after adjustment for seasonal effects (Section 6.2.2).
6. Consistent with the Level 3 default input for this soil class in the NCHRP 1-37A design (Section 6.2.2).
7. Representative of good drainage and moderate (5-25%) saturation conditions.
8. Within AASHTO-recommended range for unbound granular materials.
9. Typical value for JPCP with tied PCC shoulders and dowelled joints.

6.2.2 NCHRP 1-37A Design

Flexible Pavement

Consistent with the 1993 AASHTO design, the baseline flexible pavement structure for the NCHRP 1-37A design methodology is a three-layer new construction consisting of an asphalt concrete (AC) surface layer over a nonstabilized graded aggregate base (GAB) layer over subgrade (SG). However, the input parameters required for the NCHRP 1-37A methodology are considerably more extensive than those for the 1993 AASHTO Design Guide. The NCHRP 1-37A design inputs are summarized in Table 6-4. Refer to Chapter 5 for detailed explanations of all input parameters and to Appendix D for a summary of the NCHRP 1-37A design procedure. All of the inputs correspond to Level 3 quality, and the default values provided within the NCHRP 1-37A software are used wherever appropriate.

The NCHRP 1-37A procedure requires the evaluation of several trial pavement sections in order to find the design that best meets the performance requirements. The baseline pavement structure from the 1993 AASHTO design procedure can be conveniently taken as the initial trial section. The predicted rutting performance for a trial section corresponding to the 1993 AASHTO design of 5.3 inches of AC over 12.7 inches of GAB is shown in Figure 6-1; total rutting (after adjustment for reliability) at the end of the 15-year initial service life is 0.646 inches. Fatigue and thermal cracking are negligible for this design scenario, and rutting is the controlling distress type. The design limit for predicted total rutting is an explicit input in the NCHRP 1-37A procedure that would, in general, be set by individual agency policy. For the examples in this chapter, however, the design limit for total rutting is taken as the predicted rutting for the 1993 AASHTO design section in order to make the 1993 AASHTO and NCHRP 1-37A designs equivalent for the baseline conditions. The design limit for total rutting (after adjustment for reliability and rounding) is thus 0.65 inches, which is slightly less than the 0.75 inch default value in the NCHRP 1-37A software.

Table 6-4. Input parameters for NCHRP 1-37A flexible pavement baseline design.

Input Parameter	Design Value	Notes
General Information		
Design life	15 years	1
Base/subbase construction month	September	2
Pavement construction month	September	2
Traffic open month	October	2
Site/Project Identification		
Functional class	Principal Arterials - Others	
Analysis Parameters		
Initial IRI	63 in./mi	2
Terminal IRI	172 in./mi	2
Alligator cracking limit	25%	2
Total rutting limit	0.65 in	See text
Reliability	90%	
Traffic		
Initial two-way AADTT	2000	
Number of lanes in design direction	2	
Percent of trucks in design direction	50%	2
Percent of trucks in design lane	95%	2
Operational speed	55 mph	2
Monthly adjustment	1.0 throughout	2
Vehicle class distribution	Level 3 defaults	Table 6-5
Hourly distribution	Level 3 default	Table 6-6
Traffic growth factor	0%	
Axle load distribution factors	Level 3 defaults	Table 6-7
Mean wheel location from edge	18 in	2
Traffic wander standard deviation	10 in	2
Design lane width	12 ft	2
Number of axles per truck	Level 3 defaults	Table 6-8
Average axle outside width	8.5 ft	2
Dual tire spacing	12 in	2
Tire pressure	120 psi	2
Tandem axle spacing	51.6 in	2
Tridem axle spacing	49.2 in	2
Quad axle spacing	49.2	2
Climate		
Latitude	38.98^o	
Longitude	-76.94^o	
Elevation	48 ft	
Depth of water table	20 ft	

Input Parameter	Design Value	Notes
College Park, MD climate data	Generated	3
Thermal Cracking		
Average AC tensile strength at 14^{o}F	366.5 psi	4
Creep test duration	100 sec	4
Creep compliance	Level 3 defaults	4
Mixture VMA	14.1%	5
Aggregate coefficient of thermal contraction	5x10^{-6}/^{o}F	4
Drainage and Surface Properties		
Surface shortwave absorptivity	0.85	2
Infiltration	n/a	2
Drainage path length	n/a	2
Pavement cross slope	n/a	2
AC Surface Layer		
Cumulative % retained on 3/4 inch sieve	4	5
Cumulative % retained on 3/8 inch sieve	39	5
Cumulative % retained on #4 sieve	59	5
% passing #200 sieve	3	5
Asphalt binder grade	PG 64-22	5
Reference temperature	70^{o}F	2
Effective binder content	10.1%	5
Air voids	4.0%	5
Total unit weight	151 pcf	5
Poisson's ratio	0.35	2
Thermal conductivity	0.67 BTU/hr-ft-^{o}F	2
Heat capacity	0.23 BUT/lb-^{o}F	2
Granular Base Layer		
Unbound material type	AASHTO A-1-a	
Analysis type	ICM Inputs	
Poisson's ratio	0.35	2
Coefficient of lateral pressure K_0	0.5	2
Modulus	40,000 psi	2,6
Plasticity index	1%	
% passing #200 sieve	3	
% passing #4 sieve	20	
D_{60}	8 mm	
Compaction state	Compacted	2
Maximum dry unit weight	122.2 pcf	2
Specific gravity of solids	2.66	2
Saturated hydraulic conductivity	263 ft/hr	2
Optimum gravimetric water content	11.1%	2
Calculated degree of saturation	82%	2

Input Parameter	Design Value	Notes
SWCC parameter a_f	11.1 psi	2
SWCC parameter b_f	1.83	2
SWCC parameter c_f	0.51	2
SWCC parameter h_r	361 psi	2
Compacted Subgrade (top 6 inches)		
Unbound material type	AASHTO A-7-5	
Analysis type	ICM Inputs	
Poisson's ratio	0.35	2
Coefficient of lateral pressure K_0	0.5	2
Modulus	12,000 psi	2,6
Plasticity index	30%	2
% passing #200 sieve	85	2
% passing #4 sieve	99	2
D_{60}	0.01 mm	2
Compaction state	Compacted	
Maximum dry unit weight	97.1 pcf	2
Specific gravity of solids	2.75	2
Saturated hydraulic conductivity	3.25×10^{-5} ft/hr	2
Optimum gravimetric water content	24.8%	2
Calculated degree of saturation	88.9%	2
SWCC parameter a_f	301 psi	2
SWCC parameter b_f	0.995	2
SWCC parameter c_f	0.732	2
SWCC parameter h_r	1.57×10^{4} psi	2
Natural Subgrade (beneath top 6 inches)		
Unbound material type	AASHTO A-7-5	
Compaction state	Uncompacted	
Maximum dry unit weight	87.4 pcf	2
(other properties same as for compacted subgrade)		
Distress Potential		
Block cracking	None	2
Sealed longitudinal cracks outside wheel path	None	2

Notes:

1. Typical initial service life for flexible pavement design.
2. Level 3 default/calculated/derived value from NCHRP 1-37A software.
3. Based on interpolated climate histories at IAD, DCA, and BWI airports.
4. Level 3 default/calculated/derived values from NCHRP 1-37A software for baseline AC mixture properties. Thermal cracking is not expected for the baseline design. However, these values are included here because they will be used in subsequent design scenarios.
5. Based on a Maryland State Highway Administration 19.0mm Superpave mix design.
6. Default input value at optimum moisture and density conditions before adjustment for seasonal effects (adjustment performed internally within the NCHRP 1-37A software).

Table 6-5. AADTT distribution by truck class (Level 3 defaults for Principal Arterials – Others).

Class 4	1.3%
Class 5	8.5%
Class 6	2.8%
Class 7	0.3%
Class 8	7.6%
Class 9	74.0%
Class 10	1.2%
Class 11	3.4%
Class 12	0.6%
Class 13	0.3%

Table 6-6. Hourly truck traffic distribution (Level 3 defaults for Principal Arterials – Others).

By period beginning:

Midnight	2.3%	Noon	5.9%
1:00 am	2.3%	1:00 pm	5.9%
2:00 am	2.3%	2:00 pm	5.9%
3:00 am	2.3%	3:00 pm	5.9%
4:00 am	2.3%	4:00 pm	4.6%
5:00 am	2.3%	5:00 pm	4.6%
6:00 am	5.0%	6:00 pm	4.6%
7:00 am	5.0%	7:00 pm	4.6%
8:00 am	5.0%	8:00 pm	3.1%
9:00 am	5.0%	9:00 pm	3.1%
10:00 am	5.9%	10:00 pm	3.1%
11:00 am	5.9%	11:00 pm	3.1%

Table 6-7. Truck axle load distributions: Percentage of axle loads by truck class for single axle configurations (Level 3 defaults).

	Truck Class									
Axle Weight (lbs)	4	5	6	7	8	9	10	11	12	13
3000	1.80	10.05	2.47	2.14	11.65	1.74	3.64	3.55	6.68	8.88
4000	0.96	13.21	1.78	0.55	5.37	1.37	1.24	2.91	2.29	2.67
5000	2.91	16.42	3.45	2.42	7.84	2.84	2.36	5.19	4.87	3.81
6000	3.99	10.61	3.95	2.70	6.99	3.53	3.38	5.27	5.86	5.23
7000	6.80	9.22	6.70	3.21	7.99	4.93	5.18	6.32	5.97	6.03
8000	11.47	8.27	8.45	5.81	9.63	8.43	8.35	6.98	8.86	8.10
9000	11.30	7.12	11.85	5.26	9.93	13.67	13.85	8.08	9.58	8.35
10000	10.97	5.85	13.57	7.39	8.51	17.68	17.35	9.68	9.94	10.69
11000	9.88	4.53	12.13	6.85	6.47	16.71	16.21	8.55	8.59	10.69
12000	8.54	3.46	9.48	7.42	5.19	11.57	10.27	7.29	7.11	11.11
13000	7.33	2.56	6.83	8.99	3.99	6.09	6.52	7.16	5.87	7.32
14000	5.55	1.92	5.05	8.15	3.38	3.52	3.94	5.65	6.61	3.78
15000	4.23	1.54	3.74	7.77	2.73	1.91	2.33	4.77	4.55	3.10
16000	3.11	1.19	2.66	6.84	2.19	1.55	1.57	4.35	3.63	2.58
17000	2.54	0.90	1.92	5.67	1.83	1.10	1.07	3.56	2.56	1.52
18000	1.98	0.68	1.43	4.63	1.53	0.88	0.71	3.02	2.00	1.32
19000	1.53	0.52	1.07	3.50	1.16	0.73	0.53	2.06	1.54	1.00
20000	1.19	0.40	0.82	2.64	0.97	0.53	0.32	1.63	0.98	0.83
21000	1.16	0.31	0.64	1.90	0.61	0.38	0.29	1.27	0.71	0.64
22000	0.66	0.31	0.49	1.31	0.55	0.25	0.19	0.76	0.51	0.38
23000	0.56	0.18	0.38	0.97	0.36	0.17	0.15	0.59	0.29	0.52
24000	0.37	0.14	0.26	0.67	0.26	0.13	0.17	0.41	0.27	0.22
25000	0.31	0.15	0.24	0.43	0.19	0.08	0.09	0.25	0.19	0.13
26000	0.18	0.12	0.13	1.18	0.16	0.06	0.05	0.14	0.15	0.26
27000	0.18	0.08	0.13	0.26	0.11	0.04	0.03	0.21	0.12	0.28
28000	0.14	0.05	0.08	0.17	0.08	0.03	0.02	0.07	0.08	0.12
29000	0.08	0.05	0.08	0.17	0.05	0.02	0.03	0.09	0.09	0.13
30000	0.05	0.02	0.05	0.08	0.04	0.01	0.02	0.06	0.02	0.05
31000	0.04	0.02	0.03	0.72	0.04	0.01	0.03	0.03	0.03	0.05
32000	0.04	0.02	0.03	0.06	0.12	0.01	0.01	0.04	0.01	0.08
33000	0.04	0.02	0.03	0.03	0.01	0.01	0.02	0.01	0.01	0.06
34000	0.03	0.02	0.02	0.03	0.02	0.01	0.01	0.00	0.01	0.02
35000	0.02	0.02	0.01	0.02	0.02	0.00	0.01	0.00	0.00	0.01
36000	0.02	0.02	0.01	0.02	0.01	0.01	0.00	0.00	0.00	0.01
37000	0.01	0.01	0.01	0.01	0.01	0.00	0.01	0.00	0.01	0.01
38000	0.01	0.01	0.01	0.01	0.00	0.00	0.00	0.02	0.01	0.01
39000	0.01	0.00	0.01	0.01	0.01	0.00	0.01	0.01	0.00	0.01
40000	0.01	0.00	0.01	0.01	0.00	0.00	0.04	0.02	0.00	0.00
41000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total %	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

Table 6-8. Truck axle distribution (Level 3 defaults).

Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
Class 4	1.62	0.39	0.00	0.00
Class 5	2.00	0.00	0.00	0.00
Class 6	1.02	0.99	0.00	0.00
Class 7	1.00	0.26	0.83	0.00
Class 8	2.38	0.67	0.00	0.00
Class 9	1.13	1.93	0.00	0.00
Class 10	1.19	1.09	0.89	0.00
Class 11	4.29	0.26	0.06	0.00
Class 12	3.52	1.14	0.06	0.00
Class 13	2.15	2.13	0.35	0.00

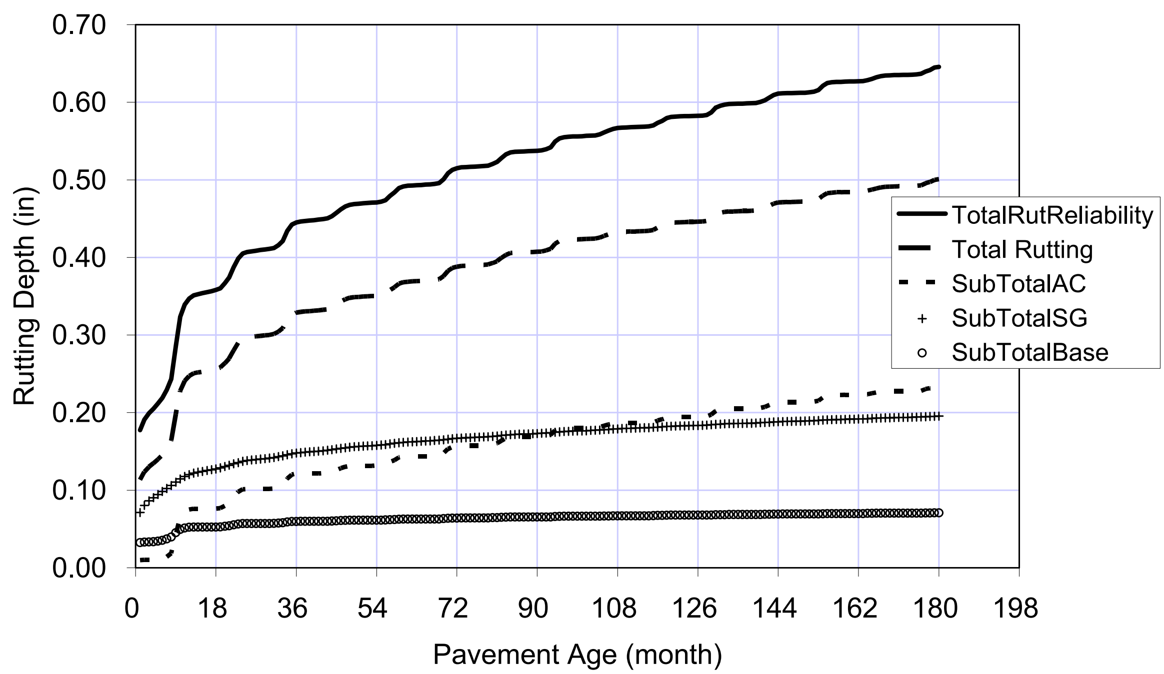


Figure 6-1. Predicted rutting performance for NCHRP 1-37A baseline flexible pavement design.

Rigid Pavement

Consistent with the 1993 AASHTO design, the baseline rigid pavement structure for the NCHRP 1-37A design methodology is a three-layer JPCP construction consisting of a Portland cement concrete (PCC) slab over a nonstabilized graded aggregate base (GAB) over subgrade (SG). However, the input parameters required for the NCHRP 1-37A methodology are considerably more extensive than for the 1993 AASHTO Design Guide. The NCHRP 1-37A design inputs are summarized in Table 6-9. Refer to Chapter 5 for detailed explanations of all input parameters and to Appendix D for a summary of the NCHRP 1-37A design methodology. All of the inputs correspond to Level 3 quality, and the default values provided within the NCHRP 1-37A software are used wherever appropriate.

The NCHRP 1-37A procedure requires the evaluation of several trial pavement sections in order to find the design that best meets the performance requirements. The baseline pavement structure from the 1993 AASHTO design procedure can be conveniently taken as the initial trial section. The predicted faulting performance for a trial section corresponding to the 1993 AASHTO Design section of 10.4 inches of PCC over 6.0 inches of GAB is shown in Figure 6-2; total faulting (after adjustment for reliability) at the end of the 25-year initial service life is 0.117 inches. Transverse fatigue cracking is negligible for this design scenario, and faulting is the controlling distress type. The design limit for predicted faulting, which is an explicit input in the NCHRP 1-37A procedure, would in general be set by individual agency policy. For the examples in this chapter, however, the design faulting limit is taken as the predicted faulting for the 1993 AASHTO design section. This is done in order to make the 1993 AASHTO and NCHRP 1-37A designs equivalent for the baseline conditions. The design limit for faulting (after adjustment for reliability and rounding) is thus 0.12 inches, which coincidentally equals the default value in the NCHRP 1-37A software. Note that initial service life for the baseline rigid pavement is 25.5 years after rounding of the faulting limit.

Table 6-9. Input parameters for NCHRP 1-37A rigid pavement baseline design.

Input Parameter	Design Value	Notes
General Information		
Initial service life	25 years	1
Pavement construction month	September	2
Traffic open month	October	2
Site/Project Identification		
Functional class	Principal Arterials - Others	
Analysis Parameters		
Initial IRI	63 in./mi	2
Terminal IRI	172 in./mi	2
Transverse cracking (% slabs cracked)	15%	2
Mean joint faulting	0.12 in	See text
Reliability	90%	
Traffic		
Initial two-way AADTT	2000	
Number of lanes in design direction	2	
Percent of trucks in design direction	50%	2
Percent of trucks in design lane	95%	2
Operational speed	55 mph	2
Monthly adjustment	1.0 throughout	2
Vehicle class distribution	Level 3 defaults	Table 6-5
Hourly distribution	Level 3 defaults	Table 6-6
Traffic growth factor	0%	
Axle load distribution factors	Level 3 defaults	Table 6-7
Mean wheel location from edge	18 in	2
Traffic wander standard deviation	10 in	2
Design lane width	12 ft	2
Number of axles per truck	Level 3 defaults	Table 6-8Table 6-8
Average axle outside width	8.5 ft	2
Dual tire spacing	12 in	2
Tire pressure	120 psi	2
Tandem axle spacing	51.6 in	2
Tridem axle spacing	49.2 in	2
Quad axle spacing	49.2	2
Wheelbase spacing	Level 3 defaults	Table 6-10
Climate		
Latitude	38.98^{o}	
Longitude	-76.94^{o}	
Elevation	48 ft	
Depth of water table	20 ft	
College Park, MD climate data	Generated	3
Design Features		
Permanent curl/warp effective	-10^{o}F	2

Input Parameter	Design Value	Notes
temperature difference		
Joint spacing	15 ft	2
Dowel bar diameter	1 in	2
Dowel bar spacing	12 in	2
Edge support	Widened slab	
Slab width	14 ft	
Bond at PCC-base interface	Unbonded	
Base erodibility index	4 (Fairly Erodable)	
Drainage and Surface Properties		
Surface shortwave absorptivity	0.85	2
Infiltration	Minor (10%)	2
Drainage path length	12 ft	2
Pavement cross slope	2%	2
PCC Surface Layer		
Unit weight	150 pcf	2
Poisson's ratio	0.2	2
Coefficient of thermal expansion	$5.5 \times 10^{-6}/^{o}F$	2
Thermal conductivity	1.25 BTU/hr-ft-^{o}F	2
Heat capacity	0.28 BTU/lb-^{o}F	2
Cement type	Type 1	2
Cement content	600 lb/yd^3	2
Water/cement ratio	0.42	2
Aggregate type	Limestone	2
PCC zero-stress temperature	120 ^{o}F	4
Ultimate shrinkage at 40% relative humidity	632 $\mu\varepsilon$	4
Reversible shrinkage	50%	2
Time to develop 50% of ultimate shrinkage	35 days	2
28-day PCC modulus of rupture	690 psi	2
28-day PCC elastic modulus	4.4×10^6 psi	4
Granular Base Layer		
Unbound material type	AASHTO A-1-a	
Analysis type	ICM Inputs	
Poisson's ratio	0.35	2
Coefficient of lateral pressure K_0	0.5	2
Modulus	40,000 psi	2,5
Plasticity index	1%	
% passing #200 sieve	3	
% passing #4 sieve	20	
D_{60}	8 mm	
Compaction state	Compacted	2
Maximum dry unit weight	122.2 pcf	2
Specific gravity of solids	2.66	2
Saturated hydraulic conductivity	263 ft/hr	2

Input Parameter	Design Value	Notes
Optimum gravimetric water content	11.1%	2
Calculated degree of saturation	82%	2
SWCC parameter a_f	11.1 psi	2
SWCC parameter b_f	1.83	2
SWCC parameter c_f	0.51	2
SWCC parameter h_r	361 psi	2
Compacted Subgrade (top 6 inches)		
Unbound material type	AASHTO A-7-5	
Analysis type	ICM Inputs	
Poisson's ratio	0.35	2
Coefficient of lateral pressure K_0	0.5	2
Modulus	12,000 psi	2,5
Plasticity index	30%	2
% passing #200 sieve	85	2
% passing #4 sieve	99	2
D_{60}	0.01 mm	2
Compaction state	Compacted	
Maximum dry unit weight	97.1 pcf	2
Specific gravity of solids	2.75	2
Saturated hydraulic conductivity	3.25×10^{-5} ft/hr	2
Optimum gravimetric water content	24.8%	2
Calculated degree of saturation	88.9%	2
SWCC parameter a_f	301 psi	2
SWCC parameter b_f	0.995	2
SWCC parameter c_f	0.732	2
SWCC parameter h_r	1.57×10^{4} psi	2
Natural Subgrade (beneath top 6 inches)		
Unbound material type	AASHTO A-7-5	
Compaction state	Uncompacted	
Maximum dry unit weight	87.4 pcf	2
(Other properties same as for compacted subgrade)		

Notes:

1. Typical initial service life for rigid pavement design.
2. Level 3 default/calculated/derived value from NCHRP 1-37A software.
3. Based on interpolated climate histories at IAD, DCA, and BWI airports.
4. Level 3 default/calculated/derived values from NCHRP 1-37A software for baseline PCC mixture properties.
5. Default input value at optimum moisture and density conditions before adjustment for seasonal effects (adjustment performed internally within the NCHRP 1-37A software).

Table 6-10. Wheelbase spacing distribution (Level 3 defaults).

	Short	Medium	Long
Average Axle Spacing (ft)	12	15	18
Percent of trucks	33%	33%	34%

Figure 6-2. Predicted faulting performance for NCHRP 1-37A baseline rigid pavement design.

6.2.3 Summary

Baseline flexible and rigid pavement designs were developed using the 1993 AASHTO Design Guide and the input parameters in Table 6-2 and Table 6-3. The final designs are 5.3 inches of AC over 12.7 inches of GAB (5.5"/13" after rounding) and 10.4 inches of PCC over 6 inches of GAB (10.5"/6" after rounding), respectively. Initial construction costs for these designs, based on the unit cost data in Table 6-1, are $180,000 and $342,000 per line-mile, respectively ($185,000 and $345,000 per lane-mile using rounded layer thicknesses).

These baseline designs were then analyzed using the NCHRP 1-37A procedures and the input parameters in Table 6-4 and Table 6-9 to determine the corresponding distress levels at the end of initial service life. Permanent deformations are the controlling distress type for the flexible pavement scenario; the predicted total rutting (after adjustment for reliability) for the baseline flexible pavement section is 0.65 inches, as compared to the 0.75-inch default value in the NCHRP 1-37A software. Joint faulting is the controlling distress type for the rigid pavement scenario; the predicted joint faulting (after adjustment for reliability) for the baseline rigid pavement design is 0.12 inches, identical to the default value in the NCHRP 1-37A software. Note that the baseline design scenarios are not greatly different from the pavement conditions at the AASHO Road Test (except perhaps for climate), and therefore the 1993 AASHTO designs should be in general agreement with those from the more sophisticated NCHRP 1-37A methodology. Discrepancies between the design procedures should become more pronounced as conditions increasingly deviate from the AASHO Road Test conditions.

6.3 SOFT SUBGRADE

The design scenario for this case is identical to the baseline conditions in Section 6.2, except for a much softer and weaker subgrade. The subgrade is now postulated to be a very soft high plasticity clay (AASHTO A-7-6, USCS CH) with M_R = 6000 psi at optimum moisture and density before adjustment for seasonal effects. Note that this M_R value is even lower than the NCHRP 1-37A default values for an A-7-6/CH material in order to accentuate the effects of low subgrade stiffness. The groundwater depth is left unchanged from the baseline scenario in order to focus on the subgrade stiffness effect. Intuitively, more substantial pavement sections are expected for this scenario as compared to the baseline conditions to achieve the same level of pavement performance. This can be achieved by increasing the thicknesses of the AC/PCC/granular base layers, increasing the quality of the AC/PCC/granular base materials, stabilizing the granular base layer, treating the soft subgrade soil, or some combination of these design modifications. In order to keep the comparisons among scenarios simple, only increases in AC or PCC thickness will be considered here.

6.3.1 1993 AASHTO Design

Flexible Pavement

The only modification to the 1993 AASHTO flexible pavement baseline inputs (Table 6-2) required to simulate the soft subgrade condition is a reduction of the seasonally-adjusted subgrade resilient modulus M_R from 7,500 to 3,800 psi.[3] The W_{18} traffic capacity for baseline flexible pavement section under the soft subgrade conditions is only 1.25×10^6 ESALs, corresponding to an 80% decrease in initial service life.

The required pavement structure for the soft subgrade condition is determined from the 1993 AASHTO design procedure, as follows (assuming changes only in the AC layer thickness):

- Required overall structural number SN = 5.76 (compared to 4.61 for the baseline conditions)
- Structural number provided by granular base (same thickness D_2 as in baseline design) $SN_2 = m_2 a_2 D_2 = (1.0)(0.18)(12.7) = 2.28$
- Required asphalt structural number $SN_1 = SN - SN_2 = 5.76 - 2.28 = 3.48$
- Required asphalt layer thickness $D_1 = \dfrac{SN_1}{a_1} = 7.9$ inches

The design for the soft subgrade condition is thus <u>7.9 inches of AC over 12.7 inches of GAB</u> (before rounding). This represents a 50% increase in AC thickness as compared to the 5.3 inches in the baseline design, which, in turn, translates to a 20% initial construction cost increase of about \$37K per lane-mile (using the typical unit cost data in Table 6-1).

Rigid Pavement

The only modification to the 1993 AASHTO rigid pavement baseline inputs (Table 6-3) required to simulate the soft subgrade condition is a reduction of the seasonally-adjusted subgrade resilient modulus M_R from 7,500 to 3,800 psi. The W_{18} traffic capacity for the baseline rigid pavement section under the soft subgrade conditions is reduced to 15.5×10^6 ESALs, corresponding to a 6% decrease in initial service life.

The reduction in foundation stiffness in this scenario has a direct effect on the design modulus of subgrade reaction k_{eff}, which decreases from its original value of 38 pci for baseline conditions to a value of 27 pci for the soft subgrade case. However, the required slab thickness is relatively insensitive to this reduction in foundation stiffness, increasing only 0.1 inches for a final design of <u>10.5 inches of PCC over 6 inches of GAB</u> for the soft subgrade

[3] Based on the results from the NCHRP 1-37A analyses for these conditions.

condition. Note that after rounding to the nearest half-inch, this design is identical to the rigid pavement design for the baseline conditions.

A common constructability concern under these soft *in-situ* soil conditions is the requirement for a stable working platform. The 6-inch granular subbase is unlikely to provide adequate stability. Consequently, a realistic final design would require either a thicker granular subbase, a separate granular working platform (not included in the structural design calculations), and/or subgrade improvement (see Chapter 7) for constructability.

6.3.2 NCHRP 1-37A Design

Flexible Pavement

Changing the subgrade soil type to A-7-6 changes many of the other Level 3 default inputs for the subgrade in the NCHRP 1-37A design methodology. The altered input parameters for the soft subgrade condition are summarized in Table 6-11. Figure 6-3 summarizes the predicted rutting vs. time for the baseline flexible pavement section (5.3″ AC over 12.7″ granular base); the time to the 0.65 inch total rutting design limit is only 93 months (7.75 years), corresponding to a 48% decrease in initial service life due to the soft subgrade conditions.

The trial designs (assuming only increases in AC thickness) and their corresponding predicted performance at end of the initial service life are listed in Table 6-12. Rutting is again the critical distress mode controlling the design in all cases; the design limit of 0.65 inches for total rutting is based on the performance of the baseline pavement section, as described previously in Section 6.2.2. Interpolating among the results in Table 6-12, the final flexible pavement design section for the soft subgrade conditions consists of 7.9 inches of AC over 12.7 inches of GAB. This design section is identical to that obtained from the 1993 AASHTO Design Guide for this scenario.

Table 6-11. Modified input parameters for NCHRP 1-37A flexible pavement design: soft subgrade scenario.

Input Parameter	Design Value	Notes
Compacted Subgrade (top 6 inches)		
Unbound material type	AASHTO A-7-6	
Analysis type	ICM Inputs	
Poisson's ratio	0.35	1
Coefficient of lateral pressure K_0	0.5	1
Modulus	6,000 psi	2,3
Plasticity index	40%	1
% passing #200 sieve	90	1
% passing #4 sieve	99	1
D_{60}	0.01 mm	1
Compaction state	Compacted	
Maximum dry unit weight	91.3	1
Specific gravity of solids	2.77	1
Saturated hydraulic conductivity	$3.25\text{x}10^{-5}$ ft/hr	1
Optimum gravimetric water content	28.8%	1
Calculated degree of saturation	89.4%	1
SWCC parameter a_f	750 psi	1
SWCC parameter b_f	0.911	1
SWCC parameter c_f	0.772	1
SWCC parameter h_r	$4.75\text{x}10^{4}$ psi	1
Natural Subgrade (beneath top 6 inches)		
Unbound material type	AASHTO A-7-6	
Compaction state	Uncompacted	
Maximum dry unit weight	82.2 pcf	2
(Other properties same as for compacted subgrade)		

Notes:

1. Level 3 default/calculated/derived value from NCHRP 1-37A software.
2. Set artificially low to simulate a soft subgrade condition.
3. Input value at optimum moisture and density conditions before adjustment for seasonal effects (adjustment performed internally within the NCHRP 1-37A software).

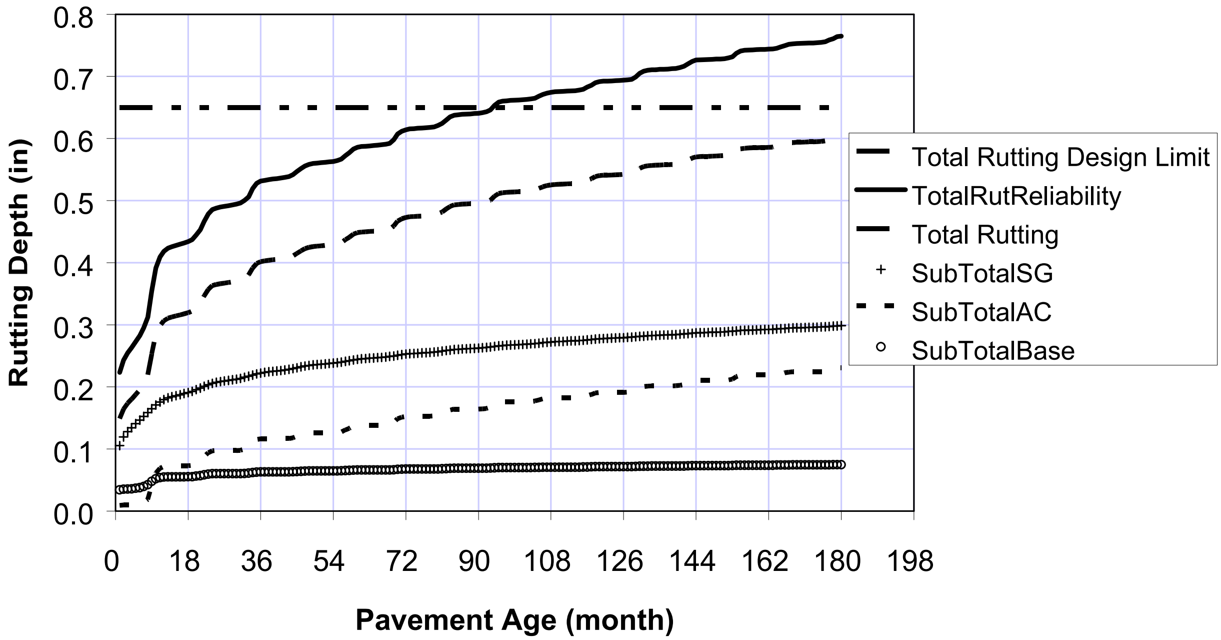


Figure 6-3. Predicted rutting performance for soft subgrade scenario.

Table 6-12. Trial cross sections for NCHRP 1-37A flexible pavement design: soft subgrade scenario.

AC Thickness (in.)	Base Thickness (in.)	Total Rutting (in.)
5.3	12.7	0.765
6.0	12.7	0.730
8.0	12.7	0.643
10.0	12.7	0.572
	Design Limit:	0.65

Rigid Pavement

Changing the subgrade soil type to A-7-6 changes many of the other Level 3 default inputs for the subgrade in the NCHRP 1-37A design methodology. The altered input parameters for the rigid pavement soft subgrade condition are the same as those summarized earlier in Table 6-11 for the corresponding flexible design condition. Figure 6-4 summarizes the predicted faulting vs. time for the baseline rigid pavement section (10.4″ PCC over 6.0″ granular base); the time to the 0.12 inch faulting design limit is only 22.2 years, corresponding to a 13% decrease in initial service life due to the soft subgrade conditions.

The trial designs (assuming only increases in PCC slab thickness) and their corresponding predicted performance at end of design life are listed in Table 6-13. Faulting is again the critical distress mode controlling the design in all cases; the design limit of 0.12 inches for faulting is based on the performance of the baseline pavement section, as described previously in Section 6.2.2. Interpolating among the results in Table 6-13, the final rigid pavement design section for the soft subgrade conditions consists of a 10.9 inch PCC slab over 6.0 inches of GAB. This slab thickness is 0.5 inch (5%) greater than that obtained from the 1993 AASHTO Design Guide for this scenario; this corresponds to an initial construction cost increase of $14K (8%) per lane-mile.

Again, the subgrade soil is so soft and weak in this scenario that some additional design features may be required to provide a stable working platform during construction. The 6 inch granular subbase is unlikely by itself to provide an adequate working platform.

Figure 6-4. Predicted faulting performance for soft subgrade scenario.

Table 6-13. Trial cross sections for NCHRP 1-37A rigid pavement design: soft subgrade scenario.

PCC Thickness (in.)	Base Thickness (in.)	Faulting (in.)
10.4	6	0.131
10.7	6	0.125
11.0	6	0.118
	Design Limit:	0.12

6.3.3 Summary

The design flexible pavement sections for the baseline and soft subgrade scenarios are summarized in Figure 6-5. As expected, the soft subgrade condition mandates a thicker pavement cross section. For the both the 1993 AASHTO and NCHRP 1-37A designs, the required AC thickness increases from 5.3 – 7.9 inches (before rounding and based on the simplest assumption of constant GAB thickness). The GAB thickness remains a constant 12.7 inches for all designs, although as described previously, the granular base thickness would probably be increased for constructability purposes in order to provide a stable working platform over the soft subgrade.

As mentioned previously, increased asphalt thickness could economically be used with a thinner aggregate base layer for the same structural capacity in the 1993 AASHTO procedure, but this adjustment has not been made here. The NCHRP 1-37A methodology does not allocate the increased overall section thickness to the individual layers in the same way as the 1993 AASHTO procedure, and in practice one should examine multiple thickness combinations to find the minimum cost design that meets the specified performance limits.

Initial construction cost ranges (based on Table 6-1) for the flexible pavement designs are summarized in Figure 6-6. The average initial construction costs increase from about $180K to $217K (~20%) per lane-mile due to the soft subgrade for both the 1993 AASHTO and NCHRP 1-37A designs.

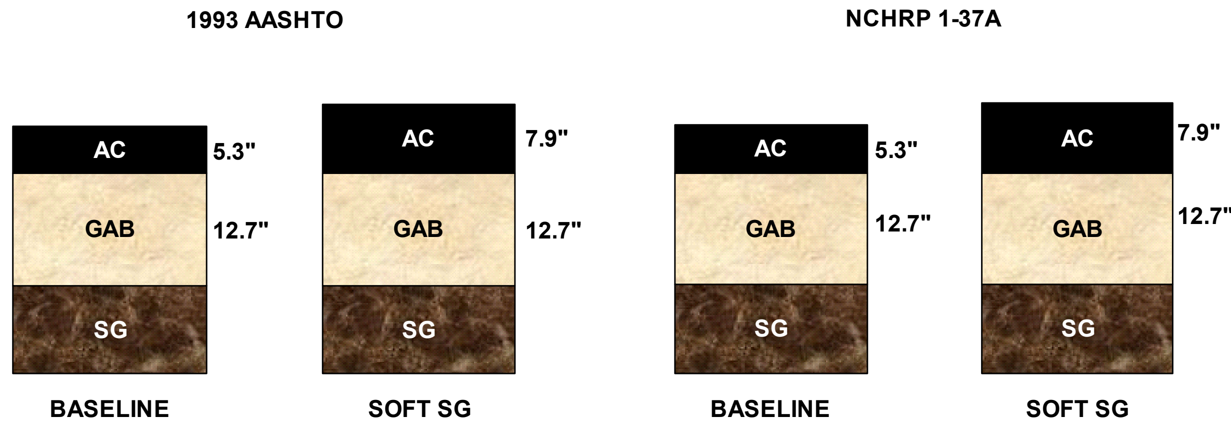


Figure 6-5. Summary of flexible pavement sections: soft subgrade scenario.

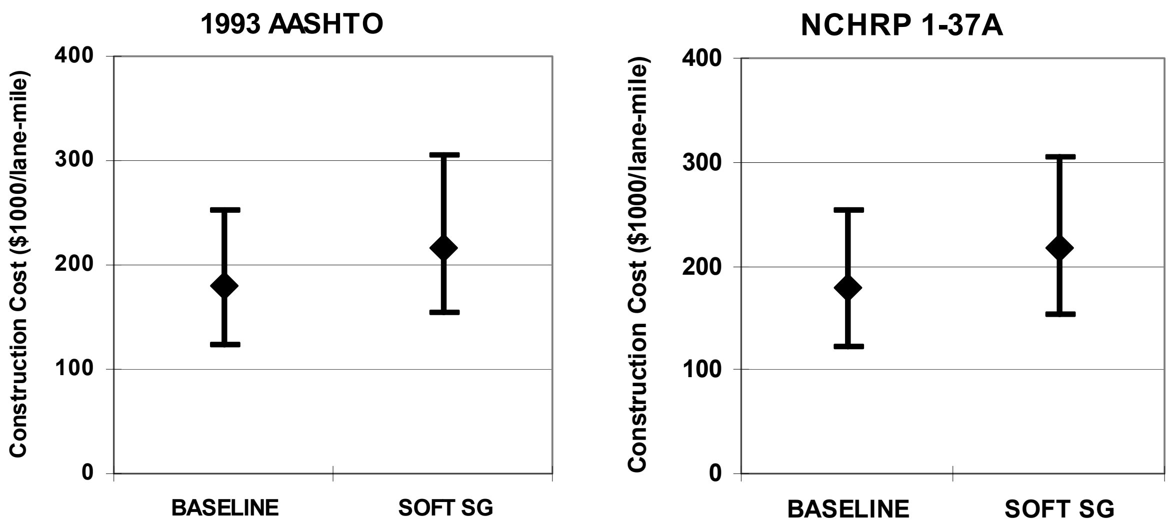


Figure 6-6. Example construction costs for flexible pavement sections: soft subgrade scenario.

The design rigid pavement sections for the baseline and soft subgrade scenarios are summarized in Figure 6-7. The increase in slab thickness required for the soft subgrade conditions is slight, from 10.4 to 10.5 inches for the 1993 AASHTO designs and from 10.4 to 10.9 inches for the NCHRP 1-37A sections (before rounding). The GAB thickness remains a constant 6 inches for all designs, although as described previously, this would probably be increased for constructability purposes.

Initial construction cost ranges (based on Table 6-1) for the rigid pavement designs are summarized in Figure 6-8. The average increase in initial construction cost due to the soft subgrade are quite small, ranging from about $2K (~1%) for the 1993 AASHTO design to about $14K (~8%) for the NCHRP 1-37A sections. Note that these initial construction costs for rigid pavements (Figure 6-8) cannot be fairly compared to the initial construction costs for flexible pavements (Figure 6-6) because of the different assumptions for the initial service life and the different maintenance and repair costs that will be required over their design lives. A fair comparison would require evaluation of life-cycle costs, including maintenance, repair, rehabilitation, and perhaps user costs, in addition to the initial construction expense.

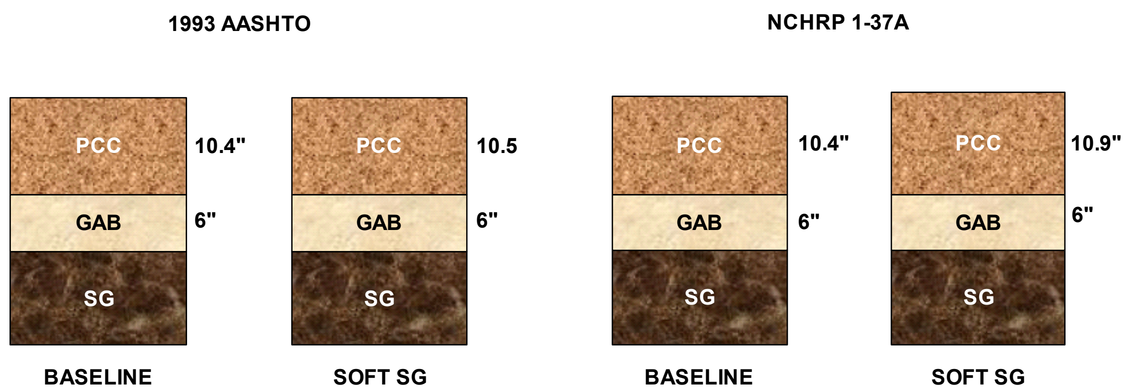


Figure 6-7. Summary of rigid pavement sections: soft subgrade scenario.

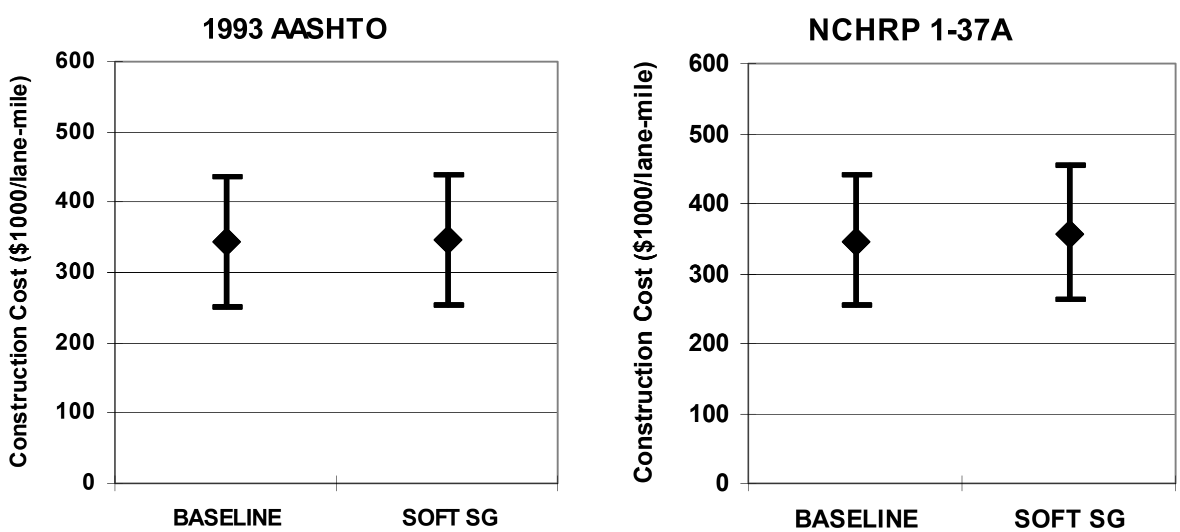


Figure 6-8. Example construction costs for rigid pavement sections: soft subgrade scenario.

Given the significant increases in flexible pavement section required to deal with the weak subgrade condition, other design approaches should be considered, such as subgrade stabilization, geosynthetic reinforcement of the base layer, or switching to a rigid pavement system that is more tolerant of poor foundation conditions. The benefits of subgrade stabilization for the flexible pavement designs are explored in the next section.

6.4 SUBGRADE STABILIZATION

As described in the preceding section, a very soft and weak subgrade requires a substantially thicker section for flexible pavements. The effect on required slab thickness for rigid pavements is slight; this confirms the conventional wisdom that rigid pavements are particularly advantageous for very poor subgrade support conditions.

Lime stabilization is a common technique for improving soft and weak subgrades beneath flexible pavements. A primary benefit of lime stabilization is a greatly increased stiffness within the stabilized zone as a function of the lime content. Consequently, the lime content—or more specifically, the effect of lime content on subgrade properties—and the thickness of the stabilized zone are the primary variables for the stabilization design.

The effect of lime content on the engineering properties of stabilized subgrades will depend greatly on the specific subgrade being stabilized. As a simple illustration of the benefits of lime stabilization, the sensitivity of predicted rutting to the thickness and resilient modulus of the stabilized zone can be examined using the NCHRP 1-37A methodology.[4] For the purpose of this illustration, all design inputs are kept the same as for the soft subgrade scenario (Table 6-4 and Table 6-11) except that the thickness and resilient modulus of the compacted upper layer of the subgrade are adjusted to simulate the lime stabilized zone.

Total rutting (after adjustment for reliability) at the end of the 15-year initial service life as predicted by the NCHRP 1-37A design methodology for various thicknesses and stiffnesses of the lime stabilized zone are summarized in Figure 6-9. Based on the data in Chapter 7, each 1% of lime in the stabilized zone corresponds <u>very</u> roughly to an increase in resilient modulus M_R of about 10,000 psi. The data in Figure 6-9 suggest that a lime content corresponding to an M_R value of 60,000 psi over a depth of 18 inches is one of several combinations that will meet the 0.65 inch design limit for total rutting. A pavement section consisting of <u>5.3 inches of AC over 12.7 inches of GAB over 18 inches of lime-stabilized subgrade (M_R=60,000 psi)</u> should provide sufficient performance. This will also be the more

[4] Note: The NCHRP 1-37A design methodology has not been calibrated for lime stabilization because of an insufficient numbers of appropriate field sections in the LTPP database.

economically feasible design if the cost of the lime stabilization is less than the $37K/lane-mile cost of the 2.6 inches of asphalt concrete saved as a consequence of the subgrade improvement.

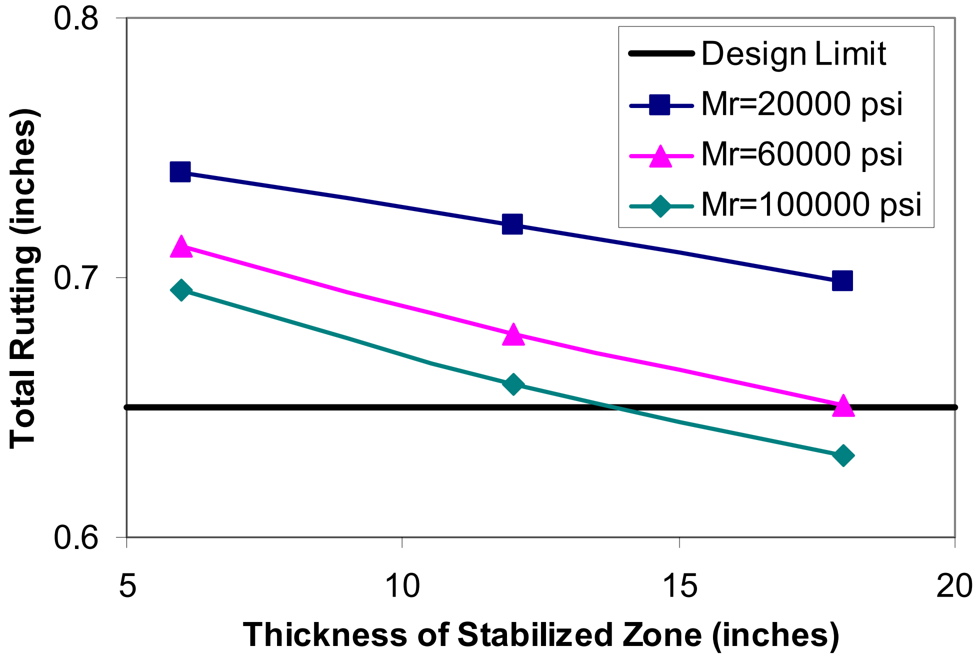


Figure 6-9. Effect of lime stabilization on predicted total rutting.

6.5 LOW QUALITY BASE/SUBBASE

The design scenario for this case is identical to the baseline conditions in Section 6.2, except that a lower quality granular base material is specified. The granular base is now postulated to be a clayey sand gravel (AASHTO A-2-6, USCS GC or SC) with M_R = 26,000 psi at optimum moisture and density. Intuitively, significantly thicker design pavement sections are expected for this scenario, as compared to the baseline conditions to achieve the same level of pavement performance.

6.5.1 1993 AASHTO Design

Flexible Pavement

Three modifications to the 1993 AASHTO flexible pavement baseline inputs (Table 6-2) are made to simulate the low quality base condition:

1. The seasonally-adjusted resilient modulus M_R for the granular base layer is reduced from 40,000 to 26,000 psi.

2. The structural layer coefficient a_2 is reduced from 0.18 to 0.12, consistent with the reduction in M_R for the granular base (see correlations in Section 5.4.5).
3. The drainage coefficient m_2 is reduced from 1.0 to 0.70 to reflect a reduction in drainage quality from good to poor, due to the substantially increased fines content.
4. This yields an SN value of only 3.4 for the baseline design layer thicknesses. The W_{18} traffic capacity for this SN value is only 0.88x10^6 ESALs, corresponding to an 86% decrease in initial service life.

The required structure is then determined from the design equations, as follows (again assuming changes only in the AC layer thickness):

- Required overall structural number $SN = 4.6$ (unchanged from baseline conditions)
- Structural number provided by granular base (same thickness D_2 as in baseline design) $SN_2 = m_2 a_2 D_2 = (0.7)(0.12)(12.7) = 1.07$
- Required asphalt structural number $SN_1 = SN - SN_2 = 4.61 - 1.07 = 3.54$
- Required asphalt layer thickness $D_1 = \dfrac{SN_1}{a_1} = 8.0$ inches

The design for the low quality base condition is thus <u>8.0 inches of AC over 12.7 inches of GAB</u> (before rounding). This represents a significant increase in AC thickness as compared to the baseline AC design thickness of 5.3 inches and is even slightly thicker than the 7.9 inches of AC required for the soft subgrade scenario (Section 6.3.1). This translates to a 21% initial construction cost increase of about $38K per lane-mile (using the typical unit cost data in Table 6-1 and assuming that the unit cost for the low quality base is the same as for high quality crushed stone).

Rigid Pavement

Three modifications to the 1993 AASHTO rigid pavement baseline inputs (Table 6-3) are made to simulate the low quality subbase condition:

1. The resilient modulus M_R for the granular subbase is reduced from 40,000 to 26,000 psi.
2. The Loss of Support LS coefficient is increased from 2 to 3 to reflect the increased erosion potential of the fines in the granular subbase.
3. The drainage coefficient C_d is reduced from 1.0 to 0.85 to reflect a reduction in drainage quality from good to poor, due to the substantially increased fines content.

The W_{18} traffic capacity for the baseline rigid pavement section under the low quality subbase condition is reduced to 8.3x10^6 ESALs, corresponding to a 50% decrease in initial service life.

The first two changes listed above will have a direct effect on the design modulus of subgrade reaction k_{eff}, which reduces from its baseline value of 38 pci to a value of 17 pci for the low quality subbase condition. The 1993 AASHTO design corresponding to this k_{eff} and C_d of 0.85 consists of an 11.5 inch PCC slab over 6 inches of GAB. Note that this design is substantially thicker than the 10.4" PCC over 6" subbase section for the 1993 AASHTO baseline scenario; this corresponds to an initial construction cost increase of $31K (9%) per lane-mile. The increase in slab thickness is due primarily to the reduction in the drainage coefficient C_d; only about 0.2 inches of the slab thickness increase is attributable to the increased erodibility of the subbase, as reflected in the larger *LS* value and the consequently reduced value for k_{eff}.

6.5.2 NCHRP 1-37A Design

Flexible Pavement

Changing the granular base soil type from A-1-a to A-2-6 changes many of the other Level 3 default inputs for this layer in the NCHRP 1-37A design methodology. The altered input parameters for the low quality base condition are summarized in Table 6-14. Figure 6-10 summarizes the predicted rutting vs. time for the baseline flexible pavement section (5.3″ AC over 12.7″ GAB); the time to the 0.65 inch total rutting design limit is only 153 months (12.75 years), corresponding to a 15% decrease in initial service life due to the low quality base. Although cracking is not the controlling distress for this scenario, reducing the quality of the base material does have a significant effect on cracking, more so than on rutting: predicted alligator cracking increases by a factor of 3 and longitudinal cracking by a factor of over 10 in the low quality base scenario, as compared to baseline conditions.

The trial designs and their corresponding predicted performance at end of the initial service life are listed in Table 6-15. Rutting is again the critical distress mode controlling the design in all cases; the design limit of 0.65 inches for total rutting is based on the performance of the baseline pavement section, as described previously in Section 6.2.2. Interpolating among the results in Table 6-15, the final flexible pavement design section for the low quality base condition consists of 5.8 inches of AC over 12.7 inches of GAB. This is significantly thinner than the 8 inches of asphalt required by the 1993 AASHTO design for this scenario, but only 0.5 inches (9%) thicker than in the baseline design, corresponding to a $7.1K/lane-mile (4%) increase in initial construction costs.

Table 6-14. Modified input parameters for NCHRP 1-37A flexible pavement design: low quality base scenario.

Input Parameter	Design Value	Notes
Granular Base Layer		
Unbound material type	AASHTO A-2-6	
Analysis type	ICM Inputs	
Poisson's ratio	0.35	1
Coefficient of lateral pressure K_0	0.5	1
Modulus	26,000 psi	2
Plasticity index	15%	1
% passing #200 sieve	20	1
% passing #4 sieve	95	1
D_{60}	0.1 mm	1
Compaction state	Compacted	
Maximum dry unit weight	117.5 pcf	1
Specific gravity of solids	2.71	1
Saturated hydraulic conductivity	1.73×10^{-5} ft/hr	1
Optimum gravimetric water content	13.9%	1
Calculated degree of saturation	85.9%	1
SWCC parameter a_f	23.1 psi	1
SWCC parameter b_f	1.35	1
SWCC parameter c_f	0.586	1
SWCC parameter h_r	794 psi	1

Notes:

1. Level 3 default/calculated/derived value from NCHRP 1-37A software.
2. Level 3 default value for this soil class. Input value is at optimum moisture and density conditions before adjustment for seasonal effects (adjustment performed internally within the NCHRP 1-37A software).

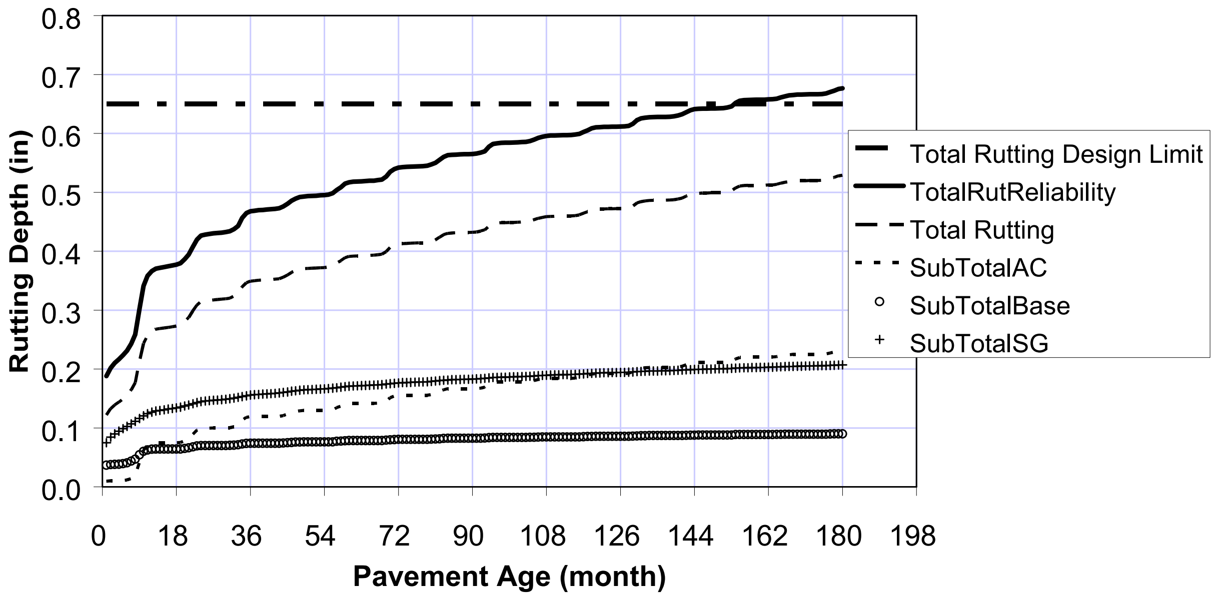


Figure 6-10. Predicted rutting performance for low quality base scenario.

Table 6-15. Trial cross sections for NCHRP 1-37A flexible pavement design: low quality base scenario.

AC Thickness (in.)	Base Thickness (in.)	Total Rutting (in.)
5.3	12.7	0.677
5.5	12.7	0.664
6.0	12.7	0.635
	Design Limit:	0.65

Rigid Pavement

Changing the granular base soil type from A-1-a to A-2-6 changes many of the other Level 3 default inputs for this layer in the NCHRP 1-37A design methodology. The altered input parameters for the base layer in the rigid pavement design are the same as those summarized earlier in Table 6-14 for the corresponding flexible design condition. In addition, the Erodibility Index for the base layer is changed from "Fairly Erodible (4)" to "Very Erodible (5)" (see Table 6-16) to reflect the increased fines content of the A-2-6 base material. In reality, the erodibility is probably somewhere in between these categories, but the NCHRP 1-37A does not permit input of intermediate erodibility conditions.

Figure 6-11 summarizes the predicted faulting vs. time for the baseline rigid pavement section (10.4″ PCC over 6.0″ GAB); the time to the 0.12 inch faulting design limit is only 24 years, corresponding to a 6% decrease in initial service life due to the low quality base condition.

The trial designs and their corresponding predicted performance at end of design life are listed in Table 6-17. Also shown in the table is the design limit for predicted faulting, which is the controlling distress for this scenario. A pavement section consisting of a 10.7 inch PCC slab over 6 inches of GAB (before rounding) meets the design faulting limit for the low quality base scenario. This pavement section is 0.3 inches thicker than the baseline design, but significantly thinner than the 11.5″ slab from the 1993 AASHTO design for this scenario. Increased initial construction cost for the additional 0.3 inches of PCC slab is \$8.5K (2.5%) per lane-mile.

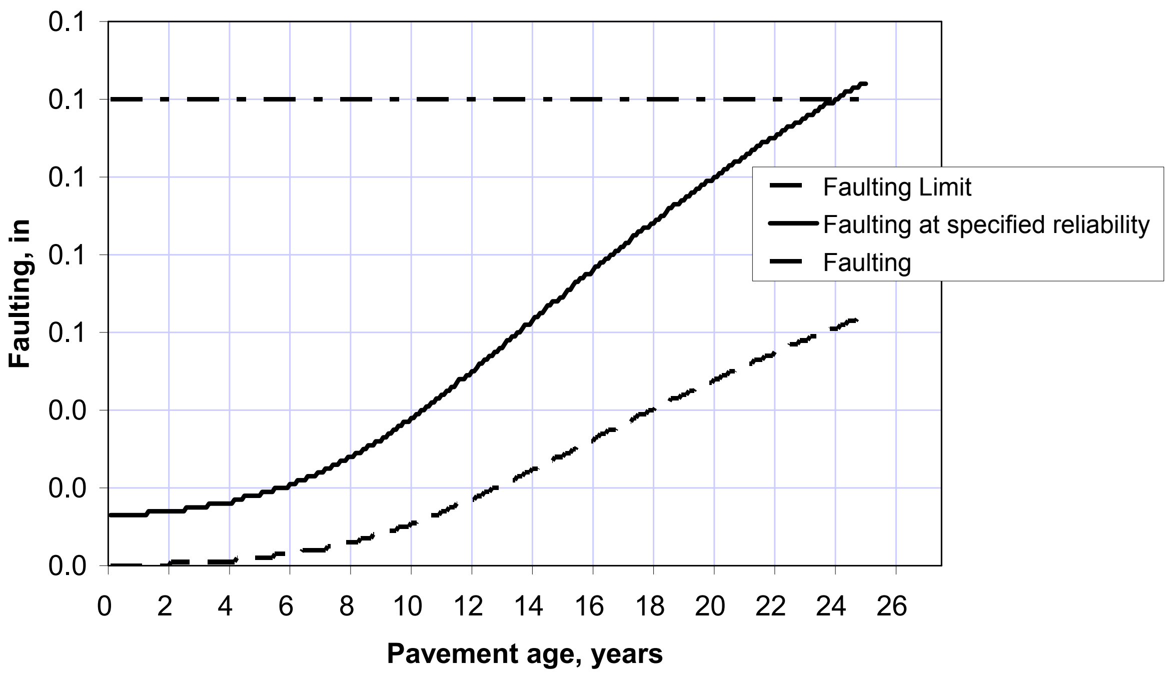


Figure 6-11. Predicted faulting performance for low quality base scenario.

Table 6-16. NCHRP 1-37A recommendations for assessing erosion potential of base material (adapted after PIARC, 1987; Christory, 1990).

Erodibility Class	Material Description and Testing
1	(1) Lean concrete with previous outstanding past performance and a granular subbase layer or a stabilized soil layer or a geotextile fabric layer is placed between the treated base and subgrade, otherwise class 2. (2) Hot mixed asphalt concrete with previous outstanding past performance and a granular subbase layer or a stabilized soil layer is placed between the treated base and subgrade, otherwise class 2. (3) Permeable drainage layer (asphalt or cement treated aggregate) and a granular or a geotextile separation layer between the treated permeable base and subgrade. (4) Unbonded PCC Overlays: HMAC separation layer (either dense or permeable graded) is specified.
2	(1) Cement treated granular material with good past performance and a granular subbase layer or a stabilized soil or a geotextile fabric layer is placed between the treated base and subgrade, otherwise class 3. (2) Asphalt treated granular material with good past performance and a granular subbase layer or a stabilized soil layer or a geotextile soil layer is placed between the treated base and subgrade, otherwise class 3.
3	(1) Cement-treated granular material that has exhibited some erosion and pumping in the past. (2) Asphalt treated granular material that has exhibited some erosion and pumping in the past. (3) Unbonded PCC Overlays: Surface treatment or sand asphalt is used.
4	Unbound crushed granular material having dense gradation and high quality aggregates.
5	Untreated subgrade soils (compacted).

Table 6-17. Trial cross sections for NCHRP 1-37A rigid pavement design: low quality base scenario.

PCC Thickness (in.)	Base Thickness (in.)	Faulting (in.)
10.4	6	0.124
10.6	6	0.121
10.7	6	0.119
11.0	6	0.113
	Design Limit:	0.12

6.5.3 Summary

The design flexible pavement sections for the baseline and low quality base scenarios are summarized in Figure 6-12. As expected, the low quality base condition mandates a thicker pavement cross section in both design methodologies. For the 1993 AASHTO design, the required asphalt thickness increases from 5.3 to 8.0 inches (assuming constant granular base thickness); the NCHRP 1-37A design requires a significantly smaller increase of only an additional half-inch of asphalt thickness. Overall, these results suggest that the 1993 AASHTO Guide assigns more weight—at least with regard to rutting, the controlling distress in these scenarios—to the structural contributions from the unbound layers than does the NCHRP 1-37A methodology; this trend has been observed in other comparison studies between the 1993 AASHTO Guide and the new NCHRP 1-37A procedure.

Initial construction cost ranges (based on Table 6-1) for the flexible pavement designs are summarized in Figure 6-12. Note that these costs are very approximate; in particular, the same unit cost has been used for both the high quality (baseline) and low quality granular base materials, but in reality these would likely be different. The initial construction costs for the low quality base scenario (based on the typical unit costs in Table 6-1) increase by about 38K or 21% per lane-mile in the 1993 AASHTO designs and only by about $7.1K or 4% in the NCHRP 1-37A designs.

The design rigid pavement sections for the baseline and low quality base scenarios are summarized in Figure 6-14. The low quality base condition necessitates a significantly thicker pavement cross section in the 1993 AASHTO design, with the slab thickness increasing by 1.1 inches. The required slab thickness increased a much smaller 0.3 inches in the NCHRP 1-37A design. As for flexible pavements, the 1993 AASHTO Guide appears to attach more weight to the structural contributions of the unbound layers than does the NCHRP 1-37A procedure.

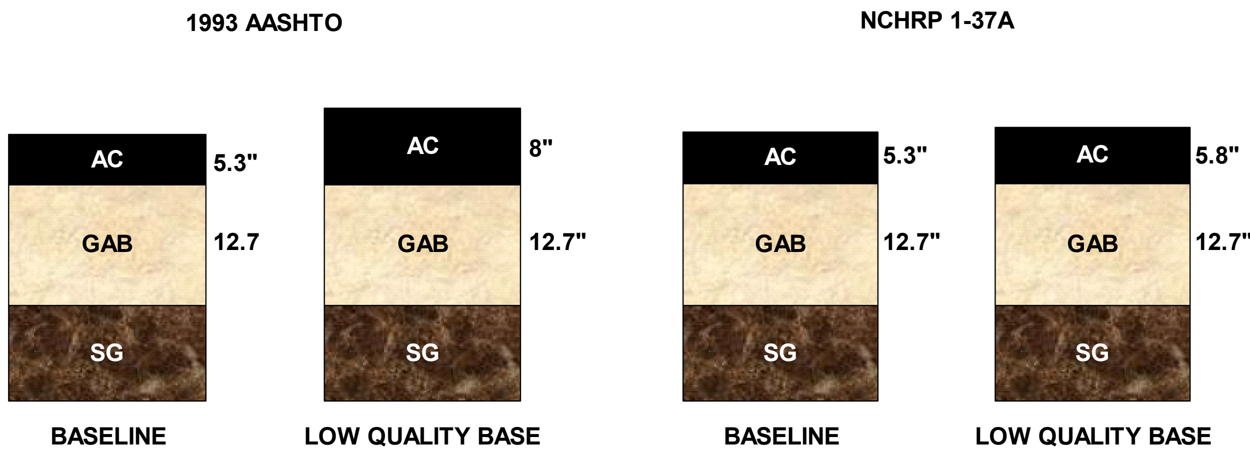


Figure 6-12. Summary of flexible pavement sections: low quality base scenario.

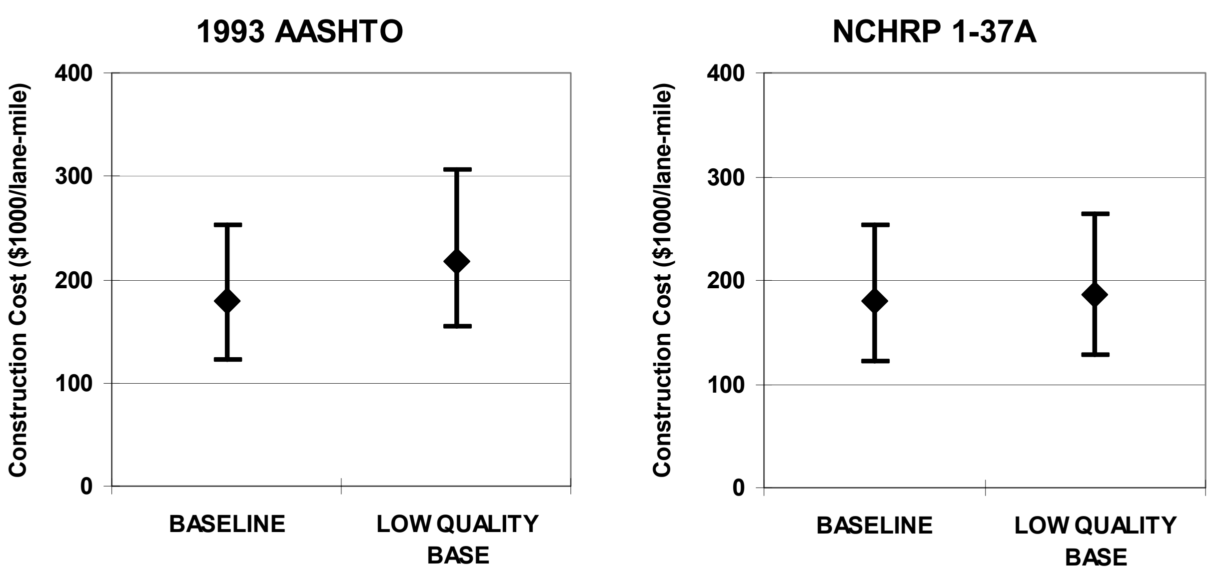


Figure 6-13. Example construction costs for flexible pavement sections: low quality base scenario.

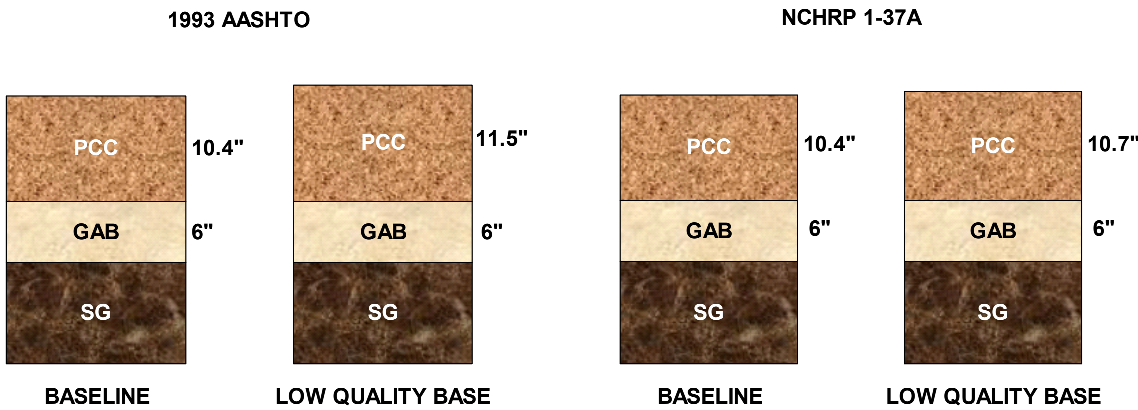


Figure 6-14. Summary of rigid pavement sections: low quality base scenario.

Initial construction cost ranges (based on Table 6-1) for the rigid pavement designs are summarized in Figure 6-15. Note again that these costs are very approximate; in particular, the same unit cost has been used for both the high quality (baseline) and low quality granular base materials, but in reality these would likely be different. The initial construction costs increase by about \$31K (9%) per lane-mile for the 1993 AASHTO designs and by \$8.4K (2.5%) per lane-mile for the NCHRP 1-37A pavement sections. Note again that the overall magnitudes of these initial construction costs for rigid pavements (Figure 6-15) cannot be fairly compared to the initial construction costs for flexible pavements (Figure 6-13) because of the different assumptions regarding initial service life and different maintenance and repair expenses over the design lives of these different pavement classes. A fair comparison would require evaluation of life-cycle costs, including maintenance, repair, rehabilitation, and perhaps user costs, in addition to the initial construction expense.

The base quality in this scenario is probably not sufficiently low to present a serious problem in design. However, if low base quality does become a critical issue and no high quality crushed material is available, cement or bituminous stabilization could be employed to improve the base quality substantially. Geosynthetics can also be employed for drainage and separation. These techniques are described more fully in Chapter 7.

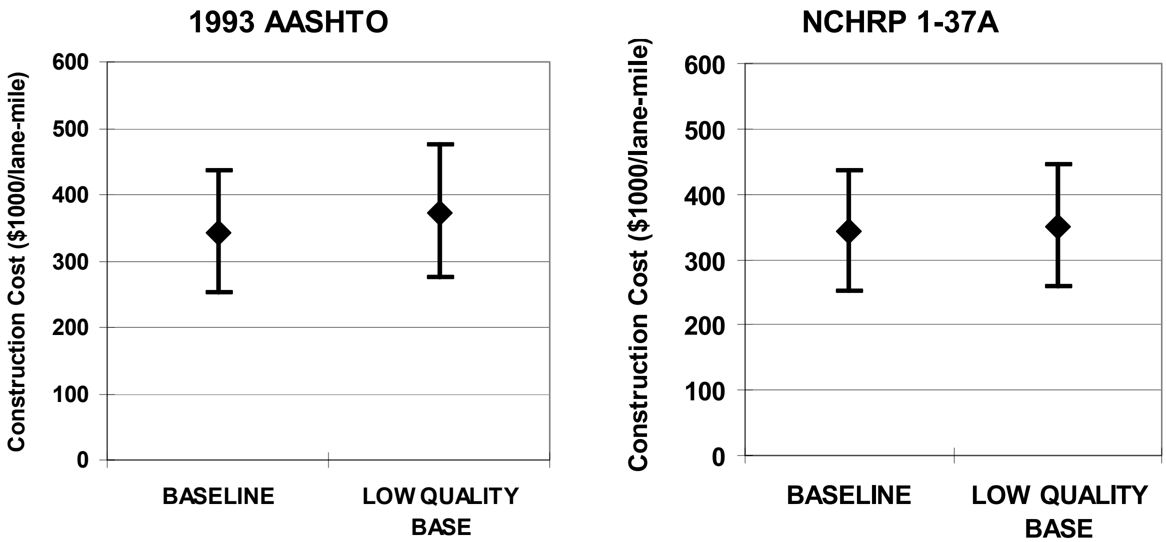


Figure 6-15. Example construction costs for rigid pavement sections: low quality base scenario.

6.6 POOR DRAINAGE

Good drainage conditions were explicitly assumed in the baseline design scenario. The values for the drainage coefficients in the 1993 AASHTO designs corresponded to a “good” drainage quality rating, defined as “water removed within 2 hours.” This implies a high-permeability base layer (like the A-1-a material assumed in the baseline design scenario) and functioning edge drains. The material properties for the granular base and the drainage length specified in the NCHRP 1-37A design procedure were also consistent with a high-permeability base layer and functioning edge drainage.

In this poor drainage scenario, the material characteristics for the granular base layer remain the same as for the baseline conditions. However, it is now assumed that the edge drains are clogged (or perhaps nonexistent), and the consequences of the ineffective drainage are evaluated. Conceptually, these expected consequences are

- a decrease in the stiffness (and strength) of the granular base layer because of higher average moisture content;
- a decrease in the stiffness (and strength) of the subgrade because of higher average moisture content;
- an increase in other moisture-related distresses (*e.g.,* freeze/thaw cycles, mixing of granular base with subgrade, etc.).

6.6.1 1993 AASHTO Design

Flexible Pavement

Three modifications to the 1993 AASHTO flexible pavement baseline inputs (Table 6-2) are appropriate for simulating the poor drainage condition:

1. The seasonally averaged resilient modulus and corresponding structural layer coefficient for the granular base material is reduced to reflect the higher average moisture content due to the ineffective drainage. From Table 5-39, the granular base resilient modulus value for wet conditions may be as low as about 40% of that for dry conditions. The E_{BS} = 40,000 psi value for excellent drainage conditions in the baseline design scenario is therefore reduced 35% to 26,000 psi for the poor drainage condition. This corresponds to a structural layer coefficient a_2 = 0.12 using the AASHTO correlation in Eq. (5.16). Note that these values coincide with those for the low quality granular base scenario in Section 6.5.
2. The base layer drainage coefficient m_2 is reduced from 1.0 to 0.6. This corresponds to a reduction in drainage quality rating from "good" to "very poor" under the assumption that the pavement is exposed to moisture conditions approaching saturation for 5-25% of the time.
3. The seasonally averaged resilient modulus for the subgrade is reduced to reflect the higher average moisture content due to ineffective drainage. From Table 5-39, wet resilient modulus values for granular base and subbase materials may be as low as about 40% of those for dry conditions. It is likely that the stiffness decrease in moisture-sensitive fine-grained subgrade soils will be even greater. Consequently, it is assumed here that the M_R = 7,500 psi value for good drainage conditions in the baseline design scenario is reduced by about 50% to 3,800 psi for the poor drainage condition (all M_R values are after seasonal adjustment). Note that this value coincides with that for the weak subgrade scenario in Section 6.3.

The W_{18} traffic capacity for baseline flexible pavement section under these poor drainage conditions is only 0.67x10^6 ESALs, corresponding to an 89% decrease in initial service life.

The required structure is then determined from the design equations, as follows (again assuming changes only in the AC layer thickness):

- Required overall structural number SN = 5.76 (unchanged from baseline conditions)
- Structural number provided by granular base (same thickness D_2 as in baseline design) $SN_2 = m_2 a_2 D_2 = (0.6)(0.12)(12.7) = 0.91$
- Required asphalt structural number $SN_1 = SN - SN_2 = 5.67 - 0.91 = 4.76$
- Required asphalt layer thickness $D_1 = \dfrac{SN_1}{a_1} = 10.8$ inches

The design for the poor drainage condition is thus 10.8 inches of AC over 12.7 inches of GAB (before rounding). This represents a significant increase in AC thickness, as compared to the baseline AC design thickness of 5.3 inches. The additional asphalt thickness required for the poor drainage condition is slightly more than the additional asphalt thicknesses for the soft subgrade and low quality base scenarios combined. This increase in asphalt thickness translates to a 43% initial construction cost increase of about $78K per lane-mile (using the typical unit cost data in Table 6-1).

Rigid Pavement

Four modifications to the 1993 AASHTO rigid pavement baseline inputs (Table 6-2) are appropriate for simulating the poor drainage condition:

1. The resilient modulus for the granular subbase material is reduced to reflect the higher average moisture content due to the ineffective drainage. Consistent with the calculations for the 1993 AASHTO flexible design, the E_{SB} = 40,000 psi value for excellent drainage conditions in the baseline design scenario is reduced to 26,000 psi for the poor drainage condition.
2. The Loss of Support LS coefficient is increased from 2 to 3 to reflect the increased erosion potential in the granular subbase due to the increased moisture levels.
3. The resilient modulus for the subgrade is reduced to reflect the higher average moisture content due to the ineffective drainage. Consistent with the calculations for the 1993 AASHTO flexible design in the preceding subsection, the M_R = 7,500 psi value for good drainage conditions in the baseline design scenario is reduced to 3,800 psi for the poor drainage condition (all M_R values are after seasonal adjustment).
4. The drainage coefficient C_d is reduced from 1.0 to 0.75. This corresponds to a reduction in drainage quality rating from "good" to "very poor" under the assumption that the pavement is exposed to moisture conditions approaching saturation for 5-25% of the time.

The W_{18} traffic capacity for the baseline rigid pavement section under these postulated poor drainage conditions is reduced to 5.2×10^6 ESALs, corresponding to a 68% decrease in initial service life.

The first three changes listed above will have a direct effect on the design modulus of subgrade reaction k_{eff}, which reduces from its baseline value of 38 pci to a value of 12 pci for the poor drainage conditions. The 1993 AASHTO design corresponding to this k_{eff} and C_d of 0.75 consists of a 12.3 inch PCC slab over 6 inches of GAB. Note that this design is substantially thicker than the 10.4" PCC over 6" subbase section for the 1993 AASHTO baseline scenario; this corresponds to an initial construction cost increase of $53K (16%) per lane-mile. As was the case for the low quality subbase scenario, the increase in slab thickness is due primarily to the reduction in the drainage coefficient C_d attributable to the subbase; only about 0.25 inches of the slab thickness increase is due to the increased erodibility of the subbase as reflected in the larger *LS* value and the consequently reduced value for k_{eff}.

6.6.2 NCHRP 1-37A Design

Flexible Pavement

The current version of the NCHRP 1-37A design software does not yet include the capability for directly modeling drainage influences. This in part is due to the paucity of field data available for calibrating the empirical distress models for drainage effects. As stated in the NCHRP 1-37A final report (NCHRP 1-37A, 2004):

> "The calibration of the flexible pavement distress models assumed that no infiltration of moisture occurred throughout the design period. Thus, the flexible pavement design procedure does not allow the designer to choose any level of infiltration at this time. However, adequate consideration to subdrainage should be given when designing flexible pavements."

Although it would be possible to run the NCHRP 1-37A analyses using the moduli values assumed for the 1993 AASHTO designs (Section 6.6.1), this defeats the purpose of including the analysis of seasonal moisture variations on material properties in the mechanistic-empirical methodology. It is also probable that this approximate approach would underestimate the detrimental effects of excess moisture in the pavement structure.

Rigid Pavement

Three modifications to the NCHRP 1-37A rigid pavement baseline inputs (Table 6-9) are appropriate for simulating the poor drainage condition:

1. The surface infiltration condition is changed from "Minor (10%)" to "Extreme (100%)".
2. The granular base layer is changed from a free-draining A-1-a to a much less permeable A-2-6 material. This is the same material as for the low quality base scenario in Section 6.5; the modified NCHRP 1-37A design inputs for this material are given in Table 6-14.
3. The Erodibility Index for the base layer is changed from "Fairly Erodible (4)" to "Very Erodible (5)" to reflect the increased fines content of the A-2-6 base material (see Table 6-16).

Figure 6-16 summarizes the predicted faulting vs. time for the baseline rigid pavement section (10.4″ PCC over 6.0″ granular base); the time to the 0.12 inch faulting design limit is only 24 years, corresponding to a 6% decrease in initial service life due to the low poor drainage conditions.

The trial designs and their corresponding predicted performance at end of design life are listed in Table 6-18. Also shown in the table is the design limit for predicted faulting, which is the controlling distress for this scenario. A pavement section consisting of a 10.7 inch PCC slab over 6 inches of GAB meets the design faulting limit for the poor drainage scenario. This pavement section is significantly thinner than the 12.3 inch slab required by 1993 AASHTO design for this scenario, but only 0.3 inches thicker than the design for the baseline conditions, corresponding to an \$8.5K per lane-mile (2.5%) increase in initial construction costs.

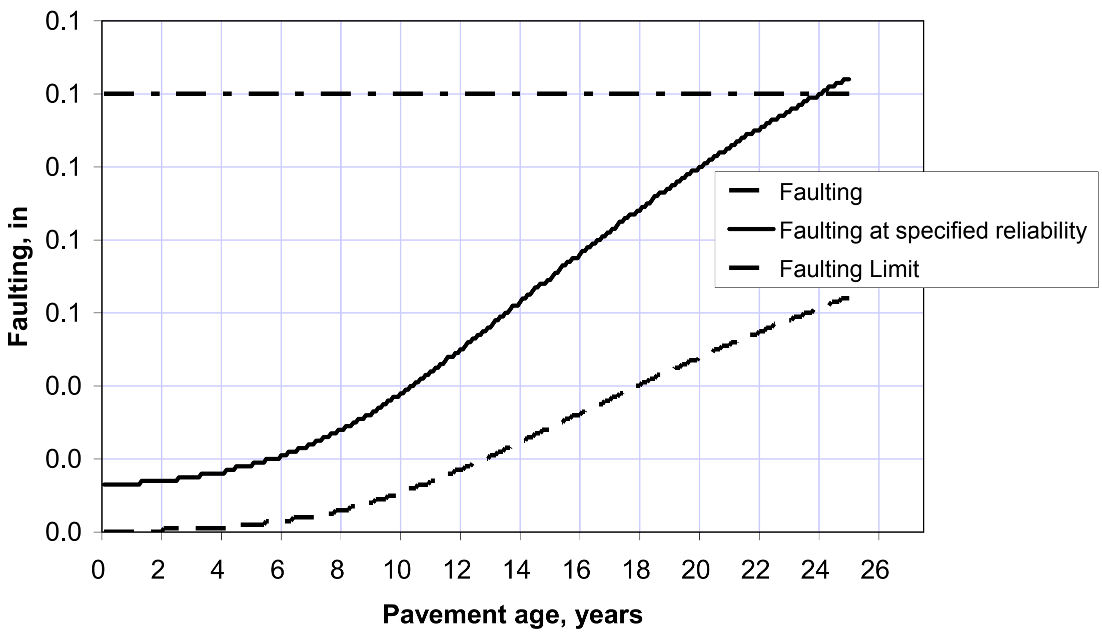


Figure 6-16. Predicted faulting performance for poor drainage scenario.

Table 6-18. Trial cross sections for NCHRP 1-37A rigid pavement design: poor drainage scenario.

PCC Thickness (in.)	Base Thickness (in.)	Faulting (in.)
10.4	6	0.124
10.7	6	0.119
11.0	6	0.113
	Design Limit:	0.12

6.6.3 Summary

The design flexible pavement sections for the baseline and poor drainage scenarios are summarized in Figure 6-17. Only designs from the 1993 AASHTO procedure are included here; as described previously, the NCHRP 1-37A procedure in its present form does not have the capability to analyze this scenario. As expected, the poor drainage condition mandates a substantially thicker pavement cross section, with the required thickness of the asphalt increasing from 5.3 to 10.8 inches (for the simple assumption of constant graded aggregate base thickness).

Initial construction cost ranges (based on Table 6-1) for the flexible pavement designs are summarized in Figure 6-18. The initial construction costs based on the typical unit costs in Table 6-1 increase by about $78K or 44% per lane-mile as a consequence of the poor drainage conditions for the 1993 AASHTO designs.

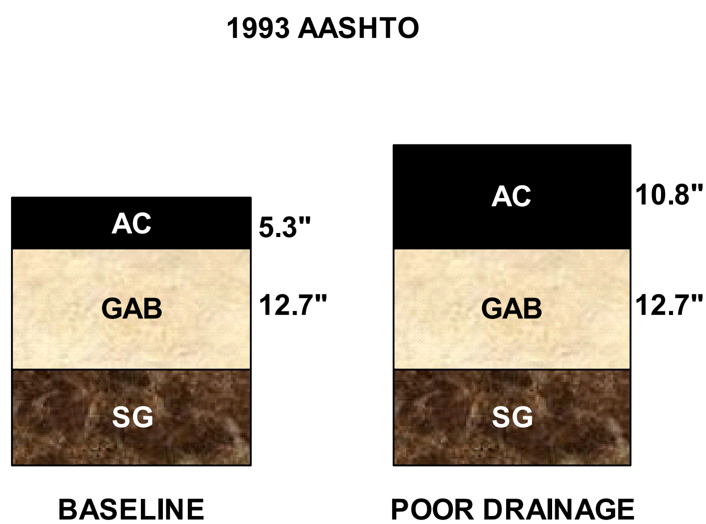


Figure 6-17. Summary of flexible pavement sections: poor drainage scenario.

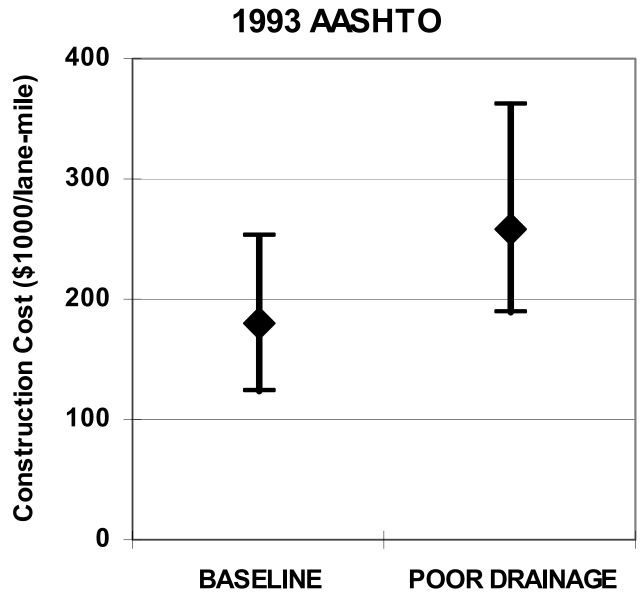


Figure 6-18. Example construction costs for flexible pavement sections: poor drainage scenario.

The design rigid pavement sections for the baseline and poor drainage scenarios are summarized in Figure 6-19. Again, the poor drainage necessitates a thicker pavement cross section. For the 1993 AASHTO design, the required thickness for the PCC slab increases substantially from 10.4 to 12.3 inches (before rounding). For the NCHRP 1-37A designs, the PCC slab thickness increases only from 10.4 to 10.7 inches. These disparities suggest that the 1993 AASHTO procedure overestimates and/or the NCHRP 1-37A methodology underestimates the impact of poor drainage on pavement performance and design requirements. Poor drainage impacts the unbound layers most significantly, and so these disparities are consistent with the trends observed in the low quality base scenario in Section 6.5.

Initial construction cost ranges (based on Table 6-1) for the rigid pavement designs are summarized in Figure 6-20. The initial construction costs increase by about $54K (16%) per lane-mile due to the poor drainage for the 1993 AASHTO designs and by about $8.5K (2.5%) per lane-mile in the NCHRP 1-37A designs. Note again that these construction costs for rigid pavements (Figure 6-20) cannot be fairly compared to the construction costs for flexible pavements (Figure 6-18) because of the different initial service lives and the different maintenance and repair expenses incurred over their design lives. A fair comparison would require evaluation of life-cycle costs, including maintenance, repair, rehabilitation, and perhaps user costs, in addition to the initial construction expense.

These examples clearly show the very substantial effect that ineffective or nonexistent drainage can have on the pavement design thickness. Methods for ensuring adequate drainage in pavements are described in Chapter 7.

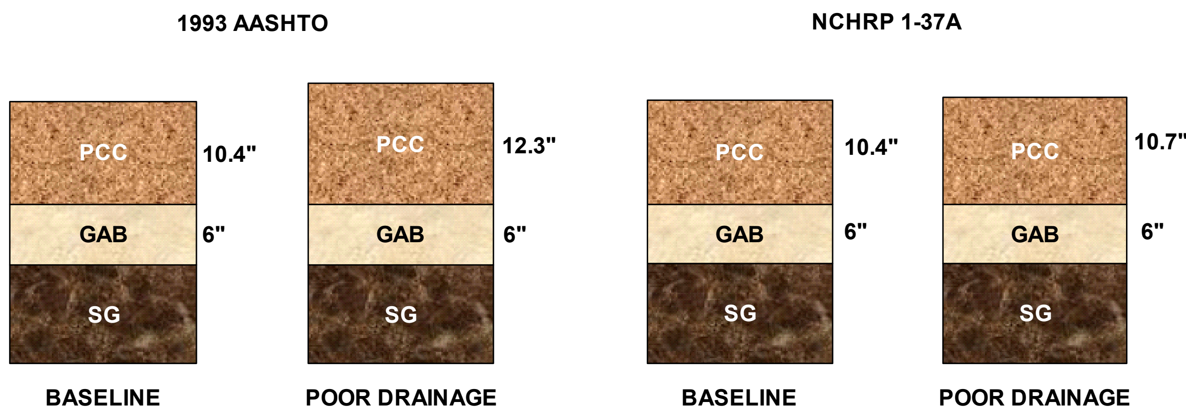


Figure 6-19. Summary of rigid pavement sections: poor drainage scenario.

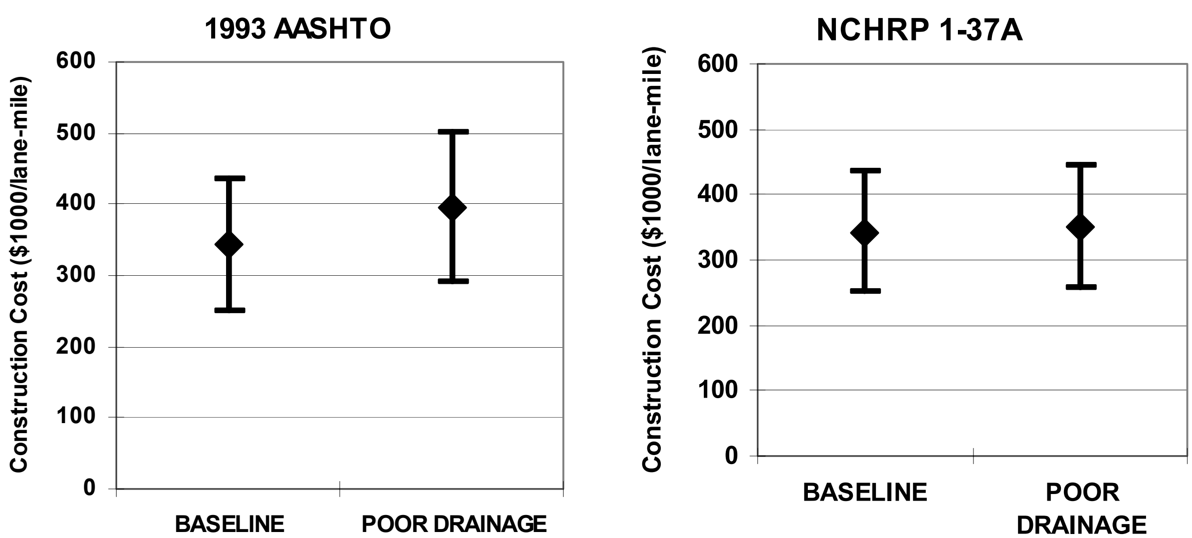


Figure 6-20. Example construction costs for rigid pavement sections: poor drainage scenario.

6.7 SHALLOW BEDROCK

6.7.1 1993 AASHTO Design

Flexible Pavement

Although the presence of shallow bedrock will clearly have a beneficial stiffening effect on the pavement foundation, the 1993 AASHTO Guide does not include any provision for including this benefit in the flexible design procedure. Consequently, the design pavement section for the shallow bedrock scenario will be identical to that for the baseline conditions

Rigid Pavement

The presence of shallow bedrock will increase the design modulus of subgrade reaction k_{eff} via the shallow bedrock correction factor (Figure 5.25). Values of k_{eff} for several bedrock depths are summarized in Table 6-19, along with the corresponding values of required slab thickness. All design inputs other than depth to bedrock are equal to the baseline conditions (Table 6-3), and the granular subbase thickness is held constant at 6 inches. The results in Table 6-19 indicate that, for the design conditions in this example, the depth to bedrock has a moderate effect on the design modulus of subgrade reaction k_{eff}, but that this has a nearly negligible effect on the required slab thickness. The design pavement section for the case of a 4 foot depth to bedrock is a slightly thinner <u>10.3 inch PCC slab over 6 inches of GAB</u>. Viewed alternatively, bedrock at a 4 foot depth will increase the allowable traffic W_{18} to 17.0×10^6, increasing the initial service life of the baseline rigid pavement section by approximately 1 year (4%).

Table 6-19. Design pavement sections for 1993 AASHTO rigid pavement design: shallow bedrock effects.

Depth to bedrock (ft)	∞	8	4	2
Composite subgrade modulus k_∞ (pci)	423	474	577	704
Design subgrade modulus k_{eff} (pci)	38	41	46	52
Slab thickness *D* (in.)	10.4	10.4	10.3	10.3

6.7.2 NCHRP 1-37A Design

Flexible Pavement

Depth to bedrock is an explicit design input in the NCHRP 1-37A procedure. Table 6-20 summarizes the predicted total rutting (adjusted for reliability) for the baseline flexible pavement conditions (Table 6-4) as a function of depth to bedrock, as predicted by the NCHRP 1-37A methodology. For the conditions in this example, a 4 foot depth to bedrock will reduce the predicted rutting by about 17%, or increase the initial service life by about 18 years (120%). This translates directly to a thinner required pavement section. Trial designs and their corresponding predicted performance at end of design life for the case of a 3 foot depth to bedrock are summarized in Table 6-21. A pavement section consisting of 3 inches of AC over 12.7 inches of GAB is sufficient for the shallow bedrock scenario. This corresponds to an initial construction cost reduction of about $38K (18%).

Table 6-20. Influence of bedrock depth on predicted total rutting: NCHRP 1-37A design methodology.

Depth to Bedrock (ft)	Total Rutting (in.)
∞	0.646
10	0.578
8	0.544
4	0.533

Table 6-21. Trial cross sections for NCHRP 1-37A flexible pavement design: baseline conditions with shallow bedrock at 4 ft depth.

AC Thickness (in.)	Base Thickness (in.)	Total Rutting (in.)
5.3	12.7	0.533
4.0	12.7	0.593
3.0	12.7	0.651
	Design Limit:	0.65

Rigid Pavement

Depth to bedrock is an explicit design input in the NCHRP 1-37A procedure. Table 6-22 summarizes the predicted joint faulting (adjusted for reliability) for the baseline rigid pavement conditions (Table 6-9) as a function of depth to bedrock, as predicted by the NCHRP 1-37A methodology. For the conditions in this example, a 4 foot depth to bedrock will reduce the predicted faulting by about 75%. This translates directly to a thinner required pavement section. Trial designs and their corresponding predicted performance at end of design life for the case of a 4 foot depth to bedrock are summarized in Table 6-23. The NCHRP 1-37A software cannot model PCC thicknesses less than 7 inches. Even at this reduced thickness, however, the predicted faulting for the shallow bedrock conditions is well below the design limit.

Table 6-22. Influence of bedrock depth on predicted joint faulting: NCHRP 1-37A design methodology.

Depth to Bedrock (ft)	Faulting (in.)
∞	0.117
20	0.093
10	0.040
8	0.033
4	0.028

Table 6-23. Trial cross sections for NCHRP 1-37A rigid pavement design: baseline conditions with shallow bedrock at 3 ft depth.

PCC Thickness (in.)	Base Thickness (in.)	Faulting (in.)
10.4	6.0	0.028
9.0	6.0	0.041
7.0	6.0	0.079
	Design Limit:	0.12

6.8 CONCLUDING COMMENTS

It is worthwhile to review the primary objectives of the design studies presented in this chapter:

1. to illustrate via examples how the geotechnical properties described in Chapter 5 are incorporated in the pavement design calculations in the 1993 AASHTO and NCHRP 1-37A procedures; and
2. to highlight the effects of the geotechnical factors and inputs on pavement design performance.

The design scenarios are intentionally highly idealized and simplified. Their point is to emphasize in quantitative terms how changes in geotechnical inputs affect the overall pavement design and performance. These design scenarios are also good examples of the types of sensitivity studies one should perform during design to evaluate the importance of the various design inputs, especially with reference to the quality of the information used to determine these inputs. As succinctly stated by Hamming (1973) in the frontispiece of his pioneering book on engineering computation: "The purpose of computing is insight, not numbers."

The ultimate measure for comparing the designs for the various scenarios is cost. Life-cycle costs are the best measure, but calculation of life-cycle costs is beyond the scope of these exercises. As a fallback, the various design scenarios can be compared in terms of their initial construction cost ratios using the example unit cost data from Table 6-1 and the baseline design conditions as the reference (*i.e.,* cost index=1). These comparisons are presented in Figure 6-21 and Figure 6-22 for flexible and rigid pavement designs, respectively. Remember that it is inappropriate to compare flexible versus rigid pavements based only on initial construction costs because of the different assumptions regarding initial service life and different maintenance and repair expenses that would be incurred over the design lives of these different pavement classes.

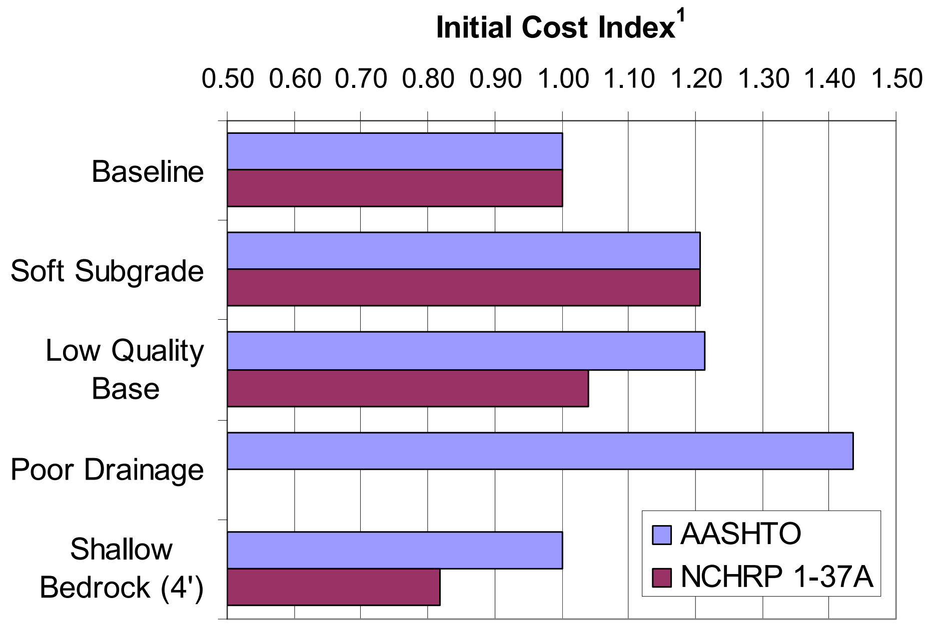
Some key observations to be drawn from Figure 6-21 and Figure 6-22 include the following:

- Poor drainage is by far the most detrimental geotechnical factor for flexible pavements. For the conditions in these examples, poor drainage drives up the initial cost of the flexible pavement design by nearly 50%.
- For the flexible pavement scenarios considered here, a very soft subgrade is the second most detrimental geotechnical factor, driving initial costs up by about 20%. However, the soft subgrade condition can be mitigated by lime stabilization or other remedial measures, as described in more detail in Chapter 7.

- More thickness may be required if lower quality materials are used for the granular base layers in flexible pavements, but the thickness increase may be partially or fully compensated by lower unit costs for the lower quality materials. This will be very sensitive to the specific unit costs of competing materials in the project location.
- Overall, the rigid pavement designs were much less sensitive to geotechnical factors than were the flexible designs. The range and variations in the cost index among the various design scenarios were much less for the rigid pavement designs (Figure 6-22) than for the flexible pavements (Figure 6-21).

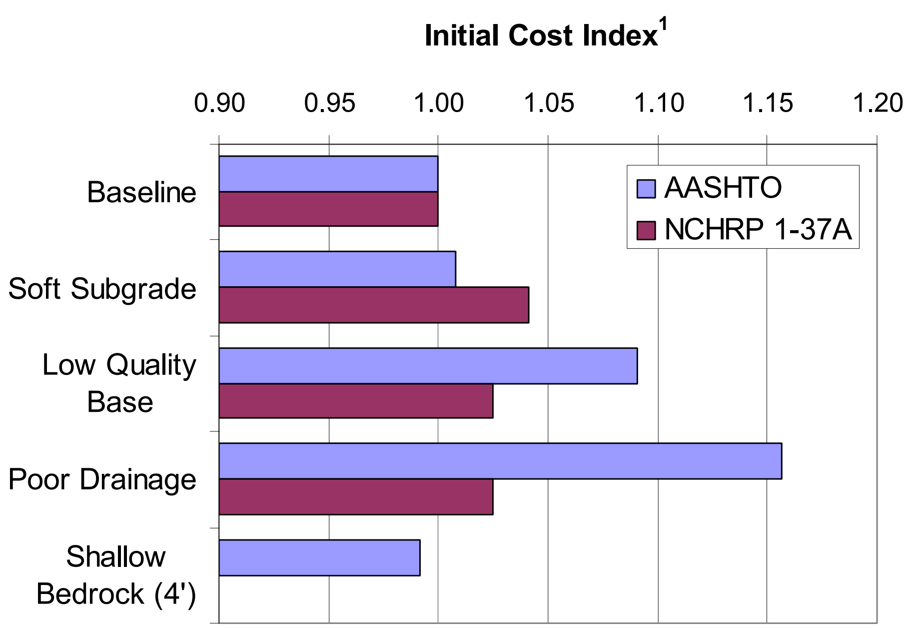
An alternate way of looking at the impact of the various geotechnical design parameters is in terms of their effect on initial service life for the baseline pavement section. Initial service lives for the various design scenarios are summarized in Figure 6-23 and Figure 6-24 for the flexible and rigid pavement sections, respectively. The observations in terms of service life are similar to those from costs. For the flexible pavements, poor drainage, soft subgrade, and low quality base had the most impact on service life (especially for the AASHTO designs), reducing the initial service life from 15 to as little as 2 years in the most extreme case. For the rigid pavements, poor drainage and low quality base again had the most significant impact (again, especially for the AASHTO designs), reducing the initial service life from 25 to as little as 8 years. The soft subgrade conditions had comparatively less effect on the rigid pavement service life, as compared to the flexible pavements, confirming the advantages of rigid pavements for very poor foundation conditions.

Of course, all of the specific observations from these results apply only to the particular design scenarios considered in these illustrative studies. Pavement design conditions and in-place unit costs will vary considerably across agencies and regions. Nevertheless, the simple design scenarios presented here demonstrate quite convincingly the important effects that geotechnical factors can have on design pavement sections and costs.



[1]Based on typical unit cost data from Table 6-1. For low quality base scenario, unit cost for granular base is assumed to be the same as for high quality base; in reality, this unit cost will likely be lower.

Figure 6-21. Summary of costs for example design scenarios: flexible pavement designs.



[1]Based on typical unit cost data from Table 6-1. For low quality base scenario, unit cost for granular base is assumed to be the same as for high quality base; in reality, this unit cost will likely be lower.

Figure 6-22. Summary of costs for example design scenarios: rigid pavement designs.

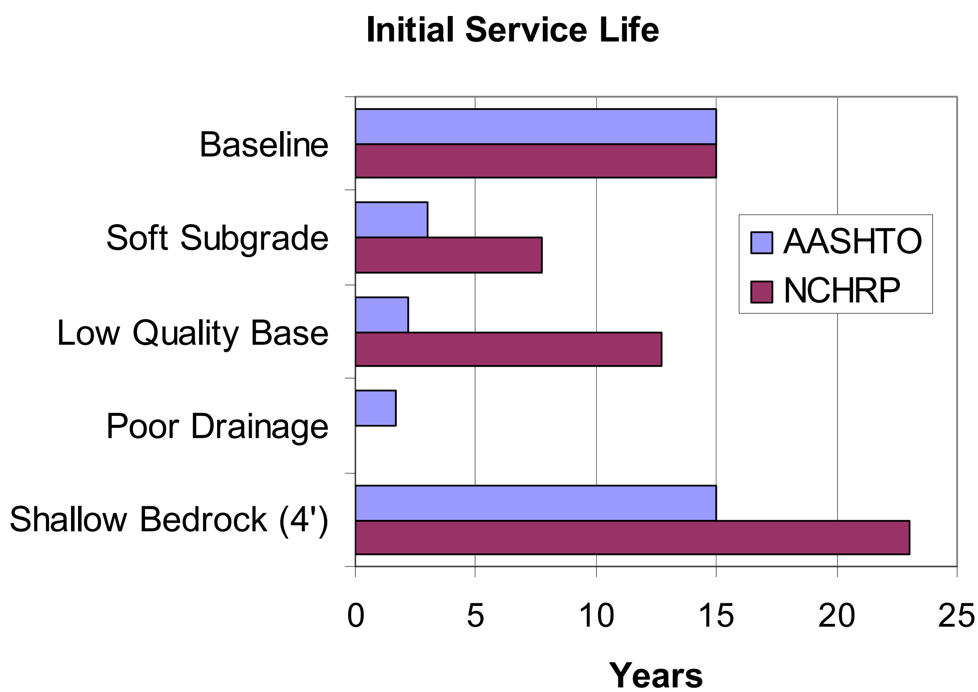


Figure 6-23. Initial service lives for example design scenarios: flexible pavement designs.

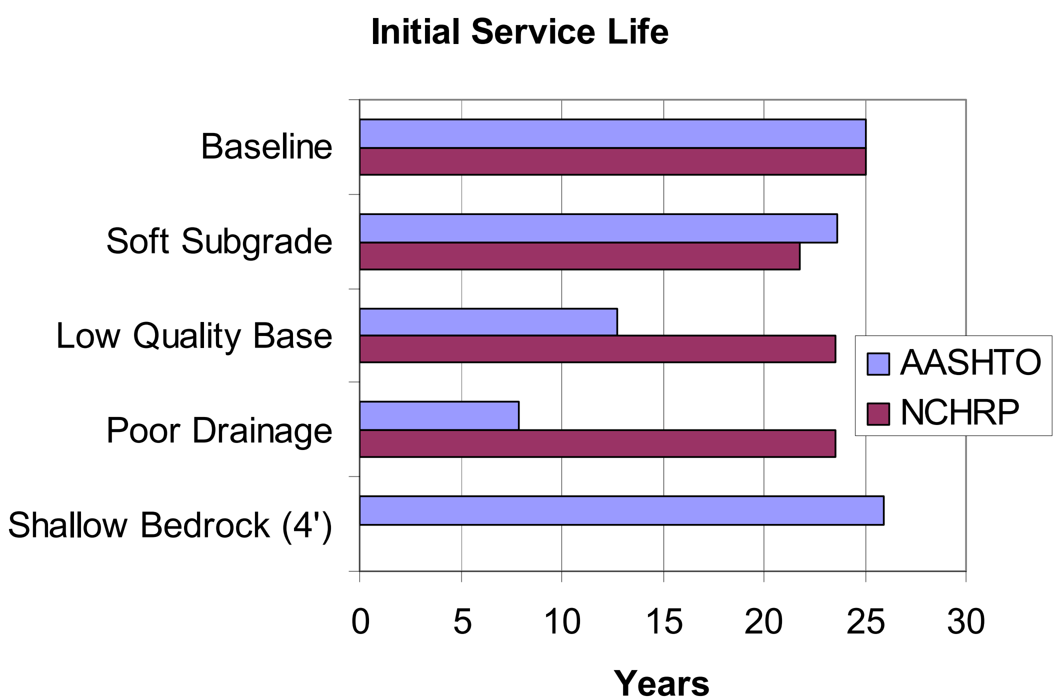


Figure 6-24. Initial service lives for example design scenarios: rigid pavement designs.

6.9 EXERCISES

Students will divide into groups to develop designs for the Main Highway project. Each group will focus on either a flexible or a rigid pavement design. Specific tasks for each group are as follows:

- Develop a pavement design based on the most-likely conditions expected at the site.
- Identify the short list of critical geotechnical inputs to which the design is expected to be most sensitive.
- Perform some initial sensitivity evaluation for the critical geotechnical inputs.
- Complete computations using a simple Excel spreadsheet provided by the instructors.

6.10 REFERENCES

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CHAPTER 7.0 DESIGN DETAILS AND CONSTRUCTION CONDITIONS REQUIRING SPECIAL DESIGN ATTENTION

7.1 INTRODUCTION

This chapter includes:

- Design details for key geotechnical components in the pavement system, including drainage and base layer requirements.
- Compaction of subgrades and unbound pavement layers.
- A general overview of the types of potential subgrade problems.
- Identification and treatment of select, widely occurring special geotechnical challenges, including collapsible or highly compressible soils, expansive or swelling soils, subsurface water and saturated soils, and frost-susceptible soils.
- Detailed guidelines on alternate stabilization methods, which are often used to mitigate special problems.

In this chapter, design details for the specific pavement features of base materials and drainage systems are provided. Compaction, one of the key geotechnical issues in pavement design and construction is also covered.

Special challenges generally relate to poor subgrade conditions that occur due to the type of soil and environmental conditions. In this chapter, the various types of problematic soil conditions are reviewed along with widely occurring specific subgrade problems. Although these problematic conditions can be detected with a detailed subsurface exploration, problematic conditions can potentially go unnoticed if they are located between borings.

7.2 SUBSURFACE WATER AND DRAINAGE REQUIREMENTS

The damaging effects of excess moisture on the pavement have long been recognized. Moisture from a variety of sources can enter a pavement structure. This moisture, in combination with heavy traffic loads and freezing temperatures, can have a profound negative effect on both material properties and the overall performance of a pavement system.

As was shown in Figure 3-3, Chapter 3, moisture in the subgrade and pavement structure can come from many different sources. Water may seep upward from a high groundwater table, or it may flow laterally from the pavement edges and shoulder ditches. Knowledge of groundwater and its movement are critical to the performance of the pavement as well as stability of adjacent sideslopes, especially in cut situations. Groundwater can be especially troublesome for pavements in low-lying areas. Thus, groundwater control, usually through interception and removal before it can enter the pavement section, is an essential part of pavement design.

In some cases, pavements are constructed beneath the permanent or a seasonally high watertable. Obviously, drainage systems must perform or very rapid pavement failure will occur. In such cases, redundancy in the drainage design is used (*e.g.,* installation of underdrains and edgedrains) and, often, some monitoring is used to ensure continual function of the drain system.

Capillary action and moisture-vapor movement are also responsible for water accumulating beneath a pavement structure (Hindermann, 1968). Capillary effects are the result of surface tension and the attraction between water and soil. Moisture vapor movement is associated with fluctuating temperatures and other climatic conditions.

As was previously indicated in Chapter 3, the most significant source of excess water in pavements is typically infiltration through the surface. Joints, cracks, shoulder edges, and various other defects in the surface provide easy access paths for water. A study by the Minnesota Department of Transportation indicates that 40% of rainfall enters the pavement structure (Hagen and Cochran 1995). Demonstration Project 87, *Drainable Pavement Systems*, indicates that surface infiltration is the single largest source of moisture-related problems in PCC pavements (FHWA 1994). Although AC pavements do not contain joints, surface cracks, longitudinal cold joints that crack, and pavement edges provide ample pathways for water to infiltrate the pavement structure.

The problem only worsens with time. As pavements continue to age and deteriorate, cracks become wider and more abundant. Meanwhile, joints and edges become more deteriorated and develop into channels through which moisture is free to flow. The result is more moisture being allowed to enter the pavement structure with increasing pavement age, which leads to accelerated development of moisture-related distresses and pavement deterioration.

7.2.1 Moisture Damage Acceleration

Excessive moisture within a pavement structure can adversely affect pavement performance. A pavement can be stable at a given moisture content, but may become unstable if the materials become saturated. High water pressures can develop in saturated soils when subjected to dynamic loading. Subsurface water can freeze, expand, and exert forces of considerable magnitude on a given pavement. Water in motion can transport soil particles and cause a number of different problems, including clogging of drains, eroding of embankments, and pumping of fines. These circumstances must be recognized and accounted for in the design of a pavement.

The detrimental effects of water on the structural support of the pavement system are outlined by AASHTO (1993), as follows:

- Water in the asphalt surface can lead to moisture damage, modulus reduction, and loss of tensile strength. Saturation can reduce the dry modulus of the asphalt by as much as 30% or more.
- Added moisture in unbound aggregate base and subbase is anticipated to result in a loss of stiffness on the order of 50% or more.
- Modulus reduction of up to 30% can be expected for asphalt-treated base and increase erosion susceptibility of cement or lime treated bases.
- Saturated fine-grain roadbed soil could experience modulus reductions of more than 50%.

As noted in Chapters 3, 4, 5 and 6, modulus is the key pavement design property!

The influence of saturation on the life of the pavement is illustrated in Figure 7-1. The severity factor (shown in the figure) is the anticipated relative damage during wet versus dry periods anticipated for the type of road. As an example, Figure 7-1 shows that if the pavement system is saturated only 10% of its life (*e.g.,* about one month per year), a pavement section with a moderate stability factor will be serviceable only about 50% of its fully drained performance period. Specific distresses caused by excessive moisture within flexible and rigid pavements are summarized in Table 7-1 and 7-2, respectively.

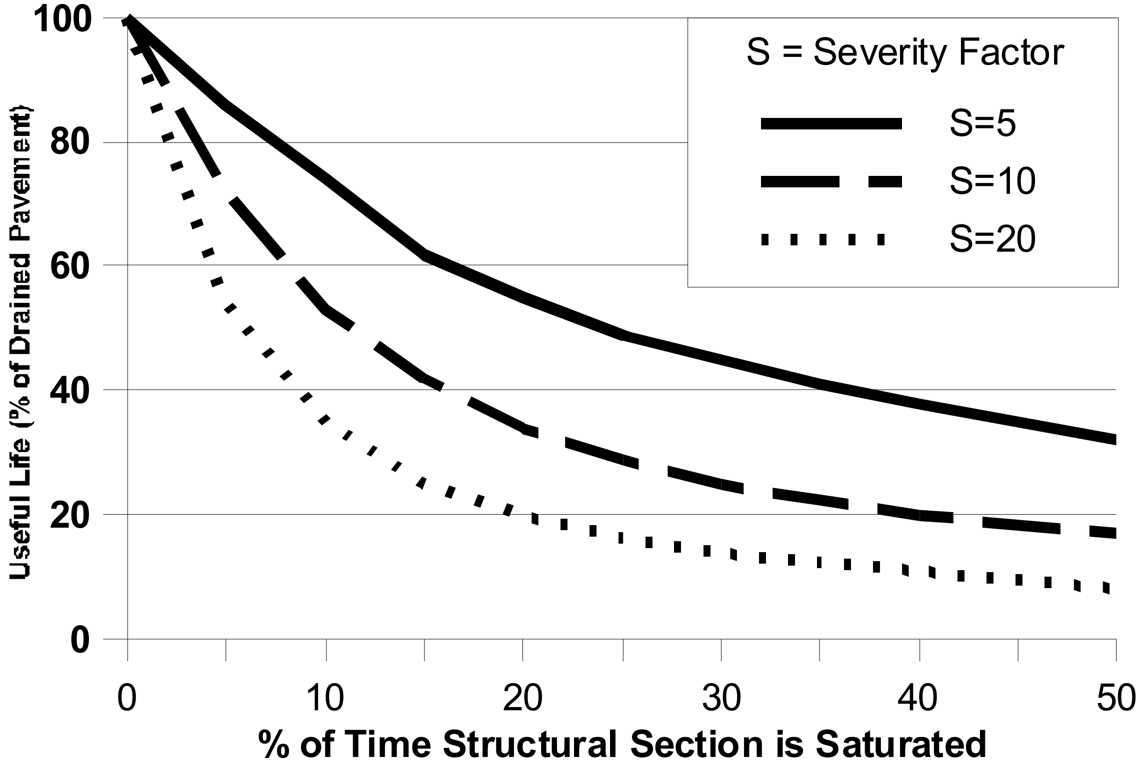

Figure 7-1. The influence of saturation on the design life of a pavement system (after Cedergren, 1987).

Table 7-1. Moisture-related distresses in flexible (AC) pavements (NHI 13126; adapted after Carpenter et al. 1979).

Type	Distress Manifestation	Moisture Problem	Climatic Problem	Material Problem	Load Associated Distress	Structural Defect Begins in		
						AC	Base	Subgrade
Surface Deformation	Bump or Distortion	Excess Moisture	Frost Heave	Volume Increase	No	No	No	Yes
	Corrugation or Rippling	Slight	Moisture and Temperature	Unstable Mix	Yes	Yes	Yes	No
	Stripping	Yes	Moisture	Loss of Bond	No	Yes	No	No
	Rutting	Excess in Granular Layers or Subgrade	Moisture	Plastic Deformation, Stripping	Yes	Yes	Yes	Yes
	Depression	Excess Moisture	Suction & Materials	Settlement, Fill Material	No	No	No	Yes
	Potholes	Excess Moisture	Moisture, Temperature	< Strength > Moisture	Yes	Yes	Yes	Yes
Cracking	Longitudinal	No; Accelerates	No	Construction	No	Faulty Construction	No	No
	Alligator (fatigue)	Yes; Accelerates	Spring-Thaw Strength loss	Thickness	Yes	Yes, Mix	Yes	No
	Transverse	No; Accelerates	Low Temp. Freeze-Thaw Cycles	Thermal Properties	No	Yes, Temp. Susceptible	No	No
	Slippage	Yes	No	Loss of Bond	Yes	Yes, Bond	No	No

Table 7-2. Moisture-related distresses in rigid (PCC) pavements (NHI 13126; adapted after Carpenter et al. 1979).

Type	Distress Manifestation	Moisture Problem	Climatic Problem	Material Problem	Load Associated Distress	Structural Defect Begins in		
						PCC	Base	Subgrade
Surface Defects	Spalling	Possible	Freeze/Thaw Cycles	Mortar	No	Yes	No	No
	Scaling	Yes	Freeze/Thaw Cycles	Chemical Influence	No	Yes; Finishing	No	No
	D-Cracking	Yes	Freeze/Thaw Cycles	Aggregate Expansion	No	Yes	No	No
	Crazing	No	No	Rich Mortar	No	Yes; Weak Surface	No	No
Surface Deformation	Blow-up	No	Temperature	Thermal Properties	No	Yes	No	No
	Pumping and Erosion	Yes	Moisture	Inadequate Strength	Yes	No	Yes	Yes
	Faulting	Yes	Moisture-Suction	Erosion-Settlement	Yes	No	Yes	Yes
	Curling/Warping	Yes	Moisture & Temperature	Moisture and Temperature Differentials	No	Yes	No	No
Cracking	Corner	Yes	Moisture	Cracking Follows Erosion	Yes	No	Yes	Yes
	Diagonal Transverse Longitudinal	Yes	Moisture	Follows Erosion	Yes	No	Yes	Yes
	Punchout (CRCP)	Yes	Moisture	Deformation Follows Cracking	Yes	No	Yes	Yes

7.2.2 Approaches to Address Moisture in Pavements

As was indicated in Chapter 3, to avoid moisture-related problems, a major objective in pavement design should be to keep the base, subbase, subgrade, and other susceptible paving materials from becoming saturated or even exposed to constant high moisture levels. The three approaches described in detail in Chapter 3 for controlling or reducing the problems caused by moisture are

- prevent moisture from entering the pavement system.
- use materials and design features that are insensitive to the effects of moisture.
- quickly remove moisture that enters the pavement system

No single approach can completely negate the effects of moisture on the pavement system under heavy traffic loading over many years. For example, it is practically impossible to completely seal the pavement, especially from moisture that may enter from the sides or beneath the pavement section. While materials can be incorporated into the design that are insensitive to moisture, this approach is often costly and in many cases not feasible (*e.g.,* may require replacing the subgrade). Drainage systems also add cost to the road. Maintenance is required for both drainage systems and sealing systems, for them to effectively perform over the life of the system. Thus, it is often necessary to employ all approaches in combination to obtain the most effective design. The first two approaches involve the surficial pavement materials, which are well covered in the NHI courses on pavement design (*e.g.,* NHI 131060A "Concrete Pavement Design Details and Construction Practices" and the participant's manual) and will not be covered herein. The geotechnical aspects of these approaches include drainage systems for removal of moisture, the requirements of which will be reviewed in the following subsections. Durable base material requirements will be reviewed in the subsequent section, and followed by subgrade stabilization methods to mitigate moisture issues in the subgrade. A method of sealing to reduce moisture intrusion into the subgrade will also be reviewed in the subgrade stabilization section.

7.2.3 Drainage in Pavement Design

Removal of free water in pavements can be accomplished by draining the free water vertically into the subgrade, or laterally though a drainage layer into a system of collector pipes. Generally, the actual process will be a combination of the two (ASSHTO, 1993). Typically in wet climates, if the subgrade permeability is less than 3 m/day (10 ft/day), some form of subsurface drainage or other design features to combat potential moisture problems should be considered. Table 7-3 provides additional climatic conditions and traffic considerations to assist in the assessment of the need for subsurface drainage.

The quality of drainage is defined in both AASHTO 1993 and NCHRP 1-37A based on the principle of time-to-drain. Time-to drain is the time required following any significant rainfall event for a pavement system to drain from a saturated state to a specific saturation or drainage level (*e.g.,* 50% drainage level in AASHTO 1993). The concept can also be applied (at least qualitatively) to other significant moisture events that would saturate the pavement (*i.e.,* flood, snow melt, or capillary rise). The definitions of poor to excellent drainage provided by AASHTO (1993) are given in Table 7-4.

Table 7-3. Assessment of need for subsurface drainage in new or reconstructed pavements (NCHRP 1-37 A, adapted after NHI 13126).

Climatic Condition	**Greater than 12 million 20-yr design lane heavy trucks**			**Between 2.5 and 12 million 20-yr design lane heavy trucks**			**Less than 2.5 million 20-yr design lane heavy trucks**		
	$k_{subgrade}$ (m/day)								
	< 3	3 to 30	> 30	< 3	3 to 30	> 30	< 3	3 to 30	> 30
Wet-Freeze	R	R	F	R	R	F	F	NR	NR
Wet-No Freeze	R	R	F	R	F	F	F	NR	NR
Dry-Freeze	F	F	NR	F	F	NR	NR	NR	NR
Dry-No Freeze	F	NR	NR	NR	NR	NR	NR	NR	NR

LEGEND:

$k_{subgrade}$ = Subgrade permeability.

R = Some form of subdrainage or other design features are recommended to combat potential moisture problems.

F = Providing subdrainage is feasible. The following additional factors need to be considered in the decision making:

(1) Past pavement performance and experience in similar conditions, if any.
(2) Cost differential and anticipated increase in service life through the use of various drainage alternatives.
(3) Anticipated durability and/or erodibility of paving materials.

NR = Subsurface drainage is not required in these situations.

Wet Climate = Annual precipitation > 508 mm (20 in.)

Dry Climate = Annual precipitation < 508 mm (20 in.)

Freeze = Annual freezing index > 83 ºC-days (150 ºF-days)

No-Freeze = Annual freezing index < 83 ºC-days (150 ºF-days)

Table 7-4. AASHTO definitions for pavement drainage recommended for use in both flexible and rigid pavement design (AASHTO, 1993).

Quality of Drainage	Water Removed* Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	Does not Drain

* Based on 50% time-to-drain.

As reviewed in Chapters 3, 5, and 6, drainage effects on pavement performance are incorporated into both the AASHTO 1993 and in the NCHRP 1-37A design methods. In AASHTO 1993, the effect of drainage is considered by modifying the structural layer coefficient (for flexible pavements) and the load transfer coefficient (for rigid pavements) as a function of the quality of drainage and the percent of time the pavement structure is near saturation. The influence of the drainage coefficient (C_d) for rigid pavement design and a drainage modifier (m) for flexible pavement design were demonstrated in the sensitivity studies shown in Chapter 6.

In the NCHRP 1-37A pavement design guide, the impact of moisture on the stiffness properties of unbound granular and subgrade materials is considered directly through the modeling of the interactions between climatic factors (rainfall and temperatures), groundwater fluctuations, and material characteristics of paving layers. Drainage coefficients are not used. However, the benefits of incorporating drainage layers are apparent in terms of distress predictions, which consider seasonal changes in unbound layers and subgrade properties due to moisture and coupled moisture-temperature effects.

Using either the AASHTO 1993 or NCHRP 1-37A method, the influence on design can be significant. For example, in high rainfall areas, the base section of a flexible pavement system (with a relatively thick base layer) can be reduced in thickness by as much as a factor of 2, or the design life extended by an equivalent amount, if excellent drainage is provided versus poor drainage. Likewise, an improvement in drainage leads to a reduction in Portland cement concrete (PCC) slab thickness.

Achieving poor drainage is relatively simple. If the subgrade is not free draining (e.g., not a clean sand or gravel), then the pavement section will require drainage features to drain. Even with edge drainage (*i.e.,* daylighted base or edgedrains), drainage could still be poor. Many designers choose to use dense graded base for its improved construction and presumed structural support over free-draining base. Unfortunately, dense graded base usually does not readily drain and, as a result, structural support will most likely decrease over time.

Due to the low permeability of dense graded base and long drainage path to the edge of the road, drainage in dense graded base is, at best, extremely slow. For example, consider that the permeability of a dense graded base with a very low percentage of fine-grain soil (less than 5% smaller than a 0.075 mm {No. 200 U.S. sieve}) is about 0.3 m/day (1ft/day)(as was reviewed in Chapter 5). Also consider that the length of the drainage path for a two-lane road (lane width of the road draining from the centerline to the edge) is typically 3.7 m (12 ft). An optimistic estimate of the time required to drain a base section that is 300 mm (1 ft) thick and has a slope of 0.02 is 2 days. According to AASHTO definitions of drainage, the pavement section has "good" to "fair" drainage. If the length of the drainage path is two lanes (*i.e.,* 7.3 m {24 ft}), it would take up to a week for the pavement to drain; a condition defined as "fair" drainage (AASHTO, 1993). Base materials often contain more than 5% fines, in which case the permeability and, correspondingly, the drainage can easily be an order of magnitude less than the estimated value for the example (AASHTO, 1993)[1]. In a recent study a Midwestern state found base materials from six different quarries to have 12% to 19% fines and corresponding field permeabilities measured at 2 to 0.01 m/day (7 to 0.03 ft/day) (Blanco et al., 2003). A month or more will then be estimated for pavement drainage; a condition defined as "poor" to "very poor" in AASHTO 1993. In reality, capillary effects and the absence of a driving head of water often cause dense graded base to act like a sponge at low hydraulic gradients. This results in trapped water in the pavement section and "very poor" drainage (*e.g.,* see Dawson and Hill, 1998).

In order to achieve good to excellent drainage, a more permeable, open-graded base and/or subbase will be required, which is tied into a subsurface drainage system. However, this approach only works for new or reconstructed pavements. For existing pavements, retrofitting drainage along the edges of the pavement is the only option, and the existing base material may not drain. However, a significant amount of water can enter the pavement at the crack between the shoulder and the pavement, as well as from lateral movement of water from outside the shoulder. Specific guidelines do not exist currently for retrofit pavements, as only limited data are available. Local experience should be used in selecting pavement candidates for retrofitting. Performance of similar retrofitted sections, if available, can be a valuable tool in the decision making process.

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[1] Based on hydraulic conductivity tests, AASHTO notes a decrease in permeability from 3 m/day (10 ft/day) with 0% fines down to 0.02 m/day (0.07 ft/day), with the addition of only 5% non-plastic fines and (0.0003 m/day (0.001 ft/day) with 10% non-plastic fines. An additional order of magnitude decrease was observed with base containing plastic fines.

7.2.4 Types of Subsurface Drainage

In the past, pavement systems were designed without any subdrainage system. These sections are commonly labeled "bathtub" or "trench" sections because infiltrated water is trapped in the base and subbase layers of the pavement system.

Many types of subsurface drainage have been developed over the years to remove moisture from the pavement system. These subsurface drainage systems can be classified into several groups. One criterion for classifying various subsurface drainage systems is the source of moisture that the system is designed to control. For example, a **groundwater control system** refers to a subsurface drainage system designed to remove and control the flow of groundwater. Similarly, an **infiltration control system** is designed to remove water that seeps into the pavement structural section. A **capillary break system** is designed to intercept and remove rising capillary water and vapor movement.

Probably the most common way to classify a subsurface drainage system is in terms of its location and geometry. Using this classification, subsurface drainage systems are typically divided into five distinct types:

- Longitudinal edgedrains.
- Transverse and horizontal drains.
- Permeable bases.
- Deep drains or underdrains.
- Interceptor drains.

Each type may be designed to control several sources of moisture and may perform several different functions. In addition, the different types of subsurface drainage system may be used in combination to address the specific needs of the pavement being designed. Drains constructed primarily to control groundwater general consist of underdrains and/or interceptor drains. The interceptor drains are usually placed outside the pavement system to intercept the lateral flow of water (*e.g.,* from cut slopes) and remove it before it enters the pavement section. Deep underdrains (greater than 1 m {3 ft} deep) are usually installed to lower the groundwater level in the vicinity of the pavement. The design and placement of these interceptor and underdrains should be addressed 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 edge of the pavement from above. Edgedrains are combined with permeable base and, in some cases, transverse and horizontal drains to form a drainable pavement system to control surface infiltration water. Drainable pavement systems generally consist of the following design features (as shown in Figure 7-2):

- a full-width permeable base under the AC- or PCC-surfaced travel lanes,
- a separator layer under the permeable base to prevent contamination from subgrade materials,
- longitudinal edgedrains with closely spaced outlets. An alternative to closely spaced outlets is dual drainage systems with parallel collector drains. An alternative to edgedrains is daylighting directly into a side ditch.

Designs not incorporating these combinations of features cannot be expected to function properly. Drainage systems for new construction and rehabilitation are described in more detail in the following sections.

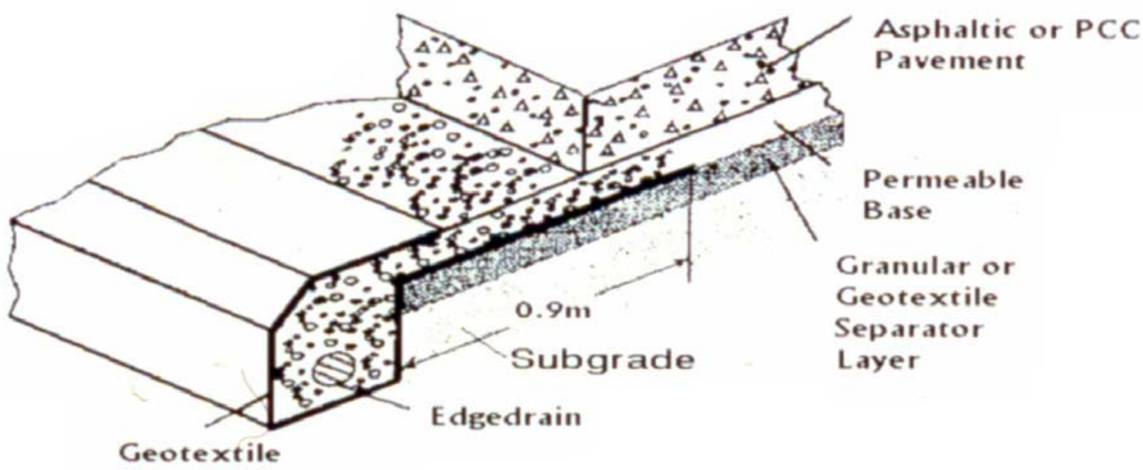


Figure 7-2. Design elements of a drainable pavement system (after FHWA, 1992).

7.2.5 Daylighted Base Sections

Daylighted bases were one of the first attempts to remove surface infiltration water from the pavement system. The original daylighted base consists of a dense-graded aggregate base that extends to the ditchline or side slope. Daylighted dense-graded bases are expected to intercept water that infiltrates through the pavement surface and drain the water through the base to the ditch. However, most dense-graded daylighted bases are slow draining and, therefore, not very effective in removing infiltrated water.

This situation led to the development of a new generation of daylighted bases—daylighted permeable bases (Fehsenfeld 1988), as illustrated in Figure 7-3. Several studies have reported that daylighted permeable bases are as effective in removing infiltrated water and reducing moisture-related distresses as permeable bases with edgedrains (Yu et al. 1998b). However, they require regular maintenance because the exposed edge of daylighted bases easily becomes clogged with fines, soil, vegetation, and other debris. Also, stormwater from ditch lines can easily backflow into the pavement structure. Further study into daylighted permeable bases is needed to verify long-term performance of this design.

7.2.6 Longitudinal Edgedrains

Longitudinal edgedrains consist of a drainage system that runs parallel to the traffic lane. The edgedrains collect water that infiltrates the pavement surface and drains water away from the pavement through outlets. Four basic types of edgedrains systems have been used:

- pipe edgedrains in an aggregate filled trench,
- pipe edgedrains with porous concrete (*i.e.,* cement treated permeable base) filled trench,
- prefabricated geocomposite edgedrains in a sand backfilled trench, and
- aggregate trench drain ("French" drain).

The most commonly used edgedrain is a perforated pipe varying in diameter from 100 – 150 mm (4 – 6 in.). The pipe is generally situated in an aggregate trench to allow water to flow toward the pipe. Another type of edgedrain that is often used in rehabilitation projects is a geocomposite drain in a sand filled trench with pipe outlets. Typical cross sections of edgedrains are illustrated in Figures 7-5 and 7-6.

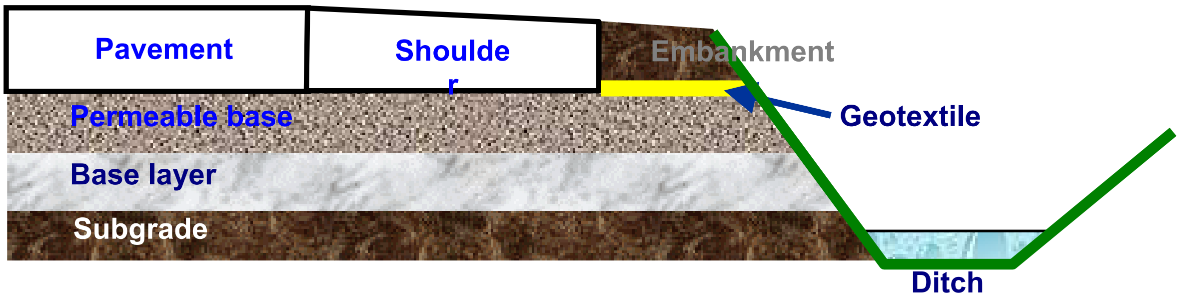


Figure 7-3. Typical AC pavement with a daylighted base.

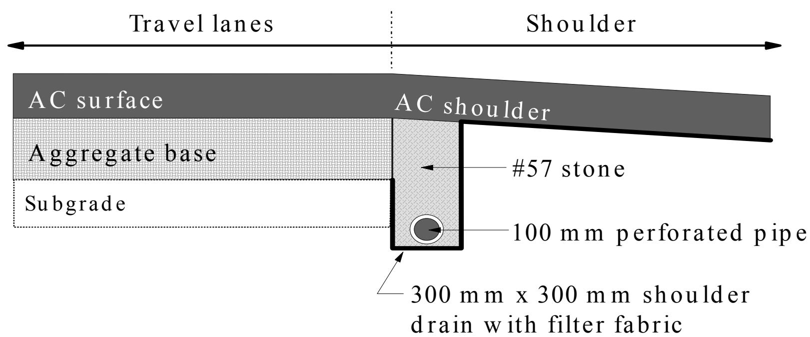


Figure 7-4. Typical AC pavement with pipe edgedrains (ERES, 1999).

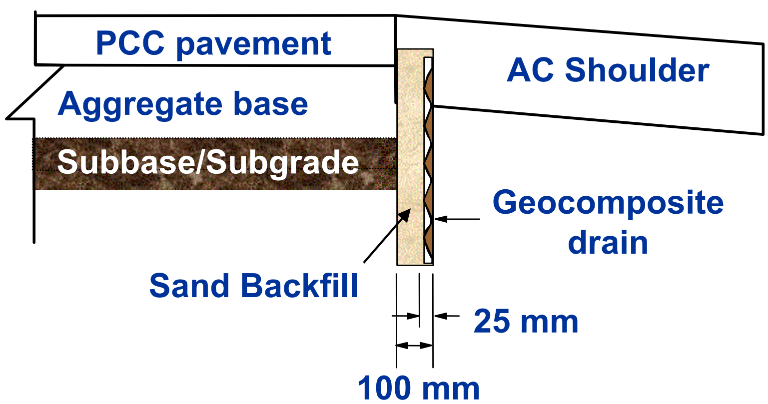


Figure 7-5. Typical PCC pavement with geocomposite edgedrains (ERES, 1999).

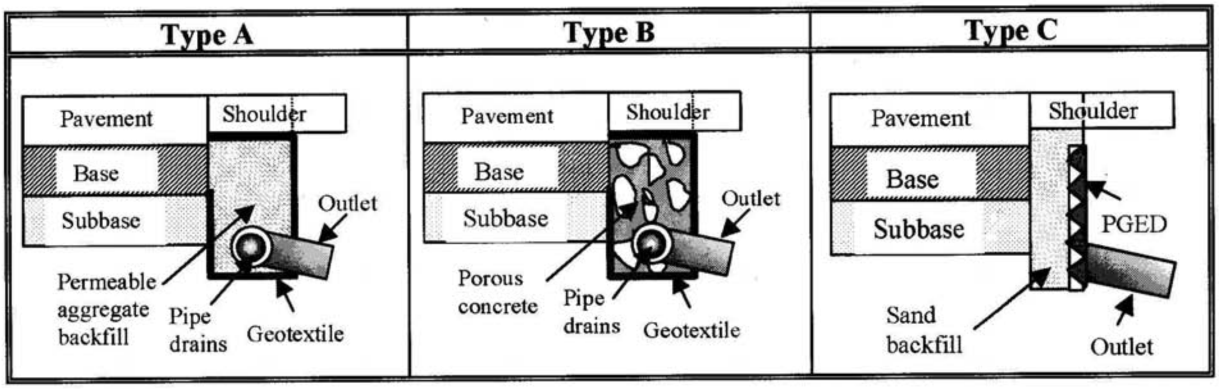


Figure 7-6. Typical edgedrains for rehabilitation projects (NCHRP 1-37A).

The effectiveness of longitudinal edgedrains depends on how they are used. Longitudinal edgedrains can be effective if used with other drainage features. Typical application of edgedrains include the following:

- New construction
    - Longitudinal edgedrains (pipe or geocomposite) with nonerodible dense-graded bases*.
    - Longitudinal edgedrains (pipe or geocomposite) with permeable bases.
- Existing pavement
    - Retrofit longitudinal edgedrains (pipe or geocomposite) with nonerodible dense-graded bases*.
- Rehabilitation projects
    - Retrofit longitudinal edgedrains (pipe or geocomposite) with nonerodible dense-graded bases*. (On projects using rubblized base or dense graded base with erodible fines, the geotextile filter in the trench should not be placed between the base and the edgedrain aggregate to avoid clogging the geotextile filter - see Figure 7-6.)

* *Retrofit edgedrains are usually not recommended for pavements with dense-graded aggregate bases containing more than 15% fines (fraction passing the 0.075 mm (No. 200 sieve)). Excessive fines can clog the drains, and the loss of fines through the pipes can lead to significant base erosion. As previously indicated, dense graded base with greater than 5% fines is not anticipated to freely drain, however edgedrains can be used to effectively remove water entering the longitudinal joint (or crack) between the pavement and the shoulder.*

The field performance of edgedrains installed without a permeable base has been mixed. Some studies show little or no benefit, but others report significant improvement in pavement performance. Considering that only about 30% of all edgedrains in-service are functioning properly, mostly due to improper construction (Daleiden 1998; Sawyer 1995), the mixed report is not surprising. In many cases, the outlet pipes are crushed during construction or clogged due to inadequate maintenance. The performance of edgedrains placed in untreated dense-graded base sections seems to be dependent to a significant degree on local climatic conditions, natural drainage characteristics, subgrade type, pavement design, construction and construction inspection, and maintenance. Longitudinal edgedrains with permeable bases have been found to be effective in draining pavements and reducing moisture-related distresses when well designed, constructed, and maintained.

The type of geocomposite edgedrains used also affects performance. Older versions did not have sufficient hydraulic capacity and had not been recommended for draining permeable bases. However, some of the geocomposites available today do provide sufficient hydraulic capacity to drain permeable bases. The main disadvantage of geocomposite edgedrains is that they are difficult to maintain.

The use of aggregate trench drains, however, is not recommended because of low hydraulic capacity and inability to be maintained. An exception might be permeable cement stabilized aggregate placed in a trench.

The size of the longitudinal perforated pipe in the edgedrain is often based on maintenance requirements for cleaning capabilities and reasonable distance between outlets. Maintenance personnel should be consulted before finalizing these dimensions. The smallest diameter suitable for cleaning is 75 mm (3 in.), however many state highway agencies and the FHWA suggest a minimum pipe size of 100 mm (4 in.) based on maintenance considerations (FHWA, 1992). FHWA also recommends a maximum outlet spacing of 75 m (250 ft).

One of the most critical items for edgedrains is the grade of the invert. Construction control of very flat grades is usually not possible, leaving ponding areas that result in subgrade weakening and premature failures. Although not a popular concept, it may be more economical to raise the pavement grade to develop adequate drain slopes for the subsurface drainage facilities (*e.g.,* Florida). To achieve a desirable drainage capacity, a minimum slope may be required for the edgedrain that is greater than the slope of the road. However, this requirement may not be practical, and the pipe will mostly be sloped the same as the roadway. It is suggested that rigorous maintenance be anticipated, especially when adequate slopes cannot be achieved (FHWA, 1992).

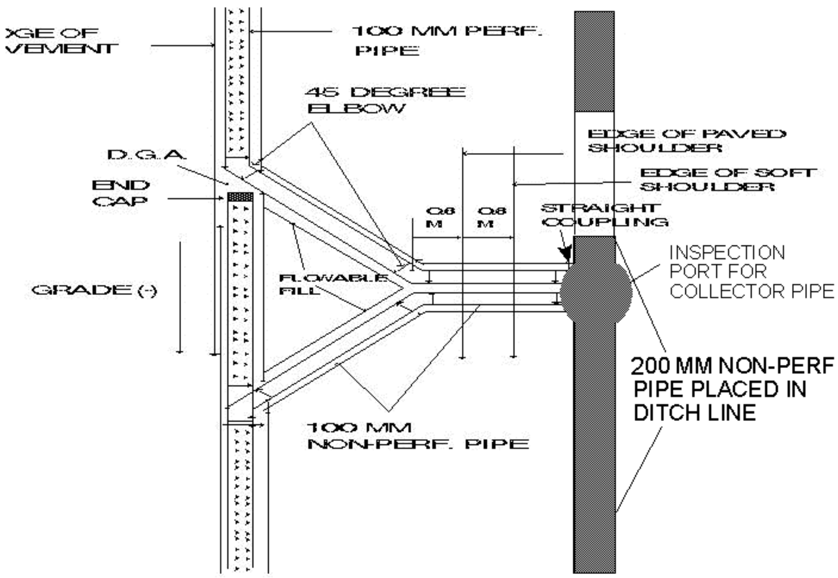
The ditch or storm drain pipe must be low and large enough to accept the inflow from the edgedrain without backup. FHWA recommends the outlet be at least 150 mm (6 in.) above the ten-year storm flow line of the ditch or structure. The outlet should also be at a location and elevation that will allow access for maintenance activities (both cleaning and repair). Outlets and shallow pipes should be located well away from areas of expected future surface maintenance activities, such as sign replacement and catch basin cleanout or repair. FHWA also recommends angled or radius outlet connections to facilitate clean out and video inspection. Outlet headwalls, typically precast concrete, are also an essential part of the edgedrain system to prevent displacement of the outlet pipe and crushing during roadway and ditchline maintenance operations. Locations of guardrail, sign, signal, and light posts must be adjusted to prevent damage to the subsurface drainage facilities.

An offset dual pipe with a large diameter parallel collector drain line is an alternative to decrease the number of outlets (see Figure 7-7). The large diameter collector pipe (either heavy walled plastic or concrete) runs either adjacent to or below a perforated drainage pipe, as shown in Figure 7-7, to facilitate quick removal of subsurface water. The collector pipe can outflow into culverts or stormwater systems. Manholes can be installed for cleanout. These systems are quite common in Europe and have been used by a few U.S. agencies to reduce outlet maintenance issues (*e.g.,* California and, experimentally, in Kentucky).

7.2.7 Permeable Bases

A permeable base is designed to rapidly move surface infiltration water from the pavement structure to the side ditch through longitudinal edgedrains with outlets or by daylighting directly into the side ditch. Permeable bases contain no fines (0% passing the 0.075-mm (No. 200) sieve) to allow easy flow of water. In order to meet excellent drainage requirements (*i.e.,* time-to-drain of less than 2 hours from Table 7-4), permeable bases typically are required to have permeability values in excess of 300 m/day (1000 ft/day) and thicknesses of 100 mm (4 in.) (as recommended by FHWA, 1992). The performance of permeable base layers meeting these requirements will be demonstrated later in Section 7.2.12 on design of pavement drainage.

The structural capacity of angular, crushed aggregate permeable base, with a percentage of two-face crushing, is usually equivalent to the structural capacity of an equal thickness of dense-graded base. However, in order to meet these hydraulic requirements, a coarse uniform gravel must be used, which is often difficult to construct. Asphalt or cement treatments are often used to stabilize the gravel for construction, as discussed in Section 7.3. While stabilizing the base with a cement or asphalt binder will initially offer greater structural support than dense-graded base, it should be remembered that the primary purpose of the stabilizer is to provide stability of the permeable base during the construction phase. It is generally assumed that the binder will either break down or be removed by stripping with time. Thus, increase in structural support is generally not assumed for stabilized aggregate.



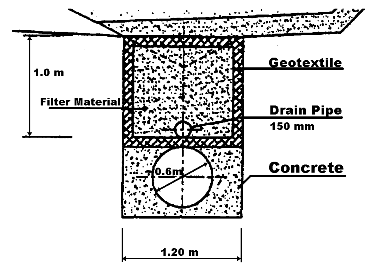


Figure 7-7. Dual pipe edgedrain systems showing alternate locations of the parallel collector pipe, either adjacent to or beneath the drain line (Christopher, 2000).

Typical cross-sections of AC and PCC pavements with permeable bases were illustrated in Figures 7-4, 7-5, and 7-6. Note that a geotextile filter should be wrapped around a portion of the trench, but not over the interface between the permeable base and drainage aggregate.

7.2.8 Dense-Graded Stabilized Base with Permeable Shoulders

This system consists of a nonerodible dense-graded base, typically lean concrete base (LCB) or asphalt treated base (ATB), under the traffic lanes and a permeable base under the shoulder. Longitudinal edgedrains are placed in the permeable base course to carry the excess moisture from the pavement structure. The recommended design for a dense-graded stabilized base with permeable shoulders is illustrated in Figure 7-8. This design offers better support under the traffic lanes where it is needed most, while still providing a means to remove water from the pavement structure. This design is now required for all high-type PCC pavements (pavements designed for more than 2.5 million equivalent single axle loads {ESALs}) in California (CALTRANS 1995).

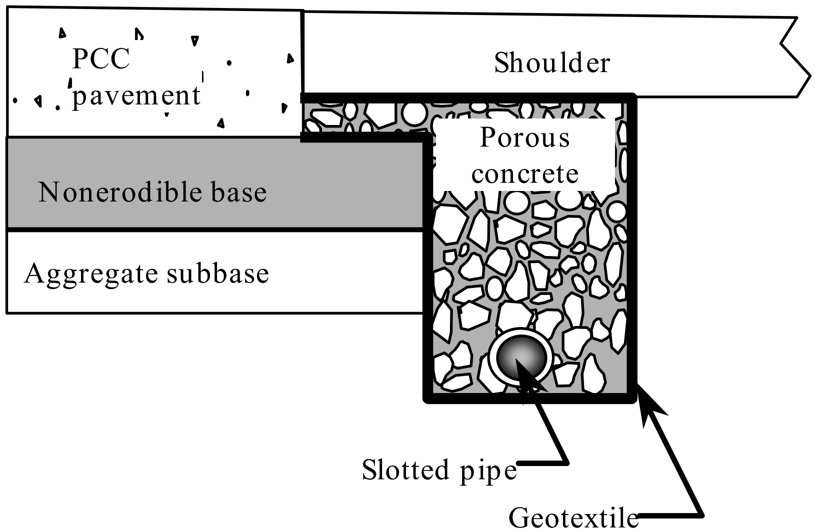


Figure 7-8. Recommended design of PCC pavement with a nonerodible dense-graded base and permeable shoulders (ERES, 1999).

7.2.9 Horizontal Geocomposite Drains

Several states (*i.e.,* Maine, Wisconsin, and Virginia) have experimented with the use of horizontal geocomposite drains, with properties sufficient to handle the estimated flow and support traffic loads, placed either below or above dense graded base, placed as a drainage layer beneath full depth asphalt, or placed between a crack and seat concrete surface and a new asphalt layer. When placed below the base aggregate, the geocomposite shortens the drainage path and reduces the time-to-drain. When placed directly beneath the pavement surface, the geocomposite intercepts and removes infiltration water before it enters the base and/or subgrade. The geocomposite is tied into an edgedrain system. Systems using this technology have been found to have excellent drainage (*i.e.,* time-to-drain of less than 2 hours from Table 7-4). Additional information, including preliminary performance information, is reported by Christopher et al. (2002).

7.2.10 Separator Layers

Separator layers play an essential role in the performance of a pavement with a permeable base by preventing fines in the underlying layers and subgrade soils from infiltrating into the permeable base, thus maintaining the permeability and effective thickness of the base course. Various combinations of materials have been used as separator layers, including the following (FHWA 1994a):

- Dense-graded aggregate (most used by far)
- Geotextiles
- Cement-treated granular material
- Asphalt chip seals
- Dense-graded asphalt concrete

These materials have been used with varying degrees of success. Lime- or cement-treated subgrades alone are not acceptable as separator layers over fine-grained soils. There have been some classic failures of lime-treated soils used as separator layers in which pumping into the permeable base caused excessive settlements.

According to a survey of 33 states, 27 used dense-graded aggregates or asphalt-treated mixtures as separator layers on a regular basis. Sixteen states used geotextiles sparingly, and 11 states used either dense-graded material or geotextiles as separator layers (Yu et al., 1998). Generally, a dense-graded aggregate or a dense-graded AC material separator layer is preferred over a geotextile for competent subgrades because the aggregate layer will provide a strong construction platform and distribute traffic loads to the subgrade. However, geotextile separator layers have been used directly beneath base layers where the additional support of a subbase is not required. For sensitive subgrades that are easily disturbed by construction (*e.g.,* silts and saturated cohesive soils), a geotextile separator layer used in conjunction with a granular subbase minimizes disturbance and provides a good construction platform. Geotextile separators also allow the use of a more open-graded, freer-draining subbase, reducing the potential for subbase saturation. Geotextiles can also be used as a separator layer in conjunction with compacted or treated subgrades, or granular subbases. If appropriately design, geosynthetics can also be used to increase subgrade support, as reviewed later in Section 7.6.5.

7.2.11 Performance of Subsurface Drainage

Many studies have shown the benefits of subsurface drainage in terms of improved performance. Cedergren (1988) believes that all important pavements should have internal drainage, claiming drainage eliminates damage, increases the life of the pavement, and is cost-effective.

Moisture-related damage to pavements has become more significant as traffic loadings have increased over the past 40 years. The annual rate of ESAL applications has virtually doubled every 10 years, causing tremendous problems related to moisture accelerated damage. A pavement may be adequately drained for one level of traffic, but as traffic increases, moisture-related damage may increase greatly. As a result, more and more states have begun to employ subsurface drainage systems (Yu et al. 1998b). Many preliminary studies indicate that drainage systems are indeed beneficial in terms of reducing certain types of pavement deterioration. However, due to some instances of poor design, construction, and/or maintenance, all have not performed as well as expected.

One example of unsatisfactory performance is some early cracking observed on a few PCC pavements with permeable bases. This occurs for a variety of reasons, including:

- Inadequate design of permeable bases and separator layers.
- Inadequate edgedrains.
- Lack of quality control during construction, such as inadequate joint sawing. Sometimes the concrete from the slab enters the permeable base, creating a thicker slab than was originally designed. Joints must be sawed deeper to ensure the proper depth is obtained to cause cracking through the joint.
- Lack of maintenance of the drainage system after the highway is open to traffic.
- Possible settlement of the PCC slab over untreated aggregate permeable bases.

Permeable bases must be constructed of durable, crushed aggregate to provide good stability through aggregate interlock. They must have a separator layer capable of preventing the pumping of fines into the permeable base from underlying layers and from preventing any intermixing of the permeable base and separator layer. Permeable bases must also have pipe edgedrains to drain the infiltrated water with suitable outlets at reasonable outlet spacing or must be daylighted directly into the ditch. Finally, to ensure good performance, the drainage system must be regularly maintained.

7.2.12 Design of Pavement Drainage

Design of pavement drainage consists of determining:

1. The hydraulic requirements for the permeable layer to achieve the required time-to-drain.
2. The edgedrain pipe size and outlet spacing requirements.
3. Either the gradation of requirements for a graded aggregate separation layer or the opening size, permeability, endurance, and strength requirements for geotextile separators.
4. The opening size, permeability, endurance, and strength requirements for geotextile filters, or the gradation of the granular filters (to be used in the edgedrain).

The following provides an outline of the design steps and procedures required for the design of each of these subsurface drainage components. Complete design details and supporting information can be found in NHI 13126 on Pavement Subsurface Drainage Design – Reference Manual (ERES, 1999).

7.2.13 Hydraulic Requirements for the Permeable Layer(s)

Basically there are two approaches to the hydraulic design of a permeable layer:

1. Time-to-drain
2. Steady-state flow.

The time-to-drain approach was previously discussed in Section 7.2.3 and simply means the time required for a percentage of the free water (*e.g.,* 50%) to drain, following a moisture event where the pavement section becomes saturated. In the steady-state flow approach, uniform flow conditions are assumed, and the permeable layer is designed to drain the water that infiltrates the pavement surface. The time-to-drain approach will be the basis for design in this manual, as it is currently the procedure recommended by the FHWA, AASHTO, and NCHRP 1-37A for pavement design. Elements of steady state flow will be used to determine outlet spacing. (For additional discussion of steady state flow methods see FHWA, 1992 and ERES, 1999.)

The time-to-drain approach assumes the flow of water into the pavement section until it becomes saturated (the drainage layer plus the material above the drainage layer). Excess precipitation will not enter the pavement section after it is saturated; this water will simply run off the pavement surface. After the rainfall event, the drainage layer will drain to the edgedrain system. Engineers must design the permeable layer to drain relatively quickly to prevent the pavement from being damaged.

A time-to-drain of 50% of the drainable water in 1 hour is recommended as a criterion for the highest class roads with the greatest amount of traffic (FHWA, 1992). For most other high use roadways, a time-to-drain of 50% of the drainable water in 2 hours is recommended. For secondary roads, a minimum target value of 1 day is recommended (U.S. Army Corps of Engineers, 1992). Remember, in all cases, the goal of drainage is to remove all drainable water as quickly as possible.

The time-to-drain is determined by the following equation:

$$t = T \times m \times 24 \qquad \textit{Eq. 7.1}$$

where, t = time-to-drain in hours
T = Time Factor
m = "m" factor (see Eq. 7.3)

A simplified design chart for determining a time-to-drain of 50% time factor, T_{50}, is provided in Figure 7-9. This chart was developed for one degree (*i.e.,* direction) of drainage and is adequate for most designs. For expanded charts to cover additional degrees of drainage and desired percent drained see FHWA, 1992 and ERES, 1999.

The time factor is based on the geometry of the drainage layer (e.g., the permeable base layer). The geometry includes the resultant slope (S_R) and length (L_R); the thickness of the drainage layer (H), which is the length the water must travel within a given layer; and, the percent drained (U), (i.e., 50%). S_R and L_R are based on the true length of drainage and are determined by finding the resultant of the cross and longitudinal pavement slopes (S_X and S, respectively) and lengths (L_x and L, respectively). The resultant length is measured from the highest point in the pavement cross-section to the point where drainage occurs (*i.e.,* edgedrain or daylighted section). First, the slope factor (S_l) must be calculated:

$$S_1 = \frac{L_R\, S_R}{H} \qquad \textit{Eq. 7.2}$$

where, S_R	=	$(S^2 + S_X^2)^{1/2}$
L_R	=	$W\,[1 + (S/S_X)^2]^{1/2}$
W	=	width of permeable layer in m (ft)
H	=	thickness of permeable layer in m (ft)
1 ft	=	0.3 m

Figure 7-9 is then entered with the S_l, and the resulting T_{50} to be used in Eq. 7.1 is determined.

The "m" factor in Eq 7.1 is determined by the equation:

$$m = \frac{N_o\, L_R^{\,2}}{kH} = \frac{N_o\, L_R^{\,2}}{\psi} \qquad \textit{Eq.}7.3$$

where, N_o	=	the effective porosity of the drainage layer
k	=	permeability of drainage layer in m/day (ft/day)
H	=	thickness of drainage layer in m (ft)
ψ	=	the transmissivitty of the drainage layer in m^2/day (ft^2/day)
1 ft	=	0.3 m

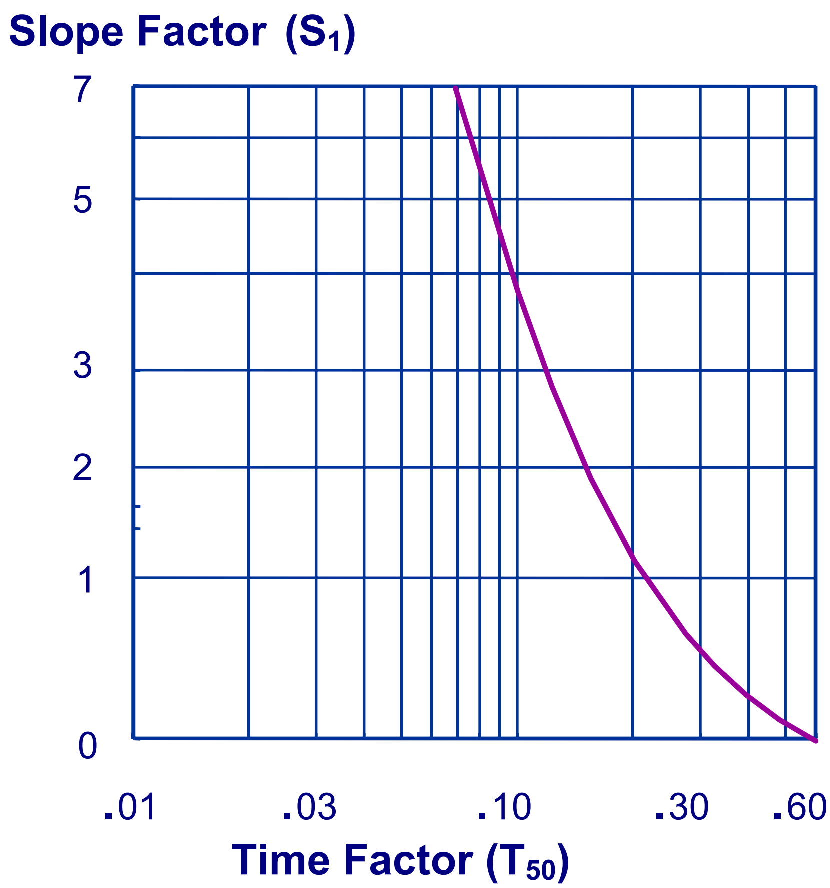


Figure 7-9. Time Factor for 50% Drainage (ERES, 1999).

The intrinsic factors that represent the drainage capabilities of drainage layer base are represented by the effective porosity (N_o) and the coefficient of permeability (k) or, if H is known, the transmissivity of the drainage layer. The effective porosity is the ratio of the volume of water that can drain under gravity from the material to the total volume of the material. It is a measure of the amount of water that can be drained from a material. The value can be easily determined by saturating a sample of material and measuring the amount of water that drains. Additional information on the determination of these characteristics for aggregate drainage layers are covered in detail in FHWA, 1992 and NHI 13126.

For example, using the recommended 4-inch-thick open-graded base layer with a permeability of 300 m/day (1000 ft/day) at a cross slope of 2% in a relatively flat (1% grade) road alignment would produce the following time-to-drain for a four lane road draining from the center (W = 7.3 m (24 ft)):

$$S_R = (S^2 + S_X^2)^{1/2} = (0.01^2 + 0.02^2)^{1/2} = 0.022$$

$$L_R = W((1 + (S/S_X)^2)^{1/2} = 24 \text{ ft } [1 + (0.01/0.02)^2]^{1/2} = 26.8 \text{ ft}$$

$$S_l = (L_R\, S_R)/H = (26.8 \text{ ft} \times 0.022)/\ 0.33 \text{ ft} = 1.8$$

$$m = (N_e L_R^2) / (kH) = (0.25 \times (26.8 \text{ ft})^2)/(1000 \text{ ft/day} \times 0.33 \text{ ft}) = 0.54 \text{ days}$$

From Figure 7-9 with $S_l = 1.8$, $T = 0.16$

Therefore, $t = T \times m \times 24 = 0.16 \times 0.54$ days $\times$ 24 hrs/day = 2.1 hrs

Since the time-to-drain is close to 2 hrs, the drainage layer would provide excellent drainage, as defined in Table 7-4.

According to a sensitivity analysis on time-to-drain performed in ERES, 1999, time-to-drain is most sensitive to changes in the coefficient of permeability and the resultant slope, decreasing exponentially with increasing permeability and slope values. Time to drain increases linearly with increasing length and effective porosity, while thickness has very little effect.

The DRIP microcomputer program developed by FHWA can be used to rapidly evaluate the effectiveness of the drainage system and calculate the design requirements for the permeable base design, separator, and edgedrain design, including filtration requirements. The program can also be used to determine the drainage path length based on pavement cross and longitudinal slopes, lane widths, edgedrain trench widths (if applicable), and cross-section geometry crowned or superelevated. The software can be downloaded directly from the FHWA WEB page http://www.fhwa.dot.gov/pavement/library.htm and is included with the NCHRP 1-37A pavement design software.

7.2.14 Edgedrain Pipe Size and Outlet Spacing Requirements

The FHWA recommends a minimum pipe diameter of 100 mm (4 in.) and a maximum outlet spacing of 75 m (250 ft) to facilitate cleaning and video inspection. The adequacy of these requirements can be confirmed by evaluating the anticipated infiltration rate or, more conservatively, from the maximum flow capacity of the drainage layer.

With the flow capacity method, the estimated discharge rate from drainage layer is determined. For example, the conventional 100-mm (4-in.) thick open-graded base layer with a permeability of 300 m/day (1000 ft/day) used in the previous time-to-drain example provides excellent drainage for most conditions (FHWA, 1992). This 100-mm (4-in.) thick free-draining base layer has a transmissivity (*i.e.,* permeability multiplied by the thickness) of about 28 m^2/day (300 ft^2/day). For a typical roadway gradient of 0.02 (for a 2% grade), the open-graded base layer has a flow capacity of 0.13 ft^3/day (6 ft^3/day) per ft length of road. Thus at an outlet spacing of 75 m (250 ft), the quantity of flow at the discharge (Q) of the edgedrain system would be 33 m^3/day (1500 ft^3/day).

The capacity of a circular pipe flowing full can be determined by Manning's equation:

$$Q = \frac{53.01}{n} D^{8/3} S^{1/2} \qquad Eq. 7.4$$

where,	Q	= Pipe capacity, cu ft/day
	D	= Pipe diameter, in.
	S	= Slope, ft/ft
	n	= Manning's roughness coefficient
		= 0.012 for smooth pipe
		= 0.024 for corrugated pipe
	1 ft	= 0.3 m
	1 in	= 25.4 mm

Thus, for a 100-mm (4-in.) smooth wall pipe at a 1% grade, the flow capacity is 504 m^3/day (17800 ft^3/day), which is more than adequate to handle the maximum quantity of flow anticipated for the edgedrain system. However, the 100-mm (4-in.) pipe is still recommended to facilitate inspection and cleaning.

In the infiltration method, a design rainfall and an infiltration ratio are selected. Pavement infiltration is determined by the equation

$$q_i = C \times R \times 1/12 \text{ (ft/in)} \times 24 \text{ (hr/day)} \times 1 \text{ ft} \times 1 \text{ ft)} \qquad Eq. 7.5$$

which can be simplified to:

$$q_i = 2\,C\,R \qquad Eq. 7.5a$$

where,	q_i	= Pavement infiltration, ft^3/day/ft^2 of pavement
	C	= Infiltration ratio
	R	= Rainfall rate, in./hr

The infiltration ratio C represents the portion of rainfall that enters the pavement through joints and cracks. The following design guidance for selecting the infiltration coefficient is suggested (FHWA, 1992):

Asphalt concrete pavements	0.33 to 0.50
Portland cement concrete pavements	0.50 to 0.67

To simplify the analysis and provide an adequate design, FHWA suggest using a value of 0.5. The design storm whose frequency and duration will provide an adequate design must be selected. A design storm of 2-year frequency, 1-hour duration, is suggested. Figure 7-10 provides a map of generalized rainfall intensity.

The analysis is then performed by substituting into the above equation for the specific region of the country. The drainage layer discharge rate q_d can then be determined by multiplying the infiltration rate by the resultant length of the pavement section L_R as follows:

$$q_d = q_i L_R \qquad \textit{Eq. 7.6}$$

This discharge rate can then be compared to the flow capacity of the drainage layer and the lower value of the two used to evaluate the outlet spacing and pipe size.

7.2.15 Separator Layer

As indicated in the previous section, the separator consists of a layer of aggregate material (treated or untreated) or a geotextile layer placed between the permeable base and the subgrade or other underlying layers. The separator layer has to maintain separation between permeable base and subgrade, and prevent them from intermixing and support construction traffic. It may also be desirable to use a low permeable layer that will deflect water from the permeable base horizontally toward the pavement edge (NCHRP 1-37A).

If dense-graded aggregate separator layers are used, the aggregate must be a hard, durable material. Based on FHWA guide specifications for materials selection and construction of aggregate separation layers, the aggregate should meet the following requirements:

- The aggregate should have at least two fractured faces, as determined by the material retained on the 4.74 mm (No. 4) sieve; preferably, it should consist of 98% crushed stone.
- The L.A. abrasion wear should not exceed 50%, as determined by AASHTO T 96, *Resistance to Abrasion of Small Size Coarse Aggregate by Use of Los Angeles Machine*.

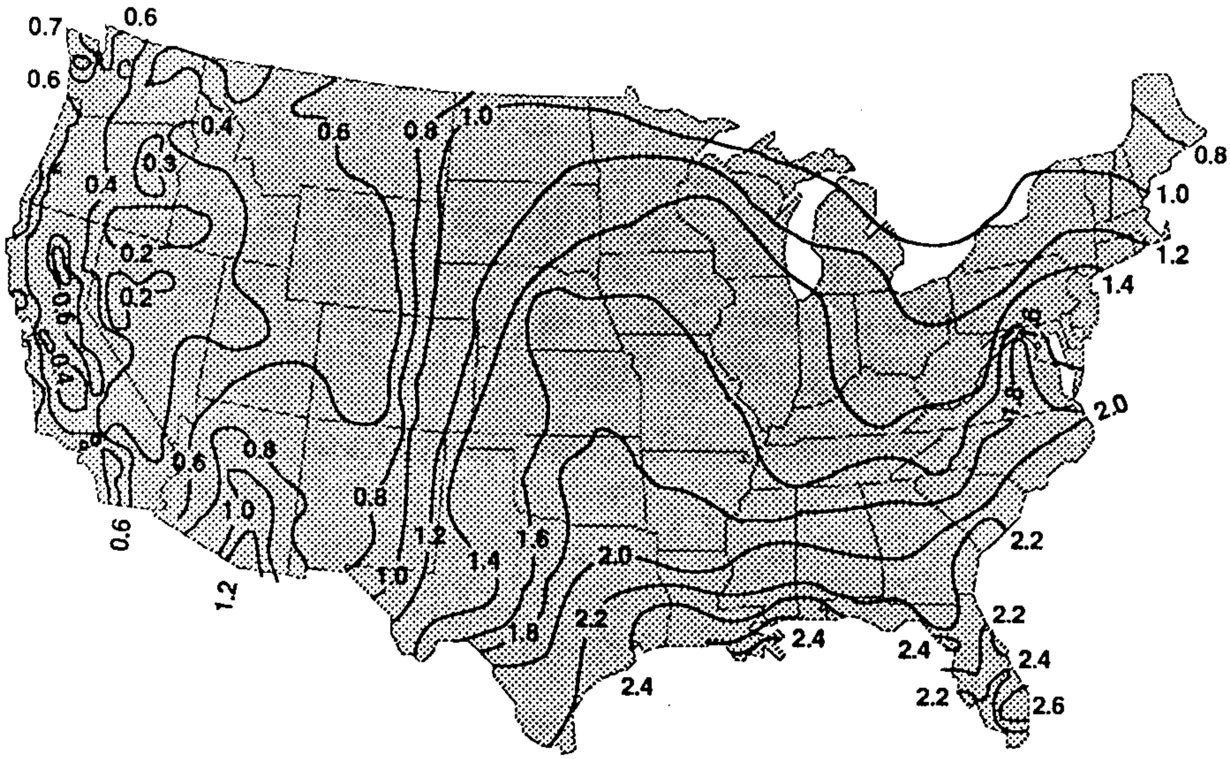


Figure 7-10. Rainfall Intensity in in./hr for a 2-year, 1-hour Storm Event (FHWA, 1992).

- The soundness loss percent should not exceed 12 or 18%, as determined by the sodium sulfate or magnesium sulfate tests, respectively. The test shall be in accordance with AASHTO T 104, *Soundness of Aggregate by the Use of Sodium Sulfate or Magnesium Sulfate*.
- The gradation of this layer should be such that it allows a maximum permeability of approximately 5 m/day (15 ft/day) with less than 12% of the material passing the 0.075 mm (No. 200) sieve, by weight.
- Material passing the 425 mm (No. 40) sieve shall be nonplastic, in accordance with AASHTO T 90, "Determining the Plastic Limit and Plasticity Index of Soils."

7.2.16 Geotextile Separator and Filter Design

As a separator, just as with the granular layer, the geotextile must prevent the intermixing of the permeable base and the adjacent subgrade or subbase layer. Also as with aggregate separator layers, the geotextile layers will have to satisfy filtration criteria. In order to provide support for construction traffic, the geotextile must also satisfy survivability and endurance criteria. Additional requirements for subgrade improvement are reviewed in Section 7.6.5. Both woven and non-woven geotextiles have been used for the separation application. The criteria for filtration and survivability are outlined in the following paragraphs and are basically the same as that required for the edgedrain geotextile filters. The only notable exception is that the separation layer can have a much lower permeability (compatible with the subgrade) than the edgedrain filter (compatible with the permeable base).

As a filter for the edgedrain, the geotextile must be designed to allow unimpeded flow of water into edgedrain system over the life of the system. The geotextile must prevent soil from washing into the system without clogging over time. The FHWA presents three basic principles for geotextile design and selection (Holtz et al., 1998):

1. If the larger pores in the geotextile filter are smaller than the largest particles of soil, these particles will not pass the filter. As with graded granular filters, the larger particles of soil form a filter bridge on the geotextile, which, in turn, filters the smaller particles of the soil. Thus, the soil is retained and particle movement and piping is prevented.
2. If the smaller openings in the geotextile are sufficiently large so that the smaller particles of soil are able to pass through the filter, then the geotextile will not clog.
3. A large number of openings should be present in the geotextile so that proper flow can be maintained even if some of the openings later become clogged.

The geotextile filtration characteristics must be checked for compatibility with the gradation and permeability of the subgrade. The requirements for proper performance can be appropriately selected by using the following design steps.

Step 1. Determine the gradation of the material to be separated/filtered. The filtered material is directly above and below the geocomposite drainage layer. Determine D_{85}, D_{15} and percent finer than a 0.075 mm (No. 200) sieve.

Step 2. Determine the permeability of the base or subbase $k_{base/subbase}$, whichever is located directly above the geocomposite drainage layer. (For placement directly beneath the hot-mix or PCC pavement applications, the default permittivity requirement will be used.

Step 3. Apply design criteria to determine apparent open size (AOS), permeability (k), and permittivity (ψ) requirements for the geotextile (after Holtz et al., 1998)

$AOS \leq D_{85\ base/subbase}$ (For woven geotextile)

$AOS \leq 1.8\ D_{85\ subgrade}$ (For nonwoven geotextile)*

$k_{geotextile} \geq k_{base/subbase}$
$\psi \geq 0.1\ sec^{-1}$

* *For noncohesive silts and other highly pumping susceptible soils, a filter bridge may not develop, especially considering the potential for dynamic, pulsating flow. A conservative (smaller) AOS ≤ $D_{85\ subgrade}$ is advised, and laboratory filtration tests are recommended.*

Step 4. In order to perform effectively, the geotextile must also survive the installation process. AASHTO M288 (1997) provides the criteria for geotextile strength required to survive construction of roads, as shown in Table 7-5. Use Class 2 where a moderate level of survivability is required (*i.e.,* for subgrade CBR > 3, where at least 150 mm (6 in.)) of base/subbase and normal weight construction equipment is anticipated, and where filters are used in edgedrains). Class 1 geotextiles are recommended for CBR < 3 and when heavy construction equipment is anticipated. For separation layers, a minimum of 150 mm (6 in.) of base/subbase should be maintained between the wheel and geotextile at all times.

In projects using recycled concrete, rubblizing, or crack-and-seat techniques, geotextiles and granular filters are susceptible to clogging by precipitate and should not be indiscriminately used to separate the permeable base from the drain or wrapped around pipes. Geotextiles should not be placed between the recycled material and the drain, but could be placed beneath and on the outside of the drain to prevent infiltration of the subgrade and subbase layers (see Figure 7-2.)

Table 7-5. Geotextile survivability requirements (AASHTO M 288-96).

Test	Test Method	Units	Geotextile Class			
			Class 1		Class 2	
			< 50%*	≥ 50%*	< 50%*	≥ 50%*
Grab Strength	ASTM D 4632	N	1400	900	1100	700
Seam Strength	ASTM D 4632	N	1200	810	990	630
Tear Strength	ASTM D 4533	N	500	350	400	250
Puncture Strength	ASTM D 4833	N	500	350	400	250
Burst Strength	ASTM D 3786	kPa	3500	1700	2700	1300

*Note: Elongation measured in accordance with ASTM D 4632 with < 50% typical of woven geotextiles and ≥ 50% typical of nonwoven geotextiles. (1 N = 0.22 lbs, 1 kPa = 0.145 psi)

7.3 BASE LAYERS: REQUIREMENTS, STABLILIZATION & REINFORCEMENT

The function of the base course varies according to the type of pavement, as was described in Chapter 1. Under rigid pavements, the base course is used to: (1) provide uniform and stable support, (2) minimize damaging effects of frost action, (3) provide drainage, (4) prevent pumping of fine-grained soils at joints, (5) prevent volume change of the subgrade, (5) increase structural capacity of the pavement, and (6) expedite construction. Under flexible pavements, the prime function of the base course is to structurally improve the load-supporting capacity of the pavement by providing added stiffness and resistance to fatigue, as well as to provide a relatively thick layer to distribute the load through a finite thickness of pavement. The base may also provide drainage and give added protection against frost action where necessary.

To meet these functional requirements, the base course as a minimum should have the following characteristics:

- To prevent pumping, a base course must be either free draining or it must be highly resistant to the erosive action of water. Erodibility is covered in more detail in the next section.
- To provide drainage, the base course may or may not be a well-graded material, but it should contain little or no materials finer than a 0.075 mm (No. 200) sieve. It may sometimes be stabilized with asphalt or cement.
- A base course design for frost action should be non-frost susceptible and free draining.
- To improve resistance to deformation and improve structural support or reduce the thickness, it may be desirable to stabilize the base course with asphalt or cement, as reviewed in Section 7.3.2 and 7.3.3, or to reinforce it with geosynthetics, as reviewed in Section 7.3.4.
- A base course need not be free draining to provide structural capacity, but it should be well-graded and should resist deformation due to loading.

The aggregate used for base must be hard, durable material. As a minimum, the aggregate should meet the following requirements:

- The aggregate should have at least two fractured faces; preferably, it should consist of 98% crushed stone.
- The L.A. abrasion wear should not exceed 45% as determined by AASHTO T 96, *Resistance to Abrasion of Small Size Coarse Aggregate by Use of Los Angeles Machine*.

- The soundness loss percent should not exceed 12 or 18%, as determined by the sodium sulfate or magnesium sulfate tests, respectively. The test should be performed in accordance with AASHTO T 104, *Soundness of Aggregate by the Use of Sodium Sulfate or Magnesium Sulfate* (see Chapter 5).
- For permeable base, the gradation of this layer should enable free movement of water with a minimum permeability value around 300 m/day (1,000 ft/day) (see Section 7.2) and the material passing the 0.425 mm (No. 40) sieve should be non-plastic in accordance with AASHTO T 90, *Determining the Plastic Limit and Plasticity Index of Soils*.

7.3.1 Erodibility of Bases

Preventing significant erosion of the base and subbase materials is very important for the control of moisture-related distresses, such as pumping and faulting in JPCP and punchouts in CRCP, as discussed in NCHRP 1-37A. Erodibility is the loss of base material due to hydraulic action, most often at the joints in rigid pavements, but also along the edge of both rigid and flexible pavements. The condition is related to the durability of the base in relation to its potential to break down under dynamic traffic loads, climatic conditions, environmental effects, as well as water action. As truck traffic increases, a more erosion resistant base is required, along with more adequate joint load transfer design (*e.g.,* use of dowels in joints). Traffic level is a very critical factor in the consideration of base/subbase course erosion, especially considering that the base/subbase under PCC slabs of reconstructed projects will likely receive 10 to 20 times more load repetitions over their design life than in the past.

While the base course is the layer most often affected by erosion, any layer directly beneath a treated base can experience serious erosion. There are many examples of the erosion of fine grained soils beneath a stabilized base course causing loss of support and joint faulting. Thus, some agencies now place a dense graded granular subbase layer between the base and compacted subgrade to reduce this problem. Other agencies stabilize the top layer of a fine-grained soil with lime to reduce this problem; however, this approach must produce a sufficiently hard material with adequate compressive strength and uniformity along the project. Geotextiles are also used as separation layers to hold the subgrade materials in place. Another alternative that has been used successfully is to place a layer of recycled crushed PCC beneath the dense treated base.

The NCHRP 1-37A guide provides guidance for assessing the erodibility potential of various materials used in new JPCP and CRCP design and in PCC overlays of existing flexible or rigid pavements. The effect of erosion is considered empirically in the form of erodibility classification assessment for specific design levels. The design procedure provides the framework for which erosion can be considered on a more mechanistic basis in the future (such as iterative month-by-month damage accumulation, and inclusion of Level 1 laboratory erosion test). Tables 7-6, 7-7, and 7-8 provide the Material Classification requirements for Level 1, Level 2, and Level 3 design, respectively.

7.3.2 Bound Bases

In order to achieve the highest erodibility levels, stabilized base or subbase materials often produced by the addition of a sufficient quantity of stabilizing agent (usually cement or asphalt) to produce materials with significant tensile strength (*e.g.,* Erodibility Class 1a in Table 7-7). Such materials are considered to be bound bases and have a substantial increase in structural capacity over that of unbound and modified (treated) bases. Bound bases or subbases are not considered to be geotechnical materials, and are not covered in this manual. Users are referred to NHI courses on pavements (*e.g.,* NHI 131033) for additional information.

Table 7-6. Level 1 recommendation for assessing erosion potential of base material (NCHRP 1-37A).

Erodibility Class	Material Description and Testing
Class based on the material type and test results	Test not fully developed for nationwide uses; thus Level 1 cannot be implemented at this time. The tests currently being considered to assess the erodibility of paving materials include - Rotational shear device for cohesive or stabilized materials (Bhatti et al., 1996). - Jetting test (Bhatti et al., 1996). - Linear and rotational brush tests (Dempsey, 1982). - South African erosion test (DeBeer, 1990).

Table 7-7. Design Level 2 recommendations for assessing erosion potential of base material (*NCHRP 1-37A adapted after the Permanent International Association of Road Congresses, PIARC, 1987*).

Erodibility Class	Material Description and Testing
1	(a) Lean concrete with approximately 8% cement; or with long-term compressive strength > 17.2 MPa (2,500 psi) [> 13.8 MPa (2,000 psi) at 28-days] and a granular subbase layer or a stabilized soil layer or a geotextile fabric is placed between the bound base and subgrade; otherwise Class 2. (b) Hot mixed asphalt concrete with 6% asphalt cement that passes appropriate stripping tests and aggregate tests and a granular subbase layer or a stabilized soil layer; otherwise Class 2. (c) (c) Permeable drainage layer (asphalt-treated aggregate or cement-treated aggregate) and with an appropriate granular or geotextile separation layer placed between the treated permeable base and subgrade.
2	(a) Cement-treated granular material with 5% cement manufactured in-plant, or long-term compressive strength 13.8 to 17.2 MPa (2,000 to 2,500 psi) [10.3 MPa to 13.8 MPa (1,500 to 2,000 psi) at 28-days] and a granular subbase layer or a stabilized soil layer or a geotextile fabric is placed between the treated base & subgrade; otherwise Class 3. (b) (b) Asphalt-treated granular material with 4% asphalt cement that passes appropriate stripping test and a granular subbase layer or a treated soil layer or a geotextile is placed between the treated base and subgrade; otherwise Class 3.
3	(a) Cement-treated granular material with 3.5% cement manufactured in-plant, or with long-term compressive strength 6.9 MPa to 13.8 MPa (1,000 to 2,000 psi) [5.2 MPa to 10.3 MPa (750 to 1,500 psi) at 28-days]. (b) Asphalt-treated granular material with 3% asphalt cement that passes appropriate stripping test.
4	Unbound crushed granular material having dense gradation and high quality aggregates.
5	Untreated soils (PCC slab placed on prepared/compacted subgrade).

Table 7-8. Design Level 3 recommendations for assessing erosion potential of base material based on material description only (NCHRP 1-37A).

Erodibility Class	Material Description and Testing
1	(a) Lean concrete with previous outstanding past performance and a granular subbase layer or a stabilized soil layer or a geotextile layer is placed between the treated base and subgrade; otherwise Class B. (b) Hot mixed asphalt concrete with previous outstanding past performance and a granular subbase layer or a stabilized soil layer is placed between the treated base and subgrade; otherwise Class B. (c) Permeable drainage layer (asphalt- or cement-treated aggregate) and a granular or a geotextile separation layer between the treated permeable base and subgrade. Unbonded PCC Overlays: HMAC separation layer (either dense or permeable graded) is specified.
2	(a) Cement-treated granular material with good past performance and a granular subbase layer or a stabilized soil or a geotextile layer is placed between the treated base and subgrade; otherwise Class C. (b) Asphalt-treated granular material with good past performance and a granular subbase layer or a stabilized soil layer or a geotextile is placed between the treated base and subgrade; otherwise Class C.
3	(a) Cement-treated granular material that has exhibited some erosion and pumping in the past. (b) Asphalt-treated granular material that has exhibited some erosion and pumping in the past. Unbonded PCC Overlays: Surface treatment or sand asphalt is used.
4	Unbound crushed granular material having dense gradation and high quality aggregates.
5	Untreated subgrade soils (compacted).

7.3.3 Modified (or Treated) Bases

The addition of cement or asphalt (typically less than 5%) to stabilize unbound base or subbase with the primary purpose of improving the stability for construction are considered to be modified or treated bases. Modified materials are usually considered to behave structurally as unbound granular material. These bases or subbases are considered to be geotechnical materials. Stabilization is most often required for open graded (permeable) bases (OGB), which tend to rut and weave under construction activities. Tables 7-9 and 7-10 provide the recommendations for asphalt-treated bases and cement-treated bases, respectively.

The strength of cement-treated bases will depend in part on adequate curing during construction. The mixture must be well compacted at optimum moisture content, and adequate density must be obtained throughout the layer. Density control will also be important for the uniformity of asphalt-treated base materials. Although stabilization is often used to reduce the thickness of the base, it should be recognized that thin bases (less than 150 mm (6 in.) thickness) are often extremely difficult to construct to the exact depth, creating the potential for very thin base layers in localized areas. Construction of thin bases requires a very competent subgrade or a good working platform (as reviewed in Section 7.6). Construction quality control for cement- and asphalt-treated materials is reviewed in Chapter 8.

Table 7-9. Recommended asphalt stabilizer properties for asphalt-treated permeable base/subbase materials.

Specification	Requirement	Test Method
Aggregate	(a) hard, durable material with at least two fractured faces; preferably, consisting of 98% crushed stone. (b) L.A. abrasion wear should not exceed 45%. (c) Soundness loss percent should not exceed 12 as determined by the sodium sulfate, or 18% by the magnesium sulfate tests.	Visual Classification AASHTO T 96 AASHTO T 104, *Soundness of Aggregate by the Use of Sodium Sulfate or Magnesium Sulfate*
AC content	AC content must ensure that aggregates are well coated. Minimum recommended AC content is between 2.5 – 3% by weight. Final AC content should be determined according to mix gradation and film thickness around the coarse aggregates.	ASTM D 2489, *Test Method for Degree of Particle Coating of Bituminous-Aggregate Mixtures.*
AC grade	A stiff asphalt grade (typically 1 grade stiffer than the surface course is recommended).	Penetration, viscosity, or Superpave binder testing can be performed to determine AC grade.
Anti-stripping	Anti-stripping test should be performed on all AC treated materials.	AASHTO T283, *Resistance of Compacted Bituminous Mixture to Moisture Induced Damage.*
Anti-stripping Agents	Aggregates exhibiting hydrophilic characteristics can be counteracted with 0.5 – 1% lime.	NCHRP Report 274.
Permeability	Minimum mix permeability: 300 m/day (1000 ft/day).	AASHTO T 3637, *Permeability of Bituminous Mixtures.*

Table 7-10. Recommended Portland cement stabilizer properties for cement-treated permeable base/subbase materials.

Specification	Requirement	Test Method
Aggregate	(a) Hard, durable material with at least two fractured faces; preferably, consisting of 98 percent crushed stone. (b) (a) L.A. abrasion wear should not exceed 45%. (c) (c) Soundness loss percent should not exceed 12 or 18%, as determined by the sodium sulfate or magnesium sulfate tests, respectively.	Visual Classification AASHTO T 96-94 AASHTO T 104-86, "Soundness of Aggregate ...Use of Sodium Sulfate or Magnesium Sulfate"
Cement	Portland cement content selected must ensure that aggregates are well coated. An application rate of 130 to 166 kg/m^3 (220 to 285 lb/yd^3) is recommended.	Must conform to the specification of AASHTO M 85, *Portland Cement*
Water-to-cement ratio	Recommended water-to-cement ratio to ensure strength and workability: 0.3 to 0.5.	
Workability	Mix slump should range between 25 – 75 mm (1 – 3 in.).	
Cleanness	Use only clean aggregates	
Permeability	Minimum mix permeability: (300 m/day) 1,000 ft/day.	

7.3.4 Base Reinforcement

A more recent form of stabilization is the use of geosynthetics (primarily geogrids) to reinforce the base for flexible pavement systems, which has been found under certain conditions to provide significant improvement in performance of pavement sections. The principal effect of reinforcement in base-reinforced flexible pavements is to provide lateral confinement of the aggregate layer. Lateral confinement arises from the development of interface shear stresses between the aggregate and the reinforcement, which, in turn, transfers load to the reinforcement. The interface shear stress present when a traffic load is removed continues to grow with traffic load applications, meaning that the lateral confinement of the aggregate increases with increasing load applications. Increases in traffic volume up to a factor of 10 to reach the same distress level (25-mm (1 in.) rutting) have been observed for reinforced sections, versus unreinforced sections of the same design asphalt and base thickness (Berg et al., 2000). Table 7-11 provides a summary of the conditions for which various geosynthetic products should be considered for this application.

Table 7-11. Qualitative review of reinforcement application potential for paved permanent roads (after Berg et al., 2000).

Roadway Design Conditions		Geosynthetic Type					
		Geotextile		Geogrid [2]		GG-GT Composite	
Subgrade	Base / Subbase Thickness[1] (mm)	Nonwoven	Woven	Extruded	Knitted or Woven	Open-Graded Base[3]	Well-Graded Base
Soft (CBR < 3) (M_R <30 MPa)	150 – 300	❹	●	●	❑	●	⑤
	> 300	❹	❹	◗	◗	◗	⑤
Firm - Vy. Stiff (3 ≤ CBR ≤ 8) (30 ≤ MR ≤ 80)	150 – 300	❍	◗	●	❑	●	⑤
	> 300	❍	❍	❍	❍	❍	❍

KEY: ● — usually applicable ◗ — applicable for some conditions
❍ — usually not applicable ❑ — insufficient information at this time ⑤ — see note

NOTES: 1. Total base or subbase thickness with geosynthetic reinforcement. Reinforcement may be placed at bottom of base or subbase, or within base for thicker (usually > 300 mm (12 in.)) thicknesses. Thicknesses less than 150 mm (6 in.) not recommended for construction over soft subgrade. Placement of less than 150 mm (6 in.) over a geosynthetic not recommended.
2. For open-graded base or thin bases over wet, fine grained subgrades, a separation geotextile should be considered with geogrid reinforcement.
3. Potential assumes base placed directly on subgrade. A subbase also may provide filtration.
❹ Reinforcement usually applicable, but typically addressed as a subgrade stabilization.
⑤ Geotextile component of composite likely is not required for filtration with a well-graded base course; therefore, composite reinforcement usually not applicable.

Current design methods for flexible pavements reinforced with a geosynthetic in the unbound aggregate base layer are largely empirical methods based on a limited set of design conditions over which test sections have been constructed (*i.e.,* AASHTO 4E-SR Standard of Practice Guidelines for Base Reinforcement). These design methods have been limited in use due to 1) absence of nationally recognized reinforced base design procedure, 2) narrow range of test section design conditions from which the method was calibrated, and 3) proprietary design methods pertaining to a single geosynthetic product. Recently FHWA sponsored a study to develop an interface for including geosynthetic base reinforcement in mechanistic empirical design, consistent with the NCHRP 1-37A model. This work is currently in review, but shows excellent promise for the incorporation of these methods into pavement design.

In the interim, AASHTO 4E includes a design approach that relies upon the assessment of reinforcement benefit as defined by a Traffic Benefit Ratio (TBR) or a Base Course reduction Ratio (BCR). TBR is defined as the ratio of the number of traffic loads between an otherwise identical reinforced and unreinforced pavement that can be applied to reach a particular permanent surface deformation of the pavement. BCR defines the percentage reduction in the base course thickness of a reinforced pavement such that equivalent life (*e.g.,* surface deformation) is obtained between the reinforced and the unreinforced pavement with the greater aggregate thickness. The philosophy of this approach is one in which applicability of the technology and reinforcement benefits are assessed by empirical considerations. Reinforcement benefit defined in this manner is then used to modify an existing unreinforced pavement design.

The proposed design procedure in AASHTO 4E follows the steps listed below:

- Step 1. Initial assessment of applicability of the technology.
- Step 2. Design of the unreinforced pavement.
- Step 3. Definition of the qualitative benefits of reinforcement for the project.
- Step 4. Definition of the quantitative benefits of reinforcement (TBR or BCR).
- Step 5. Design of the reinforced pavement using the benefits defined in Step 4.
- Step 6. Analysis of life-cycle costs.
- Step 7. Development of a project specification.
- Step 8. Development of construction drawings and bid documents.
- Step 9. Construction of the roadway.

Step 1 involves assessing the project-related variables given in Table 7-11 and making a judgment on whether the project conditions are favorable or unfavorable for reinforcement to be effective and what types of reinforcement products (as defined in Table 7-11) are appropriate for the project.

Step 2 involves the design of a conventional unreinforced typical pavement design cross section or a series of cross sections, if appropriate, for the project. Any acceptable design procedure can be used for this step.

Step 3 involves an assessment of the qualitative benefits that will be derived by the addition of the reinforcement. The two main benefits that should be assessed are whether the geosynthetic will be used for an extension of the life of the pavement (*i.e.,* the application of additional vehicle passes), a reduction of the base aggregate thickness, or a combination of the two. Berg et al. (2000) has listed additional secondary benefits that should also be considered.

Step 4 is the most difficult step in the design process and requires the greatest amount of judgment. This step requires the definition of the value, or values, of benefit (TBR and/or BCR) that will be used in the design of the reinforced pavement. The definition of these benefit values for a range of design conditions is perhaps the most actively debated and most currently studied topic within this field. Given the lack of a suitable analytical solution for the definition of these terms, Berg et al. (2000) has suggested that these values be determined by a careful comparison of project design conditions, as defined in previous steps, to conditions present in studies reported in the literature. The majority of these studies have been summarized in Berg et al. (2000) in a form that allows direct comparison to known project conditions. In the absence of suitable comparison studies, an experimental demonstration method involving the construction of reinforced and unreinforced pavement test sections has been suggested and described in Berg et al. (2000), and may be used for the definition of benefit for the project conditions. The reasonableness of benefit values should be carefully evaluated such that the reliability of the pavement is not undermined.

Step 5 involves the direct application of TBR or BCR to modify the unreinforced pavement design defined in Step 2. TBR can be directly used to define an increased number of vehicle passes that can be applied to the pavement, while BCR can be used to define a reduced base aggregate thickness such that equal life results. Within the context of an AASHTO pavement design approach, it is possible to calculate a BCR knowing a TBR and vise versa for the specific project design conditions, however this approach has not been experimentally or analytically validated.

With the unreinforced and reinforced pavement designs defined, a life-cycle cost analysis should be performed to assess the economic benefit of reinforcement. This step will dictate whether it is economically beneficial to use the geosynthetic reinforcement. Remaining steps involve the development of project specifications, construction drawings, bid documents, and plans for construction monitoring. Berg et al. (2000) has presented a draft specification that may be adopted for this application.

Even though the application of geosynthetic reinforcement of flexible pavements has been proposed and examined over the past 20 years, research in this area is quite active, meaning that new design methods should be expected in the near future. These new design methods will hopefully provide less empirical methods for assessing reinforcement benefit and be expressed as a function of the variables that are known to influence benefit.

7.4 COMPACTION

Compaction of the subgrade, unbound base, and subbase materials is a basic design detail and is one of the most fundamental geotechnical operations for any pavement project. Compaction is used to increase the stiffness and strength, decrease the permeability, and increase the erosion resistance of geomaterials. Compaction can also reduce the swelling potential for expansive soils. Thus, the intent of compaction is to maximize the soil strength (and minimize the potential volume change) by the proper adjustment of moisture and the densification at or near the ideal moisture content, as discussed in this section.

In most instances, once heavy earthwork and fine grading is completed, the uppermost zone of subgrade soil (roadbed) is improved. The typical improvement technique is by means of water content adjustments and densification by compaction. Higher density requirements are routinely established for the top two feet of at-grade roadbeds and for embankments. The soil in cut areas may need to be undercut and backfilled to obtain the strength and uniformity desired. Heavy proof rolling equipment (270 to 450 kN (30 to 50 tons)) can be used to identify areas of non-uniform support in prepared subgrades. Proofrolling and other field construction aspects of compaction are covered in Chapter 8. Perhaps the most common problem arising from deficient construction is related to moisture-density control, which can be avoided or at least minimized with a thorough plan and execution of the plan as it relates to QC/QA during construction, as reviewed in Chapter 8. This plan should pay particular attention to proper moisture content, proper lift thickness for compaction, and sufficient configuration (e.g., weight and width) of the compaction equipment utilized.

7.4.1 Compaction Theory

The basic engineering principles of soil compaction date back to work by Proctor in the 1930s. Compaction can be performed in the laboratory using static, kneading, gyratory, vibratory, or impact compactors. Each method has its advantages and disadvantages, but impact compaction using a falling hammer is the standard in practice today. Standard laboratory compaction tests are described in more detail in Chapter 5. In these tests, soil is mixed with water at a range of moisture contents w and compacted using a specified compaction energy (e.g., ft-lbs/ft^3 or joules/m^3). Figure 7-11 illustrates the effect of compaction energy on laboratory compaction curves. As described in Chapter 5, the Modified Proctor compaction test (ASTM D1557/ AASHTO T-180) has a compaction energy of 2,700 kN-m/m^3 (56,000 ft-lb/ft^3), which is nearly 5 times the compaction energy of 600 kN-m/m^3 (12,400 ft-lb/ft^3) in the Standard Proctor test (ASTM D698/AASHTO T-99). Likewise, increased compaction energy in the field will increase the maximum dry unit weight and decrease the associated optimum water content.

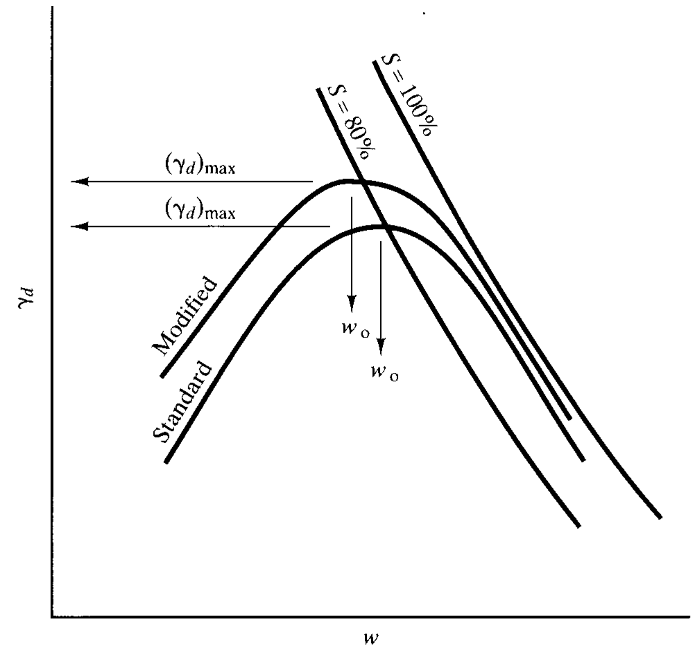


Figure 7-11. Effect of compaction energy on compaction curves (Coduto, 1999).

Different soils will generally have differently shaped compaction curves. This fact will aid in identifying the corresponding laboratory curve for materials encountered in the field. Figure 7-12 shows typical compaction curves for several different soils. Coarser, granular soils typically have fairly steep compaction curves, with large changes in density for small changes in moisture content, while highly plastic clays exhibit fairly flat compaction curves. The maximum dry density is higher for coarser soils and the optimum moisture content is lower. Some cohesionless soils will also exhibit two peaks in the compaction curve; one at very dry conditions, where there are no capillary tensions to resist the compaction effort, and the other at the optimum moisture content, where optimum lubrication between particles occurs.

Nearly all compaction specifications are based on achieving a minimum dry unit weight in the field. This is usually expressed in terms of the relative compaction C_R:

$$C_R = \frac{\gamma_d}{(\gamma_d)_{\max}} \times 100\% \qquad \text{Eq. 7-5}$$

in which γ_d is the dry unit weight achieved in the field and $(\gamma_d)_{max}$ is the maximum dry unit weight as determined from a specified laboratory compaction test.

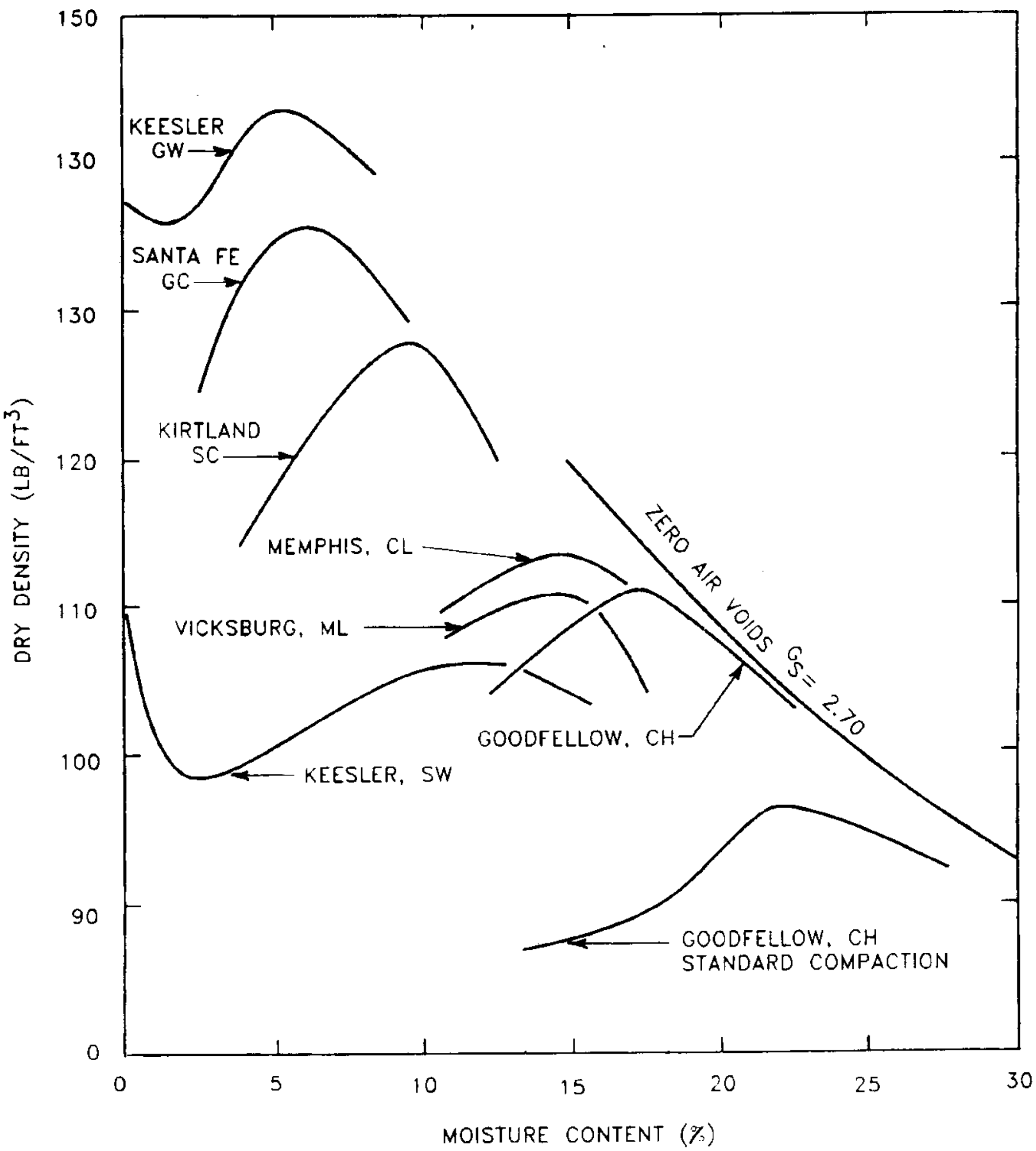


Figure 7-12. Laboratory compaction curves for different soils (Rollings and Rollings, 1996).

The water content at compaction is also sometimes specified because of its effect on soil fabric, especially for clays. Clays compacted dry of optimum have a flocculated fabric (see Figure 7-13), which generally corresponds to higher permeability, greater strength and stiffness, and increased brittleness. Conversely, clays compacted wet of optimum to the same equivalent dry density tend to have a more oriented or dispersed fabric, which typically corresponds to lower permeability, lower strength and stiffness, but more ductility.

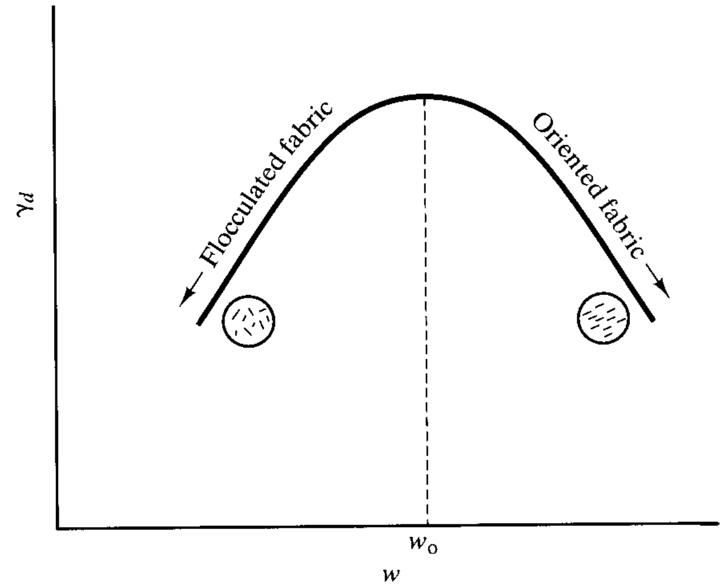


Figure 7-13. Effect of compacted water content on soil fabric for clays (Coduto, 1999).

7.4.2 Effect on Soil Properties

The principal effects of compaction on soil properties are as follows:

- *Density:* As described in the preceding sections, the most direct measurable effect of compaction is an increase in soil density. Typical laboratory values of maximum dry density values and optimum moisture contents for different soils were summarized in Chapter 5, Table 5-18 and 5-19.
- *Strength:* Intuitively, one expects strength to increase with compaction energy and to be larger at low water contents than at high values. Figure 7-14 summarizes typical strength versus water content and compaction energy for a lean clay where strength is quantified by CBR (Rollings and Rollings, 1996). The data in the figure generally confirm intuitive expectations. The strength dry of the optimum water content is larger for higher compaction energies, as expected, and is up to an order of magnitude higher than the strength when compacted wet of optimum. Note, however, that higher compaction energies can produce slightly lower strength values when a fine-grained soil is compacted at water contents higher than the optimum. **Also note that the strength in the figure is based on unsaturated soils. If material compacted dry of optimum becomes saturated, a significant decrease in strength can occur, with strengths even less than that of the same soil compacted wet of optimum**. Large changes in strength upon wetting are associated with fine-grained silts and clays, and are less pronounced or even negligible in coarse-grained soils (Rollings and Rollings, 1996).

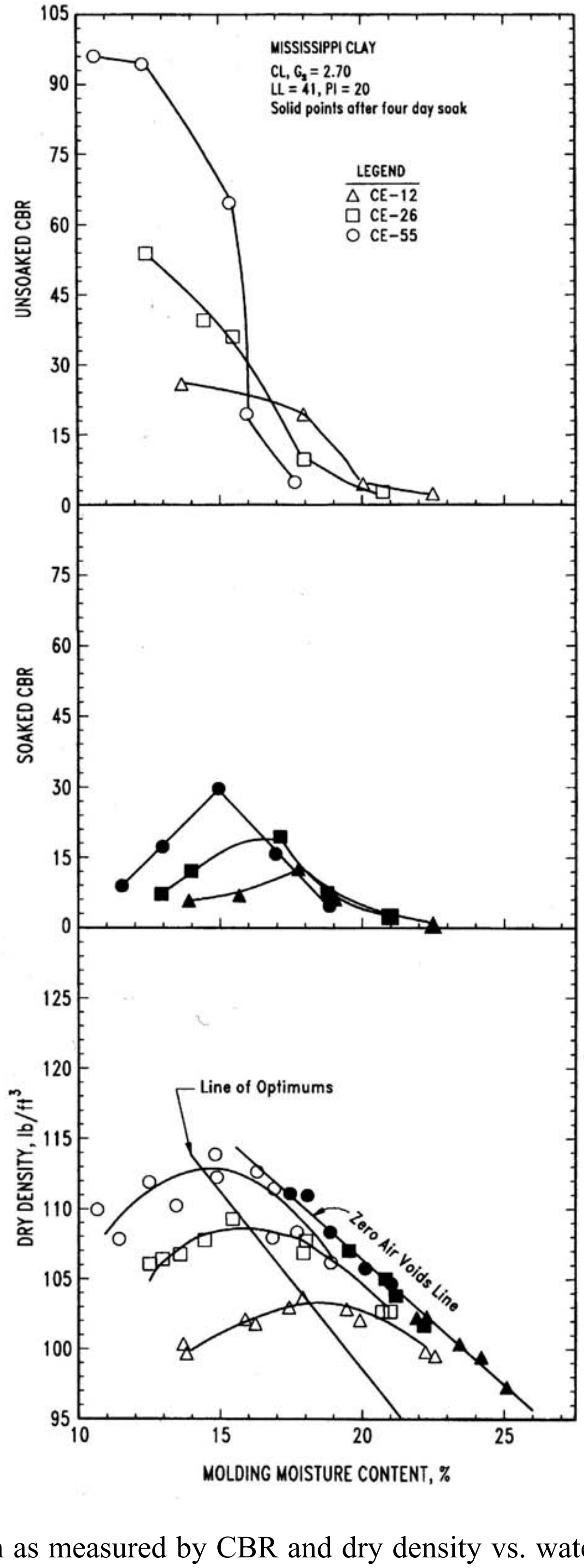


Figure 7-14. Strength as measured by CBR and dry density vs. water content for laboratory impact compaction (Rollings and Rollings, 1996).

- *Stiffness:* Figure 7-15 summarizes typical stiffness versus water content and compaction energy behavior for clays, where stiffness is defined as the stress required to case 5% and 25% axial strain in a triaxial compression test (Seed and Chan, 1959). Stiffness increases with compaction energy when compacted dry of optimum and is largely independent of compaction energy when compacted wet of optimum. The stiffness dry of optimum is also substantially larger than when compacted wet of optimum, as would be expected. **Again however, a significant decrease in stiffness can occur if the material becomes saturated to the extent that the stiffness could be less than that of the soil compacted wet of optimum.**
- *Permeability:* Permeability at constant compactive effort decreases with increasing water content and reaches a minimum at about the optimum moisture content. The permeability when compacted dry of optimum is about an order of magnitude higher than the value when compacted wet of optimum.
- *Swelling/Shrinkage Potential:* Swelling of compacted clays is greater when compacted dry of optimum. Dry clays have a greater capacity to absorb water, and thus swell more. Soils dry of optimum are in general more sensitive to environmental influences, such as changes in water content. The situation is just the opposite for shrinkage (Figure 7-16), where samples compacted wet of optimum exhibit the highest shrinkage strains as water is removed from the soil.

7.5 SUBGRADE CONDITIONS REQUIRING SPECIAL DESIGN ATTENTION

Considering variables such as soil type or mineralogy along a length of roadway, the geology (soil genesis and deposition method) and groundwater and flow properties make each project unique with respect to subgrade conditions. It is not surprising that certain conditions will exist that are not conducive to support, or even construction, of pavement systems. This section provides an overview of subgrade conditions that require special design attention. These subsurface conditions are often regional in nature and have usually been identified as problematic by the agency. Several foundation problems, such as collapsible or highly compressible soils, expansive or swelling soils, subsurface water and saturated soils, and frost-susceptible soils, occur extensively across the U.S. and are not specific to one region. For example, frost heave occurs in over half of the states in the U.S. and damage may be most severe in the central states, where many more frost cycles occur than in the most-northern states. Identification of these widely variable problematic subgrade conditions are also reviewed in this section, along with design and construction alternatives to achieve an adequate foundation on which to build the pavement structure.

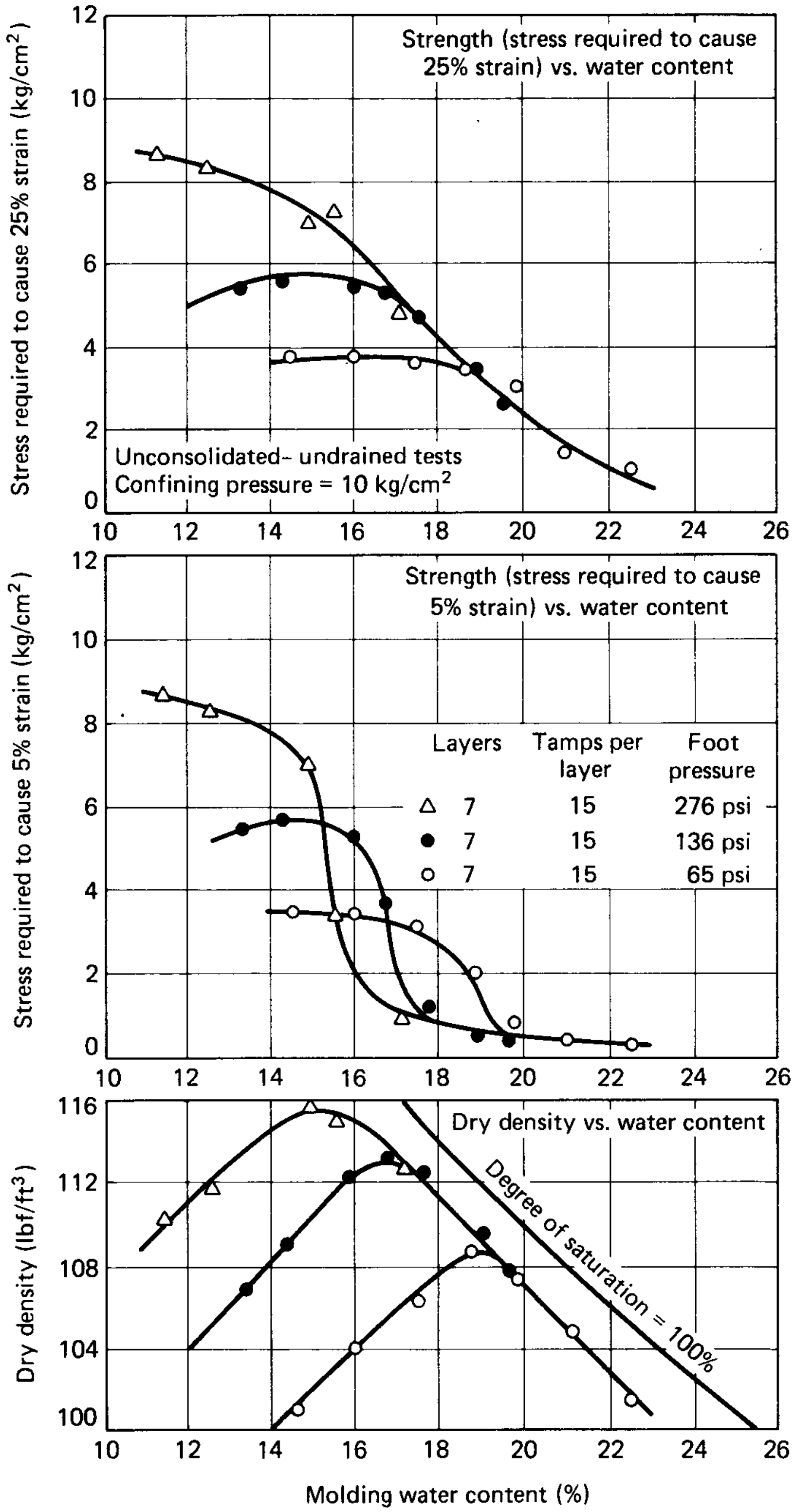


Figure 7-15. Stiffness as a function of compactive effort and water content (after Seed and Chan, 1959; from Holtz and Kovacs, 1981).

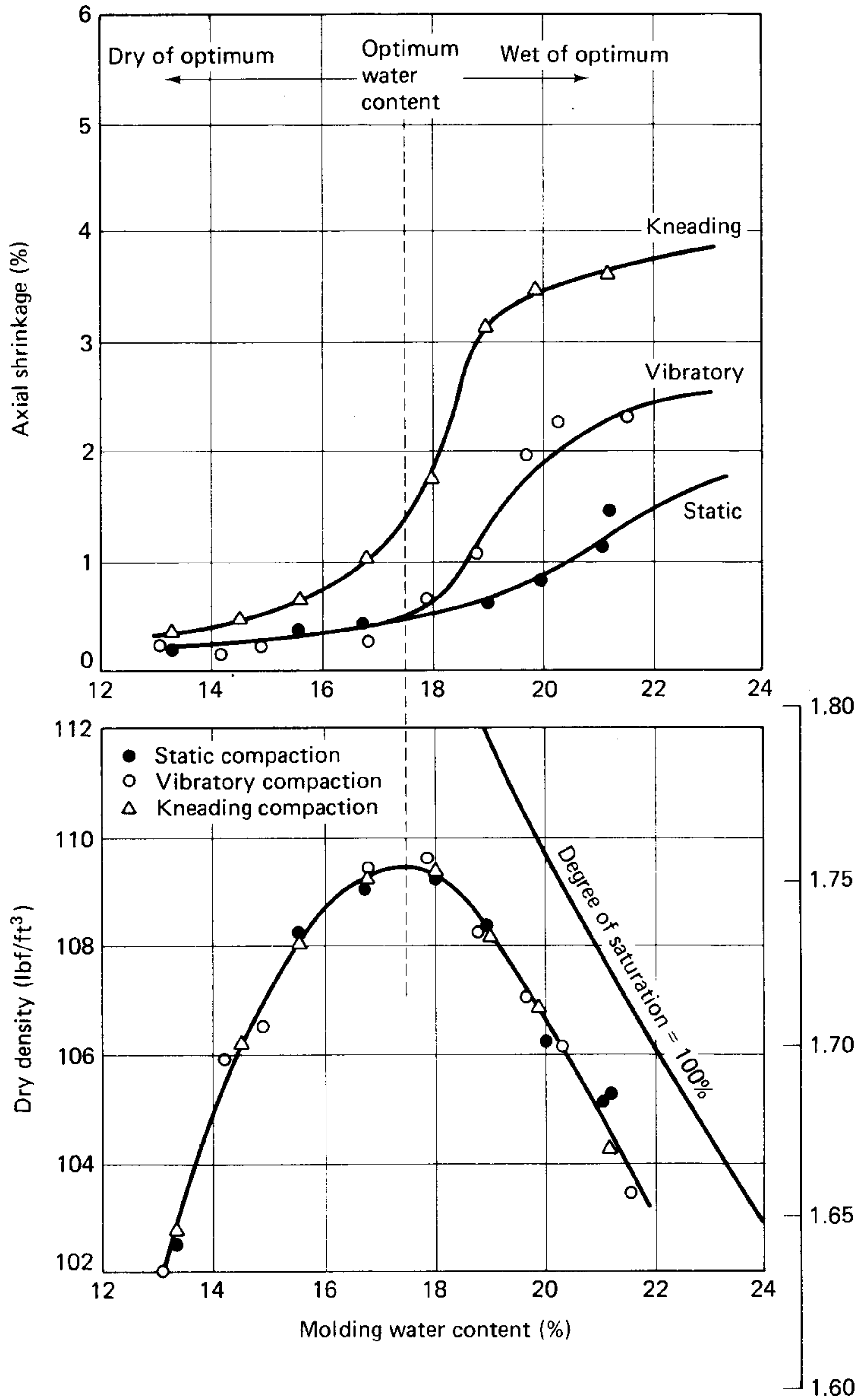


Figure 7-16. Shrinkage as a function of water content and type of compaction (after Seed and Chan, 1959; from Holtz and Kovacs, 1981).

Most of the subgrade conditions presented in this section can be anticipated through a complete exploration program, as described in Chapter 4, and mitigated or at least minimized via well-conceived designs. By identifying such subgrade issues in the design stage, or even the potential for such problems along an alignment, alternative designs can be established. Alternate designs can then be placed in the bid documents with indicators clearly identified that show where these alternatives should be considered, and then implemented if and where such conditions are encountered. When these special subgrade conditions are not recognized in design, they are often identified during construction, usually resulting in claims and overruns. However, identifying problems in construction is still somewhat fortunate, considering the impact such problems may have on the pavement performance. If the soil conditions described in this section go undetected, there typically is decreased serviceability, usually resulting in premature localized rehabilitation or, not uncommon, reconstruction of the pavement within the first few years of the pavement performance period.

7.5.1 Problematic Soil Types

Obviously, a pavement is to be constructed on whatever material and condition is naturally occurring. The strength and stability of some soils can present problems during construction and certainly can affect the long-term performance of the pavement during its service life. In order to properly discuss these potential problems, it is necessary to define some terms as they relate to problematic mineralogy (Sowers, 1979). Some of the terms are true geological terminology, while some are local or regional terminology. The terms may describe a particular material or condition, but all are problematic and care must be taken when constructing pavements in regions containing these materials.

Adobe. Sandy clays of medium plasticity found in the semiarid regions of the southwestern U.S. These soils have been used for centuries to make sun-dried brick. The name is also applied to some highly plastic clays of the West, which swell significantly when wet.

Bentonite. Highly plastic clay, usually montmorillonite, resulting from the decomposition of volcanic ash. It may be hard when dry, but swells considerably when wet.

Buckshot clay. Applied to clays of the southern and southwestern United States. Cracks into small, hard, relatively uniform sized lumps on drying. Dry lumps will degrade upon wetting (*e.g.,* after they have been used as fill). These soils also tend to swell when wet.

Caliche. A silt or sand of the semiarid areas of the southwestern United States that is cemented with calcium carbonate. The calcium carbonate is deposited by the evaporation of water brought to the ground surface by capillary action. The consistency of caliche varies from soft rock to firm soil.

Coquina. A soft, porous limestone made up largely of shells, coral, and fossils cemented together. Very friable, and breaks down during construction.

Gumbo. A fine-grained, highly plastic clay of the Mississippi Valley. It has a sticky, greasy feel, highly expansive, and forms large shrinkage cracks on drying.

Kaolin. A white or pink clay of low plasticity. It is composed largely of minerals of the kaolinite family.

Loam. A surface soil that may be described as a sandy silt of low plasticity or a silty sand that is well suited to tilling. It applies to soils within the uppermost horizons and should not be used to describe deep deposits of parent material. Loam-type soils are typically sensitive to moisture, easily disturbed in construction, and frost susceptible.

Loess. A deposit of relatively uniform, windblown silt. It has a loose structure, with numerous rootholes that produce vertical cleavage and high vertical permeability. It consists of angular to subrounded quartz and feldspar particles cemented with calcium carbonate or iron oxide. Upon saturation, it becomes soft and compressible because of the loss of cementing. Loess altered by weathering in a humid climate often becomes more dense and somewhat plastic (*loess loam*). Loess is also highly frost susceptible.

Marine clay. Clays deposited in a marine environment, which, if later uplifted, tend to be extra sensitive due to salt leaching, dramatically losing strength when disturbed.

Marl. A water-deposited sand, silt, or clay containing calcium carbonate. Marls are often light to dark gray or greenish in color and sometimes contain colloidal organic matter. They are often indurated into soft rock.

Muck or mud. An extremely soft, slimy silt or organic silt found on river and lake bottoms. The terms indicate an extremely soft consistency rather than any particular type of soil. Muck implies organic matter.

Peat. A naturally occuring highly organic substance derived primarily from plant materials (ASTM D 5715). Peats are dark brown or black, loose (void ratio may be 5 to 10), and extremely compressible. When dried, they will float. Peat bogs often emit quantities of inflammable methane gas. These soils will experience significant short-term and long-term settlement, even under light loads, and are often moisture sensitive, losing significant strength when wet. They are easily disturbed under construction activities. Peat containing a high degree of easily identifiable fibers is often called *fibrous peat* for geotechnical applications. Peat containing highly decomposed fibers and a significant highly organic soil component is often called *amorphous peat*.

Quicksand. Refers to a condition, not a soil type. Gravels, sands, and silts become "quick" when an upward flow of groundwater and/or gas takes place to such a degree that the particles are lifted.

Saprolites. Soils developed from in-situ weathering of rocks. Relic joints from the parent rock often control the weathered soils' strength, permeability, and stability. Fragments may appear sound, but prove to be weak. Identifying the transition of soil to weathered rock to sound rock is difficult, often resulting in claims.

Shale. Indurated, fine grained, sedimentary rocks, such as mudstones, siltstone, and claystone, which are highly variable and troublesome. Some are hard and stable, while others are soft and degrade into clay soon after exposure to the atmosphere or during the design life of the structure. Clays developed from shale are often highly plastic.

Sulfate. A mineral compound characterized by the sulfate radical SO_4, which may be contained in soil. It creates significant expansion problems in lime-stabilized soil and, in some cases, distress in concrete.

Sulfide. A mineral compound characterized by the linkage of sulfur with a metal, such as lead or iron, creating galena and pyrite, respectively.

Till. A mixture of sand, gravel, silt, and clay produced by the plowing action of glaciers. The name boulder clay is often given such soils, particularly in Canada and England. The characteristics of glacial till vary depending on the sediments and bedrock eroded. The tills in New England are typically coarser and less plastic that those from the Midwest. The tills in the Northeast tend to be broadly graded and often unstable under water action. The complex nature of their deposition creates a highly unpredictable material.

Topsoils. Surface soils that support plant life. They usually contain considerable organic matter. These soils tend to settle over time as organic matter continues to degrade. They are often moisture sensitive, losing significant strength when wet, and are easily disturbed under construction activities.

Tuff. The name applied to deposits of volcanic ash. In humid climates or in areas in which ash falls into bodies of water, the tuff becomes cemented into a soft, porous rock.

Varved clays. Sedimentary deposits consisting of alternate thin layers of silt and clay. Ordinarily, each pair of silt and clay layers is from 3 – 13 mm (1/8 – 1/2 in.) thick. They are the result of deposition in lakes during periods of alternating high and low water in the inflowing streams, and are often formed in glacial lakes. These deposits have a much higher horizontal than vertical permeability, with the horizontal seams holding water. They are often sensitive, and will lose strength when remolded.

7.5.2 Compressible Soils

Effect of Compressible Soils on Pavement Performance

Highly compressible (very weak) soils are susceptible to large settlements and deformations with time that can have a detrimental effect on pavement performance. Highly compressible soils are very low density, saturated soils, usually silts, clays, and organic alluvium or wind blown deposits and peats. 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 during wet weather.

Treatments for Compressible Soils

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, 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:

- 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% of the maximum density, with moisture control, as defined by AASHTO T180. Cohesive fill materials should be compacted to no less than 90%, near or slightly greater than optimum moisture content (*e.g.,* -1% to +2% of optimum), as defined by AASHTO T99.
- Consider mechanical stabilization using geosynthetics as covered in Section 7.5 to reduce the amount of undercut required.
- If soils are granular (*e.g.,* sands and some silts), consider compaction of the soils from the surface to increase the dry density through dynamic compaction techniques. Identification of soil characteristics and detailed procedures for the successful implementation of this technique covered in FHWA/NHI course 132034 on *Ground Improvement Techniques* (FHWA NHI-04-001).
- If the soil is extremely wet or saturated, consider dewatering using well points or deep horizontal drains. If horizontal drains cannot be daylighted, connection to storm drainage pipes or sump pumps may be required.
- Consolidate deep deposits of very weak saturated soils with large fills prior to pavement construction (surcharge). After construction, the fills can either be left in-place or removed, depending on the final elevation. Consider wick drains to accelerate consolidation (see FHWA NHI-04-001).
- Other techniques for deep deposits of compressible soil include piled embankments and use of lightweight fill, such as geofoam, as covered in the FHWA *Ground Improvement Techniques* manual (FHWA NHI-04-001). Although more costly than most of the previous techniques in terms of construction dollars, these techniques offer immediate improvement, thus accelerating construction. On some projects, the time savings may be more valuable than the construction cost differential.

7.5.3 Collapsible Soils

As with highly compressible soils, collapsible soils can lead to significant localized subsidence of the pavement. Collapsible soils are very low density silt type soils, usually alluvium or wind blown (loess) deposits, and are susceptible to sudden decreases in volume when wetted. Often their unstable structure has been cemented by clay binders or other deposits, which will dissolve on saturation, allowing a dramatic decrease in volume (Rollings and Rollings, 1996). Native subgrades of collapsible soils should be soaked with water prior to construction and rolled with heavy compaction equipment. In some cases, residual soils may also be collapsible due to leaching of colloidal and soluble materials. Figure 7-17 provides a method of identifying the potential for collapsible soils. Other local methods for identification may be available. Collapsible soils can also be created in fills when sand type soils are compacted on the dry side of optimum moisture. Meniscus forces between particles can create a soil fabric susceptible to collapse.

If pavement systems are to be constructed over collapsible soils, special remedial measures may be required to prevent large-scale cracking and differential settlement. To avoid problems, collapse must be induced prior to construction. Methods include

1. ponding water over the region of collapsible soils.
2. infiltration wells.
3. compaction - conventional with heavy vibratory roller for shallow depths (within 0.3 or 0.6 m (1 or 2 ft))
4. compaction - dynamic or vibratory for deeper deposits of more than half a meter (a few feet) (could be combined with inundation)
5. excavated and replaced.

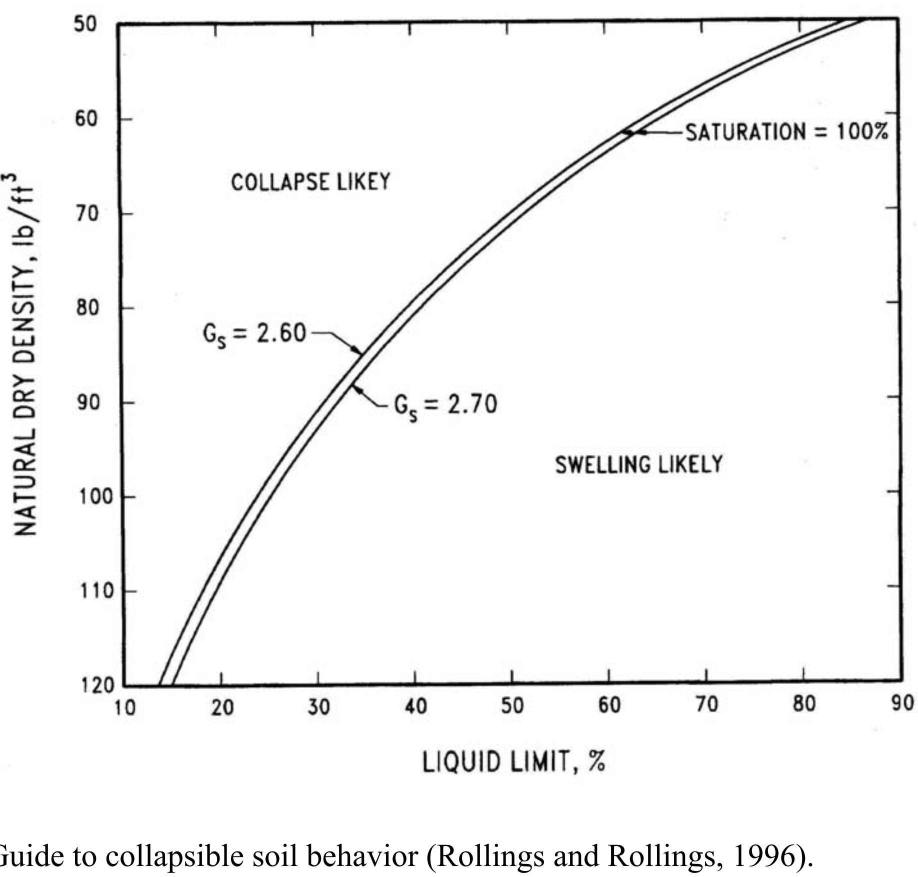


Figure 7-17. Guide to collapsible soil behavior (Rollings and Rollings, 1996).

7.5.4 Swelling Soils

Effect of Swelling Soils on Pavement Performance

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 (see Figure 7-18) 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 listed below:

- *An Evaluation of Expedient Methodology for Identification of Potentially Expansive Soils*, Report No. FHWA-RD-77-94, Federal Highway Administration, Washington, D.C., June 1977.
- *Design and Construction of Airport Pavements on Expansive Soils*, Report No. FAA-RD-76-66, Federal Aviation Administration, U.S. Department of Transportation, Washington, D.C., June 1976.

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. Montmorillonitic 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 in the subgrade 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 (ASTM D 4318), defined as the ratio of the plasticity index to the percentage of the soil by weight finer than 0.002 mm (0.08 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 states. In addition to index testing, agency practice in regions where swelling soils are a common problem may include swell testing (e.g., ASTM D 4546), 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.

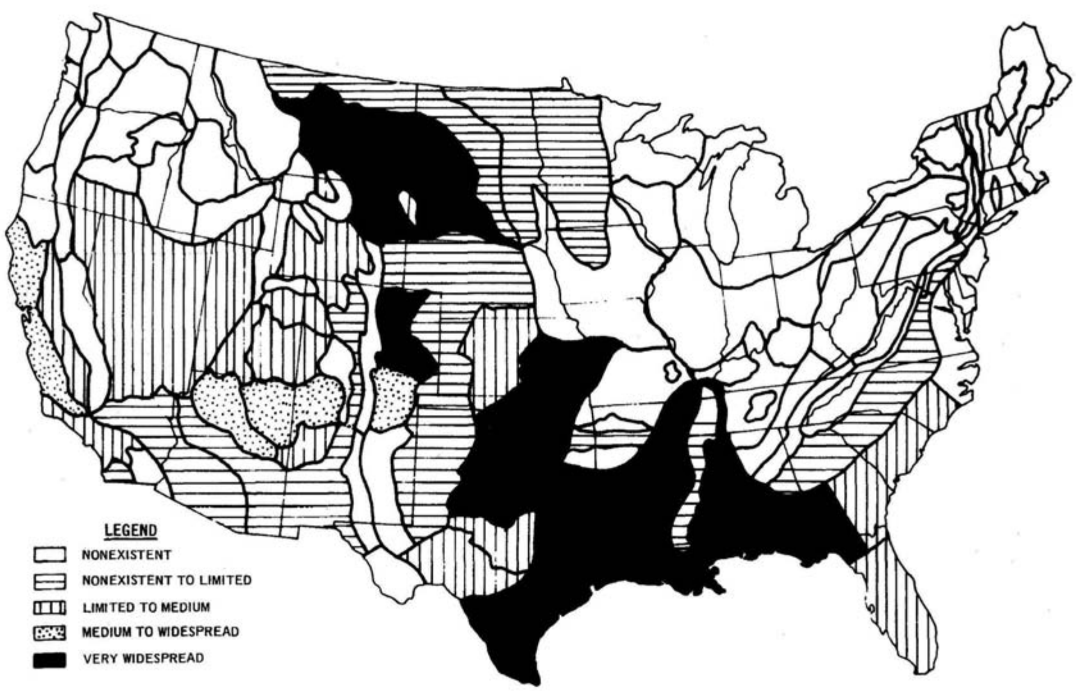


Figure 7-18. Estimated location of swelling soils (from Witczak, 1972).

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 change (*i.e.,* wetting or drying) in subgrade moisture along the pavement's edge, and increase the roadway crown to reduce infiltration moisture.
- Partial encapsulation along the edge of the pavement or full encapsulation can also be used to reduce change in subgrade moisture, as described in greater detail in Section 7.5.
- Scarify, stabilize, and recompact the upper portion of the expansive clay subgrade. Lime or cement stabilization is an accepted method for controlling the swelling of soils, as discussed in Section 7.6. (*Stabilization*, as used for expansive soils, refers to the treatment of a soil with such agents as bitumen, Portland cement, slaked or hydrated lime, and flyash to limit its volume change characteristics. This can substantially increase the strength of the treated material.)
- 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.

AASHTO 1993 (Appendix C) provides procedures and graphs to predict the direct effect of swelling soils on serviceability loss and is treated with respect to the differential effects on the longitudinal profile of the road surface. If the swelling is anticipated to be relatively uniform, then the procedures do not apply.

7.5.5 Subsurface Water

It is important to identify any saturated soil strata, the depth to groundwater, 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 significantly decrease the strength and stiffness of these materials. 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, as described in Section 7.6.
- Remove and replace the saturated soils with select borrow materials or soils. (May not be an option if excavation is required below the groundwater level).
- 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.
- Consideration should also be given to the use of subgrade drains as previously detailed in Section 7.2 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 consisting 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 (consider intercepting drains).

7.5.6 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, regional climatic conditions (*i.e.,* depth of freeze), 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. As shown in Figure 7-19, three conditions must be present to cause frost heaving and associated frost action problems:

- frost-susceptible soils;
- subfreezing temperatures in the soil; and,
- 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.

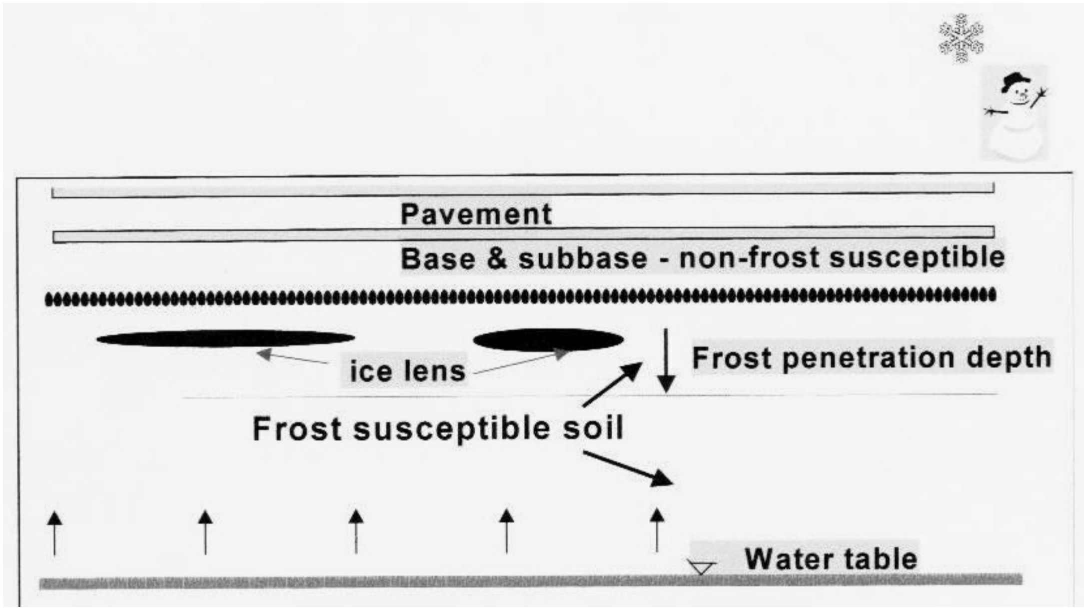


Figure 7-19. Elements of frost heave.

A second effect of frost action is thaw weakening. The bearing capacity may be reduced substantially during mid-winter thawing periods, and subsequent frost heaving is usually more severe 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. Water is also often trapped above frozen soil during the thaw, which occurs from the top down, creating the potential for long-term saturated conditions in pavement layers.

Identification of Frost-Susceptible Soils

Frost-susceptible soils have been classified into four general groups. Table 7-12 provides a summary of the typical soils in each of these four groups based on the amount of fines (material passing the 0.075 mm (No. 200) sieve. Figure 7-20 graphically displays the expected average rate of frost heave for the different soil groups based on portion of soil finer than 0.02 mm (0.8 mils).

Little to no frost action occurs in clean, free draining sands, gravels, crushed rock, and similar granular materials, 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.

Table 7-12. Frost susceptibility classification of soils (NCHRP 1-37A).

Frost Group	Degree of Frost Susceptibility	Type of Soil	Percentage Finer than 0.075 mm (# 200) by wt.	Typical Soil Classification
F1	Negligible to low	Gravelly soils	3-10	GC, GP, GC-GM, GP-GM
F2	Low to medium	Gravelly soils	10-20	GM, GC-GM, GP-GM
		Sands	3-15	SW, SP, 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

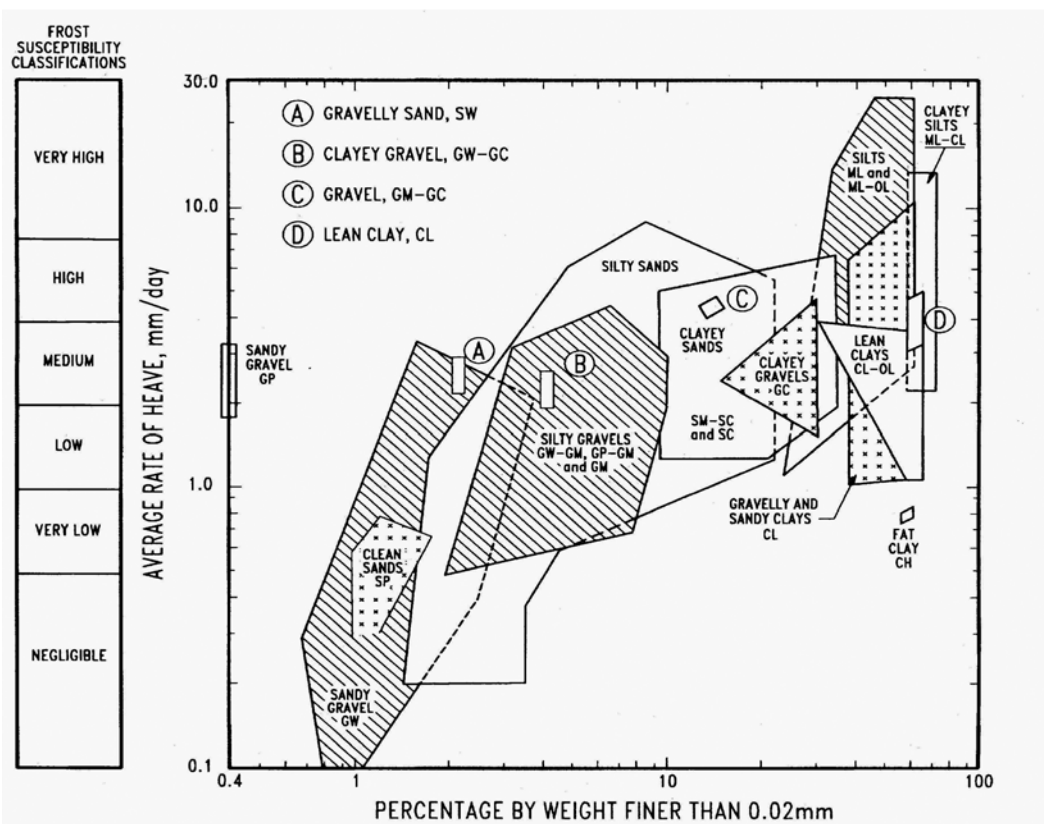


Figure 7-20. Average rate of heave versus % fines for natural soil gradations (Kaplar, 1974).

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, even in the absence of significant heave. Thawing usually takes place from the top downward, leading to very high moisture contents in the upper strata.

A groundwater level within 1.5 m (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 groundwater in excess of 3 m (10 ft). However, the magnitude of influence will be highly dependent on the depth of the freezing front (*i.e.,* frost depth penetration). For deep frost penetration, groundwater at even a greater depth could have an influence on heave.

Identification of Frost-Susceptible Conditions

The most distinguishing factor for identifying a pavement frost hazard condition is water supply. For frost susceptible soils within the frost zone, the frost hazard may be rated as high or low, according to the following conditions. An unknown rating may be appropriate when conditions for both high and low ratings occur and cannot be resolved, or when little or no information is available. The inclusion of a frost hazard rating in the site evaluation documentation verifies that an evaluation of frost action has been attempted and has not been overlooked. When the rating is unknown, a decision to include frost action mitigation measures in a design will be based more upon the unacceptable nature of frost damage than the probability of occurrence.

The conditions associated with a high frost hazard potential include

1. A water table within 3 m (10 ft) of the pavement surface (depth of influence depends on the type of soil and frost depth).
2. Observed frost heaves in the area.
3. Inorganic soils containing more than 3% (by weight) or more grains finer than 0.02 mm (0.8 mils) in diameter according to the U.S. Army Corps of Engineers.
4. A potential for the ponding of surface water and the occurrence of soils between the frost zone beneath the pavement and the surface water with permeabilities high enough to enable seepage to saturate soils within the frost zone during the term of ponding.

The conditions associated with a low frost hazard potential include

1. A water table greater than 6 m (20 ft) below the pavement surface (again, could be much shallower depending on the type of soil and frost depth).

2. Natural moisture content in the frost zone low versus the saturation level.
3. Seepage barriers between the water supply and the frost zone.
4. Existing pavements or sidewalks in the vicinity with similar soil and water supply conditions and without constructed frost protection measures that have not experienced frost damage.
5. Pavements on embankments with surfaces more than 1 – 2 m (3 – 6 ft) above the adjacent grades (provides some insulation and a weighting action to resist heave).

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:

1. Remove the frost-susceptible soil (generally for groups F3 and F4, Table 7-12) and replace with select non-frost susceptible borrow to the expected frost depth penetration.
2. Place and compact select non-frost-susceptible borrow materials to a thickness or depth to prevent subgrade freezing for frost susceptible soil groups F2, F3, and F4, Table 7-12.
3. Remove isolated pockets of frost-susceptible soils to eliminate abrupt changes in subgrade conditions.
4. Stabilize the frost-susceptible soil by eliminating the effects of soil fines by three processes: a) mechanically removing or immobilizing by means of physical-chemical means, such as cementitious bonding, b) effectively reducing the quantity of soil moisture available for migration to the freezing plane, as by essentially blocking off all migratory passages, or c) altering the freezing point of the soil moisture.
    a. Cementing agents, such as Portland cement, bitumen, lime, and lime-flyash, as covered in Section 7.5. These agents 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 soils in seasonal frost areas (see Section 7.5 & Appendix F).
    b. Soil moisture available for frost heave can be mitigated through the installation of deep drains and/or a capillary barrier such that the water table is maintained at a sufficient depth to prevent moisture rise in the freezing zone. Capillary barriers can consist of either an open graded gravel layer sandwiched between two geotextiles, or a horizontal geocomposite drain. The installation of a capillary barrier requires the removal of the frost susceptible material to a depth either below frost penetration or sufficiently significant to reduce the influence of frost heave on the pavement. The capillary break must

be drained. The frost susceptible soil can then be replaced and compacted above the capillary barrier to the required subgrade elevation.

5. Increase the pavement structural layer thickness to account for strength reduction in the subgrade during the spring-thaw period for frost-susceptible groups F1, F2, and F3.

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 (*e.g.,* treatment methods 1, 2, and 3 above).

- 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.

AASHTO 1993 (Appendix C) provides procedures and graphs to predict the direct effect of frost heave on serviceability loss and is treated with respect to the differential effects on the longitudinal profile of the road surface. If the frost is anticipated to be relatively uniform, then the procedures do not apply.

For the most part, local frost-resistant design approaches have been developed from experience, rather than by application of some rigorous theoretical computational method. A more rigorous method is available in the NCHRP 1-37A design procedure to reduce the effects of seasonal freezing and thawing to acceptable limits, as discussed in Chapter 6. The Enhanced Integrated Climatic Model is used to determine the maximum frost depth for the pavement system at a particular location. Various combinations of layer thicknesses and material types can be evaluated in terms of their impact on the maximum frost depth and total amount of base and select materials necessary to protect the frost susceptible soils from freezing.

7.5.7 Summary

Problematic soils can be treated using a variety of methods or a combination thereof. Improvement techniques that can be used to improve the strength and reduce the climatic variation of the foundation on pavement performance include

1. Improvement of subsurface drainage (see Section 7.2, and should always be considered).
2. Removal and replacement with better materials (*e.g.,* thick granular layers).
3. Mechanical stabilization using thick granular layers.
4. Mechanical stabilization of weak soils with geosynthetics (geotextiles and geogrids) in conjunction with granular layers.
5. Lightweight fill.
6. Stabilization of weak soils with admixtures (highly plastic or compressible soils).
7. Soil encapsulation.

Details for most of these stabilization methods will be reviewed in the next section.

7.6 SUBGRADE 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. Some agencies have recognized certain materials simply do not perform well, and prefer to remove and replace such soils (*e.g.,* a state specification dictating that frost susceptible loess cannot be present in the frost penetration zone). However, in many cases, this is not the most economical or even desirable treatment (*e.g.,* excavation may create disturbance, plus additional problems of removal and disposal). Stabilization provides an alternate method to improve the structural support of the foundation for many of the subgrade conditions presented in the previous section. 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 stabilization techniques. Stabilization may also be used to improve soil workability, provide a weather resistant work platform, reduce swelling of expansive materials, and mitigate problems associated with frost heave. In this section, alternate stabilization methods will be reviewed, and guidance will be presented for the selection of the most appropriate method.

7.6.1 Objectives of Soil Stabilization

Soils that are 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 stiffness (in terms of resilient modulus) of some soils is highly dependent on moisture and stress state (see Section 5.4). 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 three 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.
2. To strengthen a weak soil and restrict the volume change potential of a highly plastic or compressible soil—for this case, the *modified* soil is usually given some structural value or credit in the pavement design process.
3. To reduce moisture susceptibility of fine grain soils.

A summary of the stabilization methods most commonly used in pavements, the types of soils for which they are most appropriate, and their intended effects on soil properties is presented in Table 7-13.

Mechanical stabilization using thick gravel layers or granular layers in conjunction with geotextiles or geogrids is an effective technique for improving roadway support over soft, wet subgrades. Thick granular layers provide a working platform, but do not provide strengthening of the subgrade. In fact, construction of thick granular layers in some cases results in disturbance of the subgrade due to required construction activities. Thick granular layers are also used to avoid or reduce frost problems by providing a protection to the underlying subgrade layers.

Table 7-13. Stabilization Methods for Pavements (after Rollings and Rollings, 1996).

Stabilization Method	Soil Type	Improvement	Remarks
Mechanical			
- More Gravel	Silts and Clays	None	Reduce dynamic stress level
- Blending	Moderately plastic	None	Too difficult to mix
	Other	Improve gradation Reduce plasticity Reduce breakage	
- Geosynthetics	Silts and Clays	Strength gain through minimum disturbance and consolidation	Fast, plus provides long-term separation
- Lightweight fill	Very weak silts, clays, peats	None Thermal barrier for frost protection	Fast, and reduces dynamic stress level
Admixture			
- Portland cement	Plastic		Less pronounced hydration of cement
	Coarse		Hydration of cement
- Lime	Plastic	Drying	Rapid
		Strength gain	Rapid
		Reduce plasticity	Rapid
		Coarsen texture	Rapid
		Long-term pozzolanic cementing	Slow
	Coarse with fines	Same as plastic	Dependent on quantity of plastic fines
	Nonplastic	None	No reactive material
- Lime-flyash	Same as lime	Same as lime	Covers broader range
- Lime-cement- flyash	Same as lime	Same as lime	Covers broader range
- Bituminous	Coarse	Strengthen/bind waterproof	Asphalt cement or liquid asphalt
	Some fines	Same as coarse	Liquid asphalt
	Fine	None	Can't mix
- Pozzolanic and slags	Silts and coarse	Acts as a filler	Dense and strong
		Cementing of grains	Slower than cement
- Chemicals	Plastic	Strength increase and volume stability	See vendor literature Difficult to mix
Water proofers			
- Asphalt	Plastic and collapsible	Reduce change in moisture	Long-term moisture migration problem
- Geomembranes	Plastic and collapsible	Reduce change in moisture	Long-term moisture migration problem

A common practice in several New England and Northwestern states is to use a meter (3.3 ft) or more of gravel beneath the pavement section. The gravel improves drainage of surface infiltration water and provides a weighting action that reduces and results in more uniform heave. Washington State recently reported the successful use of an 0.4 m (18 in.) layer of cap rock beneath the pavement section in severe frost regions (Ulmeyer et al., 2002).

Blending gravel and, more recently, recycled pavement material with poorer quality soils also can provide a working platform. The gravel acts as filler, creating a dryer condition and decreasing the influence of plasticity. However, if saturation conditions return, the gravel blend can take on the same poorer support characteristics of the subgrade.

Geotextiles and geogrids used in combination with quality aggregate minimize disturbance and allow construction equipment access to sites where the soils are normally too weak to support the initial construction work. They also allow compaction of initial lifts on sites where the use of ordinary compaction equipment is very difficult or even impossible. Geotextiles and geogrids reduce the extent of stress on the subgrade and prevent base aggregate from penetrating into the subgrade, thus reducing the thickness of aggregate required to stabilize the subgrade. Geotextiles also act as a separator to prevent subgrade fines from pumping or otherwise migrating up into the base. Geosynthetics have been found to allow for subgrade strength gain over time. However, the primary long-term benefit is preventing aggregate-subgrade mixing, thus maintaining the thickness of the base and subbase. In turn, rehabilitation of the pavement section should only require maintenance of surface pavement layers.

Stabilization with admixtures, such as lime, cement, and asphalt, have been mixed with subgrade soils used for controlling the swelling and frost heave of soils and improving the strength characteristics of unsuitable soils. For admixture stabilization or modification of cohesive soils, hydrated lime is the most widely used. Lime is applicable in clay 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 12. Lime stabilization 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. 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. Following is a brief description of the characteristics of stabilized soils followed by the treatment procedures. Additional guidance on soil stabilization with admixtures and stabilization with geosynthetics can be obtained from the following resources:

- *"Lime Stabilization – Reactions, Properties, Design, and Construction," State of the Art Report 5*, Transportation Research Board, Washington D.C., 1987.
- *Soil Stabilization for Pavements,* Joint Departments of the Army and Air Force, USA, TM 5-822-14/AFMAN 32-8010, 1994.
- *Geosynthetics Design and Construction Guidelines*, FHWA HI-95-038, 1998.
- *Standard Specifications for Geotextiles* - AASHTO M288, 1997.

7.6.2 Characteristics of Stabilized Soils

Although mechanical stabilization with thick granular layers or geosythetics and aggregate subbase provides the potential for strength improvement of the subgrade over time, this is generally not considered in the design of the pavement section, and no increase in structural support is attributed to the geosynthetic. However, the increase in gravel thickness (minus an allowance for rutting) can contribute to the support of the pavement. Alternatively, the aggregate thickness used in conjunction with the geosynthetic is designed to provide an equivalent subgrade modulus, which can be considered in the pavement design, discounting the additional aggregate thickness of the stabilization layer. Geosynthetics also allow more open graded aggregate, thus providing for the potential to drain the subbase into edgedrains and improving its support value.

The improvement of subgrade or unbound aggregate by application of a stabilizing agent is intended to cause the improvements outlined above (*i.e.,* construction platform, subgrade strengthening, and control of moisture). These improvements arise from several important mechanisms that must be considered and understood by the pavement designer. Admixtures used as 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 admixture type 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 chemical reaction between the soil and stabilizing agent (as in the case of lime or Portland cement). Additional improvement can arise from other chemical-physical reactions that affect the soil fabric (typically by flocculation) or the soil chemistry (typically by cation exchange). The down side of admixtures is that they require up front lab testing to confirm their performance and very good field control to obtain a uniform, long lasting product, as outlined later in this section. There are also issues of dust control and weather dependency, with some methods that should be carefully considered in the selection of these methods.

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, constructability, and the drainage characteristics of the geometric design and the underlying soil. When only a thin zone and/or short roadway length 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 project economics of the earthwork requirements and the depth of influence for the vehicle loads.

7.6.3 Thick Granular Layers

Many agencies have found that a thick granular layer is an important feature in pavement design and performance. 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 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% or greater are used, corresponding to resilient moduli of approximately 120 MPa (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 (GW, GP, SW and SP) 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 100 – 200 mm (4 – 8 in.) 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%. 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 150 – 300 mm (6 – 12 in.) 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 0.5 – 1.5 m (1½ -- 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 0.5 – 1.5 m (1½ – 5 ft). In most cases, embankments greater than 2 m (6 ½ ft) thick have diminishing effects in terms of strength, frost protection, and drainage. Granular embankments greater than 2 – 3 m (6½ – 10 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, non-homogeneous subgrade in the vertical direction. 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 resilient modulus or CBR 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 1 m (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 1 m (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 0.5 m (1.6 ft) thick have minimal impact on the composite subgrade reaction, when loaded under the completed pavement section.

7.6.4 Geotextiles and Geogrids

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.

Geotextile and geogrid materials are the most commonly used geosynthetics 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. Table 7-14 provides a list of transportation applications for specific basic functions of the geosynthetic. 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. Stabilization, as reviewed in this section, is a combination of the separation, filtration, and reinforcement functions. Drainage can also play a role.

The *separation function* prevents the subgrade and the subbase from intermixing, which would most likely occur during construction and in-service due to pumping of the subgrade. The *filtration function* is required because soils requiring stabilization are usually wet and saturated. By acting as a filter, the geotextile retains the subgrade without clogging, while allowing water from the subgrade to pass up into the subbase, thus allowing destabilizing pore pressure to dissipate and promote strength gain due to consolidation. If the subbase is dirty (contains high fines), it may be desirable to use a thick, nonwoven geotextile, which will allow for *drainage* in its plane (*i.e.,* in this case, pore water pressure dissipates through the plane of the geotextile).

Geotextiles and geogrids also provide some level of *reinforcement* by laterally restraining the base or subbase and improving the bearing capacity of the system, thus decreasing shear stresses on the subgrade. Soft, weak subgrade soils provide very little lateral restraint, so when the aggregate moves or shoves laterally, ruts develop on the aggregate surface and also in the subgrade. A geogrid with good interlocking capabilities or a geotextile with good frictional capabilities can provide tensile resistance to lateral aggregate movement. The geosynthetic also increases the system bearing capacity by forcing the potential bearing surface under the wheel load to develop along alternate, longer mobilization paths and, thus, higher shear strength surfaces.

Geotextiles serve best as separators, filters and, in the case of nonwoven geotextiles, drainage layers, while geogrids are better at reinforcing. Geogrids, as with geotextiles, prevent the subbase from penetrating the subgrade, but they do not prevent the subgrade from pumping into the base. When geogrids are used, either the subbase has to be designed as a separator or a geotextile must be used in conjunction with the geogrid, either separately or as a geocomposite.

Table 7-14. Transportation uses of geosynthetic materials (after Koerner, 1998).

General Function	Typical Application
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

Table 7-15. Appropriate subgrade conditions for stabilization using geosynthetics (after FHWA HI-95-038).

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

As defined by AASHTO M288, geotextiles or geogrids in conjunction with an appropriately designed thickness of subbase aggregate provide stabilization for soft, wet subgrades with a CBR of less than 3 (a resilient modulus less than 30 MPa (4500 psi)). Table 7-15 provides subgrade conditions that are considered to be the most appropriate for geosynthetic use. These are conditions where the subgrade will not support conventional construction without substantial rutting. Engineers have compiled over 20+ years of successful use for this application in these types of conditions. Geosynthetics do not provide improvements for expansive soils, and use in stabilization for subgrade conditions that are better than those defined in Table 7-15 is questionable. However, geosynthetics may still provide a valuable function as separators for any subgrade containing large amounts of fines or as base reinforcement, even with competent subgrades, as discussed in Section 7.2.

Separation is a viable function, for soils that are seasonally weak (*e.g.*, from spring thaw) or for high fines content soils, which are susceptible to pumping. This is especially the case for permeable base applications, as covered in Section 7.2. A greater range of geotextile applicability is recognized in the M288 specification (AASHTO, 1997). With a CBR ≥ 3, the geotextile application is identified as separation. By simply maintaining the integrity of the subbase and base layers over the life of the pavement, the serviceability of the roadway section will be extended, and substantial cost benefits can be realized. Research is ongoing to quantify the cost-benefit life cycle ratio of using geosynthetics in permanent roadway systems. Initial work by Al-Qadi, 1997 indicates that the use a geosynthetic separator may increase the number of allowable design vehicles (ESALs) by a factor of two. Considering the cost of a geosynthetic is generally \$1.25/m^2, while the cost of a modern pavement section is on the order of \$25/m^2, the life extension of the roadway section will more than make up for the cost of the geosynthetic. In addition, as previously indicated, the geosynthetic maintains the integrity of the base such that rehabilitation should only require surface pavement restoration. **The ability of a geosynthetic to prevent premature failure and reduce long-term maintenance costs provides extremely low-cost performance insurance.**

The design of the geosynthetic for stabilization is completed using the design-by-function approach in conjunction with AASHTO M288, in the steps from FHWA HI-95-038 outlined below. 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. However, this unbound layer will provide some additional support. If the soil has a CBR of less than 3, and the aggregate thickness is determined based on a low rutting criteria in the following steps, the support for the composite system is theoretically equivalent to a CBR = 3 (resilient modulus of 30 Mpa (4500 psi)). As with thick aggregate fill used for stabilization, the support value should be confirmed though field testing using, for example, a plate load test or FWD test to verify that a minimum composite subgrade modulus has been achieved. Note that the FHWA procedure is controlled by soil CBR, as measured using ASTM C4429.

1. Identify properties of the subgrade, including CBR, location of groundwater table, AASHTO and/or USCS classification, and sensitivity.
2. Compare these properties to those in Table 7-15, 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, and reduce the thickness by 50%, considering the use of a geosynthetic.
5. Determine additional aggregate thickness t_2 needed for establishment of a construction platform. The FHWA procedure requires the use of curves for aggregate thickness vs. the expected single tire pressure and the subgrade bearing capacity, as shown in Figure 7-21, modified for highway applications. For the purposes of this manual, the curves have been correlated with common pavement construction traffic. Select N_c based on allowable subgrade ruts, where:

   N_c = 5 for a low rut criteria (< 50 mm (< 2 in.)),

   N_c = 5.5 for moderate rutting (50 – 100 mm (2 – 4 in.)), and

   N_c = 6 for large rutting (> 100 mm (> 4 in.)).

   (For comparison without a geotextile: N_c = 2.8, 3.0, or 3.3 respectively for low to large ruts.)

   Alternatively, local policies or charts may be used.
6. Select the greater of t_2 or 50% t_1.
7. Check filtration criteria for the geotextile to be used. For geogrids, check the aggregate for filtration compatibility with the subgrade (see Section 7.2), or use a

geotextile in combination with the grid meeting the following criteria. The important measures include the apparent opening size (AOS), the permeability (k), and permittivity (ψ) 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 as follows

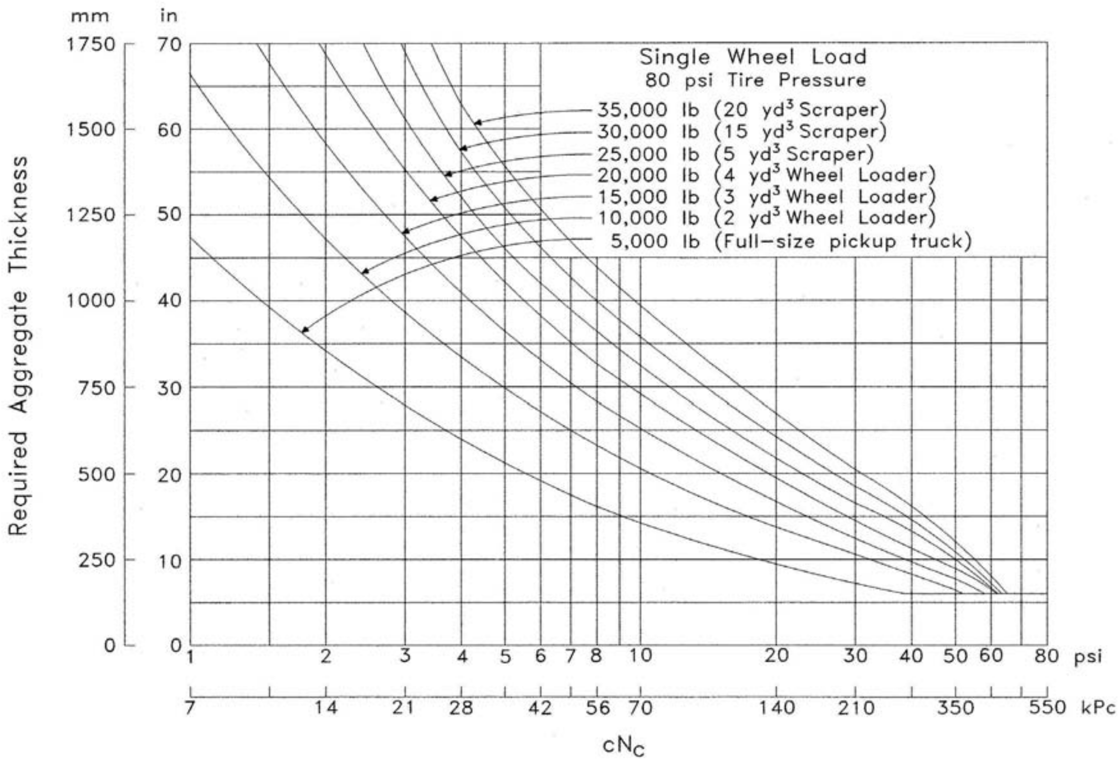
- $AOS \le D_{85}$ (Wovens)
- $AOS \le 1.8\ D_{85}$ (Nonwovens)
- $k_{geotextile} \ge k_{soil}$
- $\psi \ge 0.1\ sec^{-1}$

8. Determine geotextile survival criteria. The design is based on the assumption that the geosynthetic cannot function unless it survives the construction process. The AASHTO M288-99 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 7-5 (Section 7.2.12). For stabilization of soils, the default is Class 1, and for separation, the default is Class 2. These requirements may be reduced based on conditions and experience, as detailed in AASHTO M288. For geogrid survivability, see AASHTO PP46 and Berg et al. (2000).

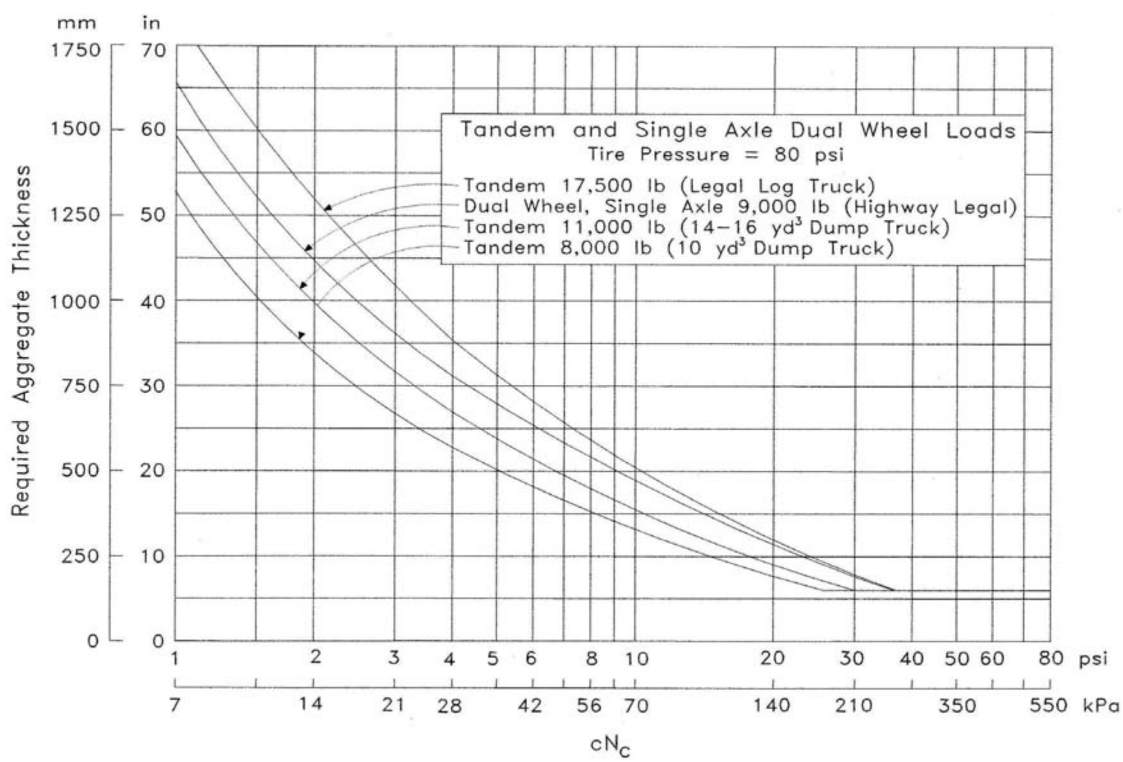
Field installation procedures introduce a number of special concerns; the AASHTO M288 standard includes a guide specification for geotextile construction. FHWA HI-905-038 (Holtz et al. 1998) recommends that this specification be modified to suit local conditions and contractors and provides example specifications. Concerns and criteria for field installation include, for example, the seam lap and sewing requirements, and construction sequencing and quality control.

7.6.5 Admixture Stabilization

As previously indicated in Section 7.6.1, there are a variety of admixtures that can be mixed with the subgrade to improve its performance. The various admixture types are shown in Table 7-15, along with initial guidance for evaluating the appropriate application of these methods. Following is a general overview of each method, followed by a generalized outline for determining the optimum admixture content requirements. Design details for each specific method are contained in Appendix F.



(a)



(b)

Figure 7-21. Thickness design curves with geosynthetics for a) single and b) dual wheel oads (after USFS, 1977, and FHWA NHI-95-038, 1998).

Table 7-16. Guide for selection of admixture stabilization method(s) (Austroads, 1998).

	MORE THAN 25% PASSING 75μm			LESS THAN 25% PASSING 75μm		
Plasticity Index	PI $\leq$ 10	10 < PI <20	PI $\geq$ 20	PI $\leq$ 6 PI x % passing 75μm $\leq$ 60	PI $\leq$ 10	PI > 10
Form of Stabilisation						
Cement and Cementitious Blends	Usually suitable	Doubtful	Usually not Suitable	Usually suitable	Usually suitable	Usually suitable
Lime	Doubtful	Usually suitable	Usually suitable	Usually not Suitable	Doubtful	Usually suitable
Bitumen	Doubtful	Doubtful	Usually not Suitable	Usually suitable	Usually suitable	Doubtful
Bitumen/ Cement Blends	Usually suitable	Doubtful	Usually not Suitable	Usually suitable	Usually suitable	Doubtful
Granular	Usually suitable	Usually not Suitable	Usually not Suitable	Usually suitable	Usually suitable	Doubtful
Miscellaneous Chemicals*	Usually not Suitable	Usually suitable	Usually suitable	Usually not Suitable	Doubtful	Usually suitable

Key: Usually suitable (solid black); Doubtful (hatched); Usually not Suitable (white)

* Should be taken as a broad guideline only. Refer to trade literature for further information.

Note: The above forms of stabilisation may be used in combination, e.g. lime stabilisation to dry out materials and reduce their plasticity, making them suitable for other methods of stabilisation.

Lime Treatment

Lime treatment or modification consists of the application of 1 – 3% 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.

Lime may also be used to treat expansive soils, as discussed in Section 7.3. Expansive soils as defined for pavement purposes are those that exhibit swell in excess of 3%. Expansion is characterized by heaving of a pavement or road when water is imbibed in the clay minerals. The plasticity characteristics of a soil often are a good indicator of the swell potential, as indicated in Table 7-17. If it has been determined that a soil has potential for excessive swell, lime treatment may be appropriate. Lime will reduce swell in an expansive soil to greater or lesser degrees, depending on the activity of the clay minerals present. The amount of lime to be added is the minimum amount that will reduce swell to acceptable limits. Procedures for conducting swell tests are indicated in the ASTM D 1883 CBR test and detailed in ASTM D 4546.

The depth to which lime should be incorporated into the soil is generally limited by the construction equipment used. However, 0.6 – 1 m (2 – 3 ft) generally is the maximum depth that can be treated directly without removal of the soil.

Lime Stabilization

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 significantly improved with lime stabilization, while the strength of coarse-grained soils is usually moderately improved. Lime has been found most effective in improving workability and reducing swelling potential with highly plastic clay soils containing montmorillonite, illite, and kaolinite. Lime is also used to reduce the water content of wet soils during field compaction. In treating certain soils with lime, some soils are produced that are subject to fatigue cracking.

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. Adequate curing is also important if the strength characteristics of the soil are to be improved.

Table 7-17. Swell potential of soils (Joint Departments of the Army & Air Force, 1994).

Liquid Limit	Plasticity Index	Potential Swell
> 60	> 35	High
50 - 60	25 – 35	Marginal
< 50	< 25	Low

The most common varieties of lime for soil stabilization are hydrated lime [$Ca(OH)_2$], quicklime [CaO], and the dolomitic variations of these high-calcium limes [$Ca(OH)_2 \cdot MgO$ and $CaO \cdot MgO$]. While hydrated lime remains the most commonly used lime stabilization admixture in the U.S., use of the more caustic quicklime has grown steadily over the past two decades. Lime is usually produced by calcining[2] limestone or dolomite, although some lime—typically of more variable and poorer quality—is also produced as a byproduct of other chemical processes.

For lime stabilization of clay (or highly plastic) soils, the lime content should be from 3 – 8% of the dry weight of the soil, and the cured mass should have an unconfined compressive strength of at least 0.34 MPa (50 psi) within 28 days. 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. As discussed later in this section, pH can be used to determine the initial, near optimum lime content value. The pozzolanic strength gain in clay soils depends on the specific chemistry of the soil – *e.g.,* whether it can provide sufficient silica and alumina minerals to support the pozzolanic reactions. Plasticity is a rough indicator of reactivity. A plasticity index of about 10 is commonly taken as the lower limit for suitability of inorganic clays for lime stabilization. The lime-stabilized subgrade layer should be compacted to a minimum density of 95%, as defined by AASHTO T99.

Typical effects of lime stabilization on the engineering properties of a variety of natural soils are shown in Table 7-18 and Figure 7-22. These are the result of several chemical processes that occur after mixing the lime with the soil. Hydration of the lime absorbs water from the soil and causes an immediate drying effect. The addition of lime also introduces calcium (Ca^{+2}) and magnesium (Mg^{+2}) cations that exchange with the more active sodium (Na^{+}) and potassium (K^{+}) cations in the natural soil water chemistry; this cation exchange reduces the plasticity of the soil, which, in most cases, corresponds to a reduced swell and shrinkage potential, diminished susceptibility to strength loss with moisture, and improved workability. The changes in the soil-water chemistry also lead to agglomeration of particles and a coarsening of the soil gradation; plastic clay soils become more like silt or sand in texture after the addition of lime. These drying, plasticity reduction, and texture effects all occur very rapidly (usually with 1 hour after addition of lime), provided there is thorough mixing of the lime and the soil.

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[2] Calcining is the heating of limestone or dolomite to a high temperature below the melting or fusing point that decomposes the carbonates into oxides and hydroxides.

Table 7-18. Examples of the effects of lime stabilization on various soils (Rollings and Rollings, 1996).

Soil	Lime %	Atterberg Limits *LL*	Atterberg Limits *PL*	Atterberg Limits *PI*	Strength q_u[a]	Strength CBR
1. CH, residual clay[b]						
(a) Site 1, Dallas–Ft.	0	63	33	30	76	
Worth Airport,	2	62	48	14	123	
residuum from Eagle	3	60	47	13	202	
Ford shale, Britton member	4	56	46	10	323	
(b) Site 2, Dallas–	0	60	27	33	70	
Ft Worth Airport,	2	48	32	16	171	
residuum from Eagle Ford	3	45	32	13	177	
shale, Tarrant member	5	48	34	14	184	
(c) Site 3, Irving, Texas,	0	76	31	45	64	
residuum from Eagle Ford	2	61	45	16	116	
shale, Britton	3	56	45	11	193	
member	5	57	45	12	302	
2. CH, Bryce silty clay,[c]	0	53	24	29	81	
Illinois, B-horizon	3	48	27	21	201	
	5	NP	NP	NP	212	
3. CH, Appling sandy loam,[d]	0	71	33	38	92	
South Carolina, residuum	3				147	
from granite	6				171	
	8				206	
4. CH, St Ann red bauxite	0	58	25	33	119	
clay loam,[d] Jamaica,	3				127	
limestone residuum	5				334	
5. CL,[e] Pelucia Creek Dam,	0	29	18	11		
Mississippi	1	32	19	13		
	2	31	22	9		
	3	30	21	9		
6. CL, Illinoian till, Illinois,[c]	0	26	15	11	43	
glacial till	3	27	21	6	126	
	5	NP	NP	NP	126	
7. SC, sandy clay, San Lorenzo,	0	54	23	31		8
Honduras[f]	5	61	38	23		20
8. MH, Surinam red earth,[d]	0	60	32	28	72	
Surinam,	3				130	
residuum from acidic	5				136	
metamorphic rock						
9. OH, organic soil with 8.1%	0	63	27	36	4	
organics[g]	2			36	4	
	4			24	8	
	8			25	7	

[a]Unconfined compressive strength in psi at 28 days unless otherwise noted; different compaction efforts used by investigators.
[b]McCallister and Petry, 1990, accelerated curing.
[c]Thompson, 1966.
[d]Harty, 1971, 7-day cure.
[e]McElroy, 1989.
[f]Personal communication, Dr. Newel Brabston, Vicksburg, Mississippi.
[g]Arman and Munfakh, 1972, limits at 48 hours, q_u at 28 days, strength samples prepared with moisture content at the *LL*.

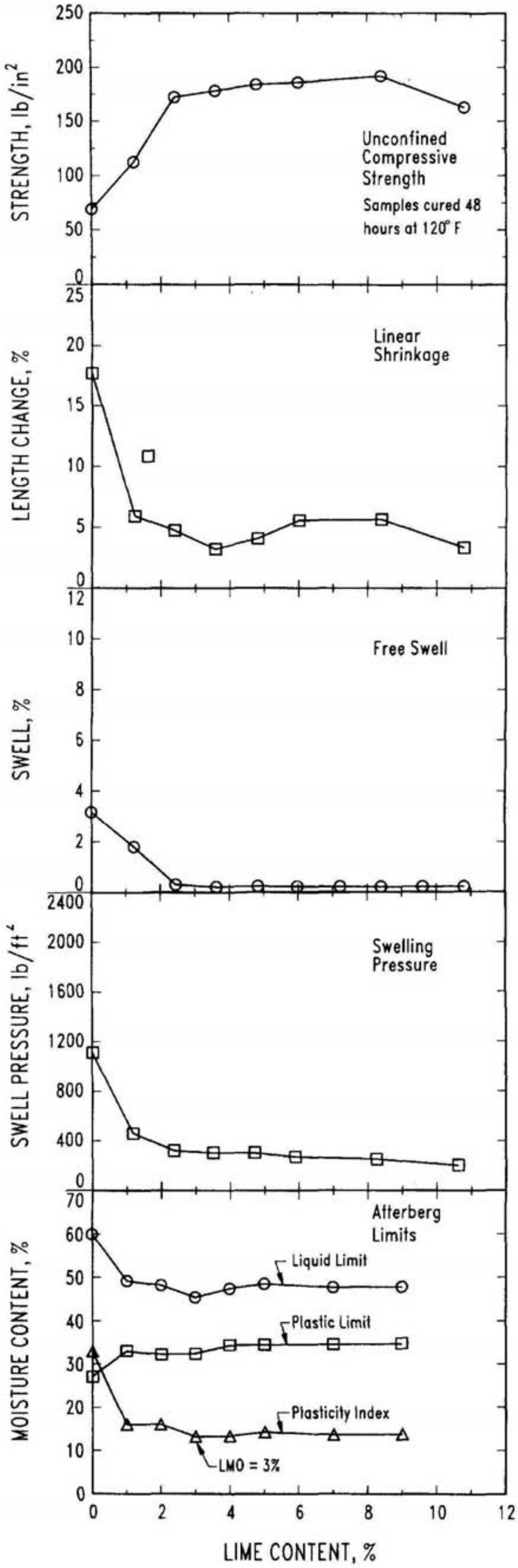

Figure 7-22. Effect of lime content on engineering properties of a CH clay (from Rollings and Rollings, 1996; from data reported by McCallister and Petry, 1990).

When soils are treated properly with lime, it has been observed that the lime-soil mixture may be subject to durability problems, the cyclic freezing and thawing of the soil. The durability of lime stabilization on swell potential and strength may be adversely affected by environmental influences:

- *Water:* Although most lime stabilized soils retain 70% to 85% of their long-term strength gains when exposed to water, there have been reported cases of poor strength retention for stabilized soils exposed to soaking. Therefore, testing of stabilized soils in the soaked condition is prudent.

- *Freeze/thaw cycles:* Freeze/thaw cycles can lead to strength deterioration, but subsequent healing often occurs where the strength loss caused by winter freeze/thaw reverses during the following warm season. The most common design approach is to specify a sufficiently high initial strength gain to retain sufficient residual strength after freeze/thaw damage.

- *Leaching:* Leaching of calcium can decrease the cation exchange in lime stabilized soil, which, in turn, can reverse the beneficial reduction in plasticity and swell potential. The potential for these effects is greater when low lime contents are used.

- *Carbonation:* If atmospheric carbon dioxide combines with lime to form calcium carbonate, the calcium silicate and calcium aluminate hydrate cements may become unstable and revert back to their original silica and alumina forms, reversing the long-term strength increase resulting from the pozzolanic reactions. Although this problem has been reported less in the United States than in other countries, its possibility should be recognized and its potential minimized by use of ample lime content, careful selection, placement, and compaction of the stabilized material to minimize carbon dioxide penetration, as well as prompt placement after lime mixing, and good curing.

- *Sulfate attack:* Sulfates present in the soil or groundwater can combine with the calcium from the lime or the alumina from the clay minerals to form ettringite, which has a volume that is more than 200% larger than that of its constituents. Massive irreversible swelling can therefore occur, and the damage it causes can be quite severe. It is difficult to predict the combinations of sulfate content, lime content, clay mineralogy and content, and environmental conditions that will trigger sulfate attack. Consequently, if there is a suspicion of possible sulfate attack, the lime stabilized soil should be tested in the laboratory to see whether it will swell when mixed and exposed to moisture.

Soils classified as CH, CL, MH, ML, SC, and GC with a plasticity index greater than 12 and with 10% passing the 0.425 mm (No. 40) sieve are potentially suitable for stabilization with lime. 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.

Hydrated lime, in powder form or mixed with water as a slurry, is used most often for stabilization.

Cement Stabilization

Portland cement is widely used 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 or cement-treated base, subbase, or subgrade.

For soils to be stabilized with cement, proper mixing requires that the soil have a PI of less than 20% and a minimum of 45% passing the 0.425 mm (No. 40) sieve. However, highly plastic clays that have been pretreated with lime or flyash are sometimes suitable for subsequent treatment with Portland cement. For cement stabilization of granular and/or nonplastic soils, the cement content should be 3 – 10% of the dry weight of the soil, and the cured material should have an unconfined compressive strength of at least 1 MPa (150 psi) within 7 days. 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% as defined by AASHTO M 134.

Several different types of cement have been used successfully for stabilization of soils. Type I normal Portland cement and Type IA air-entraining cements were used extensively in the past, and produced about the same results. At the present time, Type II cement has largely replaced Type I cement as greater sulfate resistance is obtained, while the cost is often the same. High early strength cement (Type III) has been found to give a higher strength in some soils. Type III cement has a finer particle size and a different compound composition than do the other cement types. Chemical and physical property specifications for Portland cement can be found in ASTM C 150.

The presence of organic matter and/or sulfates may have a deleterious effect on soil cement. Tests are available for detection of these materials and should be conducted if their presence is suspected.

(1) *Organic matter.* A soil may be acid, neutral, or alkaline and still respond well to cement treatment. Although certain types of organic matter, such as undecomposed vegetation, may not influence stabilization adversely, organic compounds of lower molecular weight, such as nucleic acid and dextrose, act as hydration retarders and reduce strength. When such organics are present, they inhibit the normal hardening process. If the pH of a 10:1 mixture (by weight) of soil and cement 15 minutes after mixing is at least 12.0, it is probable that any organics present will not interfere with normal hardening.

(2) *Sulfates.* Although sulfate attack is known to have an adverse effect on the quality of hardened Portland cement concrete, less is known about the sulfate resistance of cement stabilized soils. The resistance to sulfate attack differs for cement-treated, coarse-grained and fine-grained soils, and is a function of sulfate concentrations. Sulfate-clay reactions can cause deterioration of fine-grained soil-cement. On the other hand, granular soil-cements do not appear susceptible to sulfate attack. In some cases, the presence of small amounts of sulfate in the soil at the time of mixing with the cement may even be beneficial. The use of sulfate-resistant cement may not improve the resistance of clay-bearing soils, but may be effective in granular soil-cements exposed to adjacent soils and/or groundwater containing high sulfate concentrations. The use of cement for fine-grained soils containing more than about 1% sulfate should be avoided.

Stabilization with Lime-Flyash (LF) and Lime-Cement-Flyash (LCF)

Stabilization of coarse-grained soils having little or no fines can often be accomplished by the use of LF or LCF combinations. Flyash, also termed coal ash, is a mineral residual from the combustion of pulverized coal. It contains silicon and aluminum compounds that, when mixed with lime and water, forms a hardened cementitious mass capable of obtaining high compressive strengths. Lime and flyash in combination can often be used successfully in stabilizing granular materials, since the flyash provides an agent with which the lime can react. Thus LF or LCF stabilization is often appropriate for base and subbase course materials.

Flyash is classified according to the type of coal from which the ash was derived. Class C flyash is derived from the burning of lignite or subbituminous coal and is often referred to as “high lime” ash because it contains a high percentage of lime. Class C flyash is self-reactive or cementitious in the presence of water, in addition to being pozzolanic. Class F flyash is derived from the burning of anthracite or bituminous coal and is sometimes referred to as "low lime" ash. It requires the addition of lime to form a pozzolanic reaction. To be acceptable quality, flyash used for stabilization must meet the requirements indicated in ASTM C 593.

Design with LF is somewhat different from stabilization with lime or cement. For a given combination of materials (aggregate, flyash, and lime), a number of factors can be varied in the mix design process, such as percentage of lime-flyash, the moisture content, and the ratio of lime to flyash. It is generally recognized that engineering characteristics such as strength and durability are directly related to the quality of the matrix material. The matrix material is that part consisting of flyash, lime, and minus No. 4 aggregate fines. Basically, higher strength and improved durability are achievable when the matrix material is able to "float" the coarse aggregate particles. In effect, the fine size particles overfill the void spaces between the coarse aggregate particles. For each coarse aggregate material, there is a quantity of matrix required to effectively fill the available void spaces and to "float" the coarse aggregate particles. The quantity of matrix required for maximum dry density of the total mixture is referred to as the optimum fines content. In LF mixtures, it is recommended that the quantity of matrix be approximately 2% above the optimum fines content. At the recommended fines content, the strength development is also influenced by the ratio of lime to flyash. Adjustment of the lime-flyash ratio will yield different values of strength and durability properties.

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-bitumen, which produces strength in cohesionless soils, such as clean sands, or acts as a binder or cementing agent, 2) soil-bitumen, which stabilizes the moisture content of cohesive fine-grained soils, and 3) sand-gravel bitumen, which provides cohesive strength and waterproofs pit-run gravelly soils with inherent frictional strength. The durability of bitumen-stabilized mixtures generally can be assessed by measurement of their water absorption characteristics. Treatment of soils containing fines in excess of 20% is not recommended.

Stabilization of soils and aggregates with asphalt differs greatly from cement and lime stabilization. The basic mechanism involved in asphalt stabilization of fine-grained soils is a waterproofing phenomenon. Soil particles or soil agglomerates are coated with asphalt that prevents or slows the penetration of water that could normally result in a decrease in soil strength. In addition, asphalt stabilization can improve durability characteristics by making the soil resistant to the detrimental effects of water, such as volume. In noncohesive materials, such as sands and gravel, crushed gravel, and crushed stone, two basic mechanisms are active: waterproofing and adhesion. The asphalt coating on the cohesionless materials provides a membrane that prevents or hinders the penetration of water and thereby reduces the tendency of the material to lose strength in the presence of water. The second mechanism has been identified as adhesion. The aggregate particles adhere to the asphalt and the asphalt acts as a binder or cement. The cementing effect thus increases shear strength by increasing cohesion. Criteria for design of bituminous-stabilized soils and aggregates are based almost entirely on stability and gradation requirements. Freeze-thaw and wet-dry durability tests are not applicable for asphalt-stabilized mixtures.

There are three basic types of bituminous-stabilized soils, including

(1) *Sand bitumen.* A mixture of sand and bitumen in which the sand particles are cemented together to provide a material of increased stability.
(2) *Gravel or crushed aggregate bitumen.* A mixture of bitumen and a well-graded gravel or crushed aggregate that, after compaction, provides a highly stable waterproof mass of subbase or base course quality.
(3) *Bitumen lime.* A mixture of soil, lime, and bitumen that, after compaction, may exhibit the characteristics of any of the bitumen-treated materials indicated above. Lime is used with materials that have a high PI, *i.e.,* above 10.

Bituminous stabilization is generally accomplished using asphalt cement, cutback asphalt, or asphalt emulsions. The type of bitumen to be used depends upon the type of soil to be stabilized, method of construction, and weather conditions. In frost areas, the use of tar as a binder should be avoided because of its high temperature susceptibility. Asphalts are affected to a lesser extent by temperature changes, but a grade of asphalt suitable to the prevailing climate should be selected. As a general rule, the most satisfactory results are obtained when the most viscous liquid asphalt that can be readily mixed into the soil is used. For higher quality mixes in which a central plant is used, viscosity-grade asphalt cements should be used. Much bituminous stabilization is performed in-place, with the bitumen being applied directly on the soil or soil aggregate system, and the mixing and compaction operations being conducted immediately thereafter. For this type of construction, liquid asphalts, *i.e.,* cutbacks and emulsions, are used. Emulsions are preferred over cutbacks because of energy constraints and pollution control efforts. The specific type and grade of bitumen will depend on the characteristics of the aggregate, the type of construction equipment, and the climatic conditions. Generally, the following types of bituminous materials will be used for the soil gradation indicated:

(1) Open-graded aggregate.
   a. Rapid- and medium-curing liquid asphalts RC-250, RC-800, and MC-3000.
   b. Medium-setting asphalt emulsion MS-2 and CMS-2.

(2) Well-graded aggregate with little or no material passing the 0.075 mm (No. 200) sieve.
   a. Rapid and medium-curing liquid asphalts RC-250, RC-800, MC-250, and MC-800.
   b. Slow-curing liquid asphalts SC-250 and SC-800.
   c. Medium-setting and slow-setting asphalt emulsions MS-2, CMS-2, SS-1, and CSS-1.

(3) Aggregate with a considerable percentage of fine aggregates and material passing the 0.075 mm (No. 200) sieve.
   a. Medium-curing liquid asphalt MC-250 and MC-800.
   b. Slow-curing liquid asphalts SC-250 and SC-800
   c. Slow-setting asphalt emulsions SS-1, SS-01h, CSS-1, and CSS-lh.

The simplest type of bituminous stabilization is the application of liquid asphalt to the surface of an unbound aggregate road. For this type of operation, the slow- and medium-curing liquid asphalts SC-70, SC-250, MC-70, and MC-250 are used.

The recommended soil gradations for subgrade materials and base or subbase course materials are shown in Tables 7-19 and 7-20, respectively.

Table 7-19. Recommended gradations for bituminous-stabilized subgrade materials (Joint Departments of the Army and Air Force, 1994).

Sieve Size	Percent Passing
75-mm (3-in.)	100
4.75-mm (#4)	50-100
600-µm (#30)	38-100
75-µm (#200)	2-30

Table 7-20. Recommended gradations for bituminous-stabilized base and subbase materials (Joint Departments of the Army and Air Force, 1994).

Sieve Size	37.5 mm (1 ½ in.) Maximum	25 mm (1-in.) Maximum	19 mm (¾-in.) Maximum	12.7 mm (½-in.) Maximum
37.5-mm (1½-in.)	100	-	-	-
25-mm (l-in.)	8 4 ± 9	100	-	-
19-mm (¾-in.)	76 ± 9	83 ± 9	100	-
M-in	66 ± 9	73 ± 9	82 ± 9	100
9.5-mm (3/8-in.)	59 ± 9	64 ± 9	72 ± 9	83 ± 9
0.475-mm (#4)	45 ± 9	48 ± 9	54 ± 9	62 ± 9
2.36-mm (#8)	35 ± 9	36 ± 9	41 ± 9	47 ± 9
1.18-mm (#16)	27 ± 9	28 ± 9	32 ± 9	36 ± 9
600-µm (#30)	20 ± 9	21 ± 9	24 ± 9	28 ± 9
300-µm (#50)	14 ± 7	16 ± 7	17 ± 7	20 ± 7
150-µm (#100)	9 ± 5	11 ± 5	12 ± 5	14 ± 5
75-µm (#200)	5 ± 2	5 ± 2	5 ± 2	5 ± 2

Stabilization with Lime-Cement and Lime-Bitumen

The advantage of using combination stabilizers is that one of the stabilizers in the combination compensates for the lack of effectiveness of the other in treating a particular aspect or characteristic of a given soil. For instance, in clay areas devoid of base material, lime has been used jointly with other stabilizers, notably Portland cement or asphalt, to provide acceptable base courses. Since Portland cement or asphalt cannot be mixed successfully with plastic clays, the lime is added first to reduce the plasticity of the clay. While such stabilization practice might be more costly than the conventional single stabilizer methods, it may still prove to be economical in areas where base aggregate costs are high. Two combination stabilizers are considered in this section: lime-cement and lime-asphalt.

a) *Lime-cement.* Lime can be used as an initial additive with Portland cement, or as the primary stabilizer. The main purpose of lime is to improve workability characteristics, mainly by reducing the plasticity of the soil. The design approach is to add enough lime to improve workability and to reduce the plasticity index to acceptable levels. The design lime content is the minimum that achieves desired results. The design cement content is determined following procedures for cement-stabilized soils presented in Appendix F.

b) *Lime-asphalt.* Lime can be used as an initial additive with asphalt, or as the primary stabilizer. The main purpose of lime is to improve workability characteristics and to act as an anti-stripping agent. In the latter capacity, the lime acts to neutralize acidic chemicals in the soil or aggregate that tend to interfere with bonding of the asphalt. Generally, about 1 – 2% percent lime is all that is needed for this objective. Since asphalt is the primary stabilizer, the procedures for asphalt-stabilized materials, as presented Appendix F, should be followed.

Admixture Design

Design of admixtures takes on a similar process regardless of the admixture type. The following steps are generally followed and are generic to lime, cement, L-FA and L-C-FA, or asphalt admixtures.

Step 1. Classify soil to be stabilized.
(% < 0.075 mm – No. 200 sieve, % < 0.425 mm – No. 40 Sieve, PI, etc.)

Step 2. Prepare trial mixes with varying % content.
Lime: Select lowest % with pH = 12.4 in 1 hour
Cement: Use table to estimate cement content requirements
Asphalt: Use equation & table in Appendix F to estimate the quantity of cutback asphalt

Step 3. Develop moisture-density relationship for initial design.

Step 4. Prepare triplicate samples and cure specimens at target density.
Use optimum water content and % initial admixture, +2% and +4%

Step 5. Determine index strength.
Lime and Cement: Determine unconfined compressive strength (ASTM D 5102)
Asphalt: Determine Marshall stability

Step 6. Determine resilient modulus for optimum percent admixture.
Perform test or estimate using correlations (See Chapter 5)

Step 7. Conduct freeze-thaw tests (Regional as required).
(For Cement, CFA, L-C-FA)

Step 8. Select % to achieve minimum design strength and F-T durability.

Step 9. Add 0.5 – 1% to compensate for non-uniform mixing.

Appendix F provides specific design requirements and design step details for each type of admixture reviewed in this section. Additional design and construction information can also be obtained from industry publications including

- *Soil-Cement Construction Handbook*, Portland Cement Association, Skokie, Il, 1995.
- *Lime-Treated Soil Construction Manual: Lime Stabilization & Lime Modification,* National Lime Associaiton, Arlington, Virginia, 2004.
- *Flexible Pavement Manual*, American Coal Ash Association, Washington, D.C., 1991.
- *A Basic Emulsion Manual*, Asphalt Institute, Manual Series #19.
- http://www.cement.org/index.asp
- http://www.lime.org/

7.6.6 Soil Encapsulation

Soil encapsulation is a foundation improvement technique that has been used to protect moisture sensitive soils from large variations in moisture content. 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. This technique has been used by a number of states (*e.g.,* Texas and Wyoming) on selected projects to improve the foundations of higher volume roadways. It is more commonly used as a technique in Europe and in foundation or subbase layers for low-volume roadways, where the import of higher quality paving materials is restricted from a cost standpoint. More than 100 projects have been identified around the world, usually reporting success in controlling expansive soils (Steinberg, 1998).

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 stiffness (*i.e.,* 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 an 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. To form a complete encapsulation, the bottom membrane is brought up the sides and wrapped around the top, for an excavated section, or the top membrane is draped over the sides, for an embankment situation. 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 in areas over time 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 environmental effects model (EICM) to predict changes in moisture over time. Special design computations will be needed to restrict 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.

7.6.7 Lightweight Fill

When constructing pavements on soft soils, there is always a concern for settlement. For deeper deposits where shallow surface stabilization may not be effective, thicker granular aggregate as discussed in Section 7.3, may be effective for control deformation under wheel load, but would increase the concern for settlement. An alternate to replacement with aggregate would be to use lightweight fill.

The compacted unit density of most soil deposits consisting of sands, silts, or clays ranges from about 1,800 – 2,200 kg/m^3 (112 – 137 lbs/ft^3) Lightweight fill materials are available from the lower end of this range down to 12 kg/m^3 (0.75 lbs/ft^3). In many cases, the use of lighter weight materials on soft soils will likely result in both reduced settlement and increased stability. The worldwide interest and use of lightweight fill materials has led to the recent publication by the Permanent International Association of Road Congresses (PIARC) of an authoritative reference *"Lightweight Filling Materials"* in 1997.

Many types of lightweight fill materials have been used for roadway construction. Some of the more common lightweight fills are listed in Table 7-21. There is a wide range in density of the lightweight fill materials, but all have a density less than conventional soils. Addtional information on the composition and sources of the lightweight fill materials listed in Table 7-21 can be found in FHWA NHI-04-001 Ground Improvement Methods technical summaries.

Some lightweight fill materials have been used for decades, while others are relatively recent developments. Wood fiber has been used for many years by timber companies for roadways crossing peat bogs and low-lying land, as well as for repair of slide zones.

The steel-making companies have produced slag since the start of the iron and steel making industry. Initially, the slag were stockpiled as waste materials, but beginning around 1950, the slag were crushed, graded, and sold for fill materials.

Geofoam is a generic term used to describe any foam material used in a geotechnical application. Geofoam includes expanded polystyrene (EPS), extruded polystyrene (XPS), and glassfoam (cellular glass). Geofoam was initially developed for insulation material to prevent frost from penetrating soils. The initial use for this purpose was in Scandinavia and North America in the early 1960s. In 1972, the use of geofoam was extended as a lightweight fill for a project in Norway.

The technique of using pumping equipment to inject foaming agents into concrete was developed in the late 1930s. Little is known about the early uses of this product. However, the U.S. Army Corps of Engineers used foamed concrete as a tunnel lining and annular fill. This product is generally job-produced as a cement/water slurry with preformed foam blended for accurate control and immediate placement.

Table 7-21. Densities and approximate costs for various lightweight fill materials.

Fill Type	Range in Density kg/m³	Range in Specific Gravity	Approximate Cost[1] \$/m³
Geofoam (EPS)	12 to 32	0.01 to .03	40.00 to 85.00[2]
Foamed Concrete	320 to 970	0.3 to 0.8	40.00 to 55.00
Wood Fiber	550 to 960	0.6 to 1.0	12.00 to 20.00[2]
Shredded Tires	600 to 900	0.6 to 0.9	20.00 to 30.00[2]
Expanded Shale And Clay	600 to 1040	0.6 to 1.0	40.00 to 55.00[3]
Flyash	1120 to 1440	1.1 to 1.4	15.00 to 21.00[3]
Boiler Slag	1000 to 1750	1.0 to 1.8	3.00 to 4.00[3]
Air-Cooled Slag	1100 to 1500	1.1 to 1.5	7.50 to 9.00[3]

[1] See Chapter 6 for details on cost data
[2] Price includes transportation and placement cost
[3] FOB plant

Shredded tires and tire bales are a relatively recent source of lightweight fill materials. The availability of this material is increasing each year, and its use as a lightweight fill is further promoted by the need to dispose of tires. In most locations, the tires are stockpiled, but they are unsightly and present a serious fire and health hazard. Shredded tires have been used for lightweight fill in the United States and in other countries since the mid 1980s. More than 85 fills using shredded tires as a lightweight fill have been constructed in the United States. In 1995, three tire shred fills with a thickness greater than 8 m (26 ft) experienced an unexpected internal heating reaction. As a result, FHWA issued an Interim Guideline to minimize internal heating of tire shred fills in 1997, limiting tire shred layers to 3 m (9.8 ft).

Expanded shale lightweight aggregate has been used for decades to produce aggregate for concrete and masonry units. Beginning in about 1980, lightweight aggregates have also been used for geotechnical purposes. Completed projects include the Port of Albany, New York marine terminal, where lightweight fill was used behind a bulkhead to reduce the lateral pressures on the steel sheeting. Other projects include construction of roadways over soft ground. The existing high-density soils were partially removed and replaced with lightweight aggregate to reduce settlement. Other projects have included improvement of slope stability by reduction of the gravitational driving force of the soil in the slope and replacement with a lightweight fill.

Waste products from coal burning include flyash and boiler slag. Both of these materials have been used in roadway construction. One of the first documented uses of flyash in an engineered highway embankment occurred in England in 1950. Trial embankments led to the acceptance of flyash fills, and other roadway projects were constructed in other European countries. In 1965, a flyash roadway embankment was constructed in Illinois. In 1984, a project survey found that flyash was used in the construction of 33 embankments and 31 area fills. Boiler slag has been used for backfill since the early 1970s. Many state highway department specifications allow the use of boiler slag as an acceptable fine or coarse aggregate.

The FHWA NHI-04-001 provides an overview of the more common lightweight fill materials that have been used for geotechnical applications in highway construction. Typical geotechnical engineering parameters that are important for design are provided. In addition, design and construction considerations unique to each of these lightweight fill materials are presented. This information can be used for preliminary planning purposes. The technical summary also presents guidelines for preparation of specifications along with suggested construction control procedures. Four case histories are also presented to demonstrate the effectiveness of lightweight fills for specific situations. Approximate costs for the various lightweight fill materials are also presented.

With regard to pavement design, if a minimum of 1 m (3 ft) of good quality gravel type fill is placed between the pavement structure and the lightweight materials as a cover, then the lightweight material will have little impact on pavement design, even for the more compressible tire and geofoam materials. However, if a thinner cover must be used, the support value for these materials must be determined. Lab tests can be used, as discussed in Chapter 5, especially for the granular type materials. The ideal method is to perform field resilient modulus tests on placed material (*i.e.,* on cover soils after placement over the lightweight material(s)), especially for the bulkier materials, such as tires and geofoam.

7.6.8 Deep Foundations and Other Foundation Improvement Methods (from Elias et al., 2004)

In some cases, the extent (area and depth) of poor subgrade conditions are too large for surface stabilization or removal. In extreme cases, the soils may be too week to support the roadway embankment (even for embankments that only consist of the pavement structure). In these cases, other deep ground improvement methods, such as deep foundations, may be required. Ground improvement technologies are geotechnical construction methods used to alter and improve poor ground conditions so that embankment and structure construction can meet project performance requirements where soil replacement is not feasible for environmental or technical reasons, or it is too costly.

Ground improvement has one or more than one of the following main functions:

- to increase bearing capacity, shear or frictional strength,
- to increase density,
- to control deformations,
- to accelerate consolidation,
- to decrease imposed loads,
- to provide lateral stability,
- to form seepage cutoffs or fill voids,
- to increase resistance to liquefaction and,
- to transfer embankment loads to more competent layers

There are three strategies available to accomplish the above functions representing different approaches. The first method is to increase the shear strength, density, and/or decrease the compressibility of the foundation soil. The second method is to utilize a lightweight fill embankment to reduce significantly the applied load to the foundation, and the third method is to transfer loads to a more competent deeper layer.

The selection of candidate ground improvement methods for any specific project follows a sequential process. The steps in the process include a sequence of evaluations that proceed from simple to more detailed, allowing a best method to emerge. The process is described as follows:

1) *Identify potential poor ground conditions, their extent, and type of negative impact*. Poor ground conditions are typically characterized by soft or loose foundation soils, which, under load, would cause long-term settlement, or cause construction or post-construction instability.
2) *Identify or establish performance requirements.* Performance requirements generally consist of deformation limits (horizontal and vertical), as well as some minimum factors of safety for stability. The available time for construction is also a performance requirement.
3) *Identify and assess any space or environmental constraints.* Space constraints typically refer to accessibility for construction equipment to operate safely, and environmental constraints may include the disposal of spoil (hazardous or not hazardous) and the effect of construction vibrations or noise.
4) *Assessment of subsurface conditions*. The type, depth, and extent of the poor soils must be considered, as well as the location of the ground-water table. It is further valuable to have at least a preliminary assessment of the shear strength and compressibility of the identified poor soils.
5) *Preliminary selection*. Preliminary selection of potentially applicable method(s) is generally made on a qualitative basis, taking into consideration the performance

criteria, limitations imposed by subsurface conditions, schedule and environmental constraints, and the level of improvement that is required. Table 7-22, which groups the available methods in six broad categories, can be used as a guide in this process to identify possible methods and eliminate those that by themselves, or in conjunction with other methods, cannot produce the desired performance.

6) *Preliminary design*. A preliminary design is developed for each method identified under "Preliminary selection" and a cost estimate prepared on the basis of data in Table 7-23. The guidance in developing preliminary designs is contained within each Technical Summary.
7) *Comparison and selection*. The selected methods are then compared, and a selection made by considering performance, constructability, cost, and other relevant project factors.

State-of-the-art design and construction methods and/or references are provided in each of the FHWA NHI-04-001 Ground Improvement Methods technical summaries to form the basis of a final design. The success of any ground improvement method is predicated on the implementation of a QA/QC program to verify that the desired foundation improvement level has been reached. These programs incorporate a combination of construction observations, in-situ testing and laboratory testing to evaluate the treated soil in the field. Details are provided in each technical summary contained in the FHWA NHI-04-001.

Table 7-22. Ground improvement categories, functions, methods and applications (Elias et al., 2004).

Category	Function	Methods	Comment
Consolidation	Accelerate consolidation, increase shear strength	(1) Wick drains (2) Vacuum consolidation	Viable for normally consolidated clays. Vacuum consolidation viable for very soft clays. Can achieve up to 90% consolidation in a few months.
Load Reduction	Reduce load on foundation, reduce settlement	(1) Geofoam, (2) Foamed concrete (3) Lightweight granular fills, tire chips, etc.	Density varies from 1 – 12 kN/m^3 (6 – 76 lb/ft^3). Granular fills usage subject to local availability.
Densification	Increase density, bearing capacity and frictional strength of granular soils. Decrease settlement and increase resistance to liquefaction.	(1) Vibro-compaction using vibrators (2) Dynamic compaction by falling weight impact	Vibrocompaction viable for clean sands with <15% fines. Dynamic compaction limited to depths of about 10 m (33 ft), but is applicable for a wider range of soils. Both methods can densify granular soils up to 80% Relative Density. Dynamic compaction generates vibrations for a considerable lateral distance.
Reinforcement	Internally reinforces fills and/or cuts. In soft foundation soils, increases shear strength, resistance to liquefaction and decreases compressibility.	(1) MSE retaining walls (2) Soil Nailing walls (3) Stone column to reinforce foundations	Soil Nailing may not applicable in soft clays or loose fills. Stone columns applicable in soft clay profiles to increase global shear strength and reduce settlement.
Chemical Stabilization by Deep Mixing Methods	Physio-chemical alteration of foundation soils to increase their tensile, compressive and shear strength, and to decrease settlement and/or provide lateral stability and or confinement.	(1) Wet mixing methods using primarily cement (2) Dry mixing methods using lime-cement	Applicable in soft to medium stiff clays for excavation support where the groundwater table must be maintained, or for foundation support where lateral restraint must be provided, or to increase global stability and decrease settlement. Requires significant QA/QC program for verification.

Table 7-22. Ground improvement categories, functions, methods and applications (continued).

Category	Function	Methods	Comment
Chemical Stabilization by Grouting	To form seepage cutoffs, fill voids, increase density, increase tensile and compressive strength	(1) Permeation grouting with particulate or chemical grouts (2) Compaction grouting (3) Jet grouting, and (4) Bulk filling	(1) Permeation grouting to increase shear strength or for seepage control, (2) compaction grouting for densification and (3) jet grouting to increase tensile and/or compressive strength of foundations, and (4) bulk filling of any subsurface voids.
Load Transfer	Transfer load to deeper bearing layer	Column (Pile) supported embankments on flexible geosynthetic mats	Applicable for deep soft soil profiles or where a tight schedule must be maintained. A variety of stiff or semi-stiff piles can be used.

Table 7-23a. Comparative Costs (SI units) (Elias et al., 2004).

Method	Unit Cost	Cost of Treated Volume $/m^3
Wick Drains	$ 1.50 - 4.00/m	$ 0.80 - 1.60
Lightweight Fill Granular Tires-Wood Geofoam Foamed Concrete	 $ 3.00 - 21.00/m^3 $ 12.00 - 30.00/m^3 $ 35.00 - 65.00/m^3 $ 45.00 - 65.00/m^3	
Vibrocompaction	$ 15.00 - 25.00/m	$ 1.00 - 4.00
Dynamic Compaction	$ 6.00 - 11.00/m^2	$ 1.00 - 2.00
MSE Walls	$ 160.00 - 300.00/m^2	
RSS Slopes	$ 110.00 - 260.00/m^2	
Soil Nail Walls	$ 400.00 - 600.00/m^2	
Stone Columns	$ 40.00 - 60.00/m	$ 50 - 75
Deep Soil Mixing Dry w/lime-cement Wet w/cement	 $30.00/m 	 $ 60 $ 85 -150
Grouting Permeation Compaction Jet	 $ 65.00/m + $ 0.70/Liter 	 $ 30 - 200 $ 200 - 275
Column-Supported Embankments	$ 95/m^2 + cost of column	n/a

Table 7-23b. Comparative Costs (U.S. customary units) (Elias et al., 2004).

Method	Unit Cost	Cost of Treated Volume $/yd³
Wick Drains	$ 0.46 - 1.22/ft	$ 0.60 - 1.20
Lightweight Fill Granular Tires-Wood Geofoam Foamed Concrete	 $ 2.30 - 16.10/yd³ $ 9.20 - 23.00/yd³ $ 26.75 - 50.00/yd³ $ 34.50 - 50.00/yd³	
Vibrocompaction	$ 4.60 - 7.60/ft	$ 0.75 – 3.00
Dynamic Compaction	$ 5.00 - 9.20/yd²	$ 0.75 - 1.50
MSE Walls	$ 15.00 - 28.00/ft²	
RSS Slopes	$ 10.00 - 24.00/ft²	
Soil Nail Walls	$ 37.00 - 56.00/ft²	
Stone Columns	$ 12.20 - 18.30/ft	$ 38 - 57
Deep Soil Mixing Dry w/lime-cement Wet w/cement	 $9.15/ft	 $ 46 $ 65 -115
Grouting Permeation Compaction Jet	 $ 20/ft + $ 2.65/Gallon	 $ 23 - 153 $ 150 - 210
Column Supported Embankments	$ 81.50/yd² + cost of column	n/a

7.7 RECYCLE

Recycling, in principal, is a very powerful and often political concept. While the benefits of recycling including conservation of aggregate and binders and preservation of the environment, it requires serious consideration. The long-term performance of recycled materials in pavements and, in come cases the environmental impact, must be carefully evaluated to avoid costly performance and maintenance issues. In this section, the evaluation requirements for recycled materials will be reviewed. There are two forms of recycling in pavements: 1) reuse of the pavement materials themselves and 2) the use of recycled waste materials for subgrade stabilization or as a substitute for aggregate.

7.7.1 Pavement Recycling

The method of recycling the pavement will, in most cases, depend on whether the surface pavement has an AC or PCC surface pavement. In either case, the material could be rubblized, or, in some cases, processed (*e.g.,* sieving, stockpiling, and reusing the reclaimed asphalt pavement (RCP) materials or recycled concrete materials (RCM) plus the aggregate base). Both pavement types can also be rubblized in place and compacted. This procedure is known as rubblize and roll for PCC pavements and full-depth reclamation for AC pavements. For AC pavement materials, there are also several other methods, including hot mix asphalt recycling, hot in-place recycling, and cold in-place recycling, all of which produce a bound product, which is beyond the scope of this manual.

Recycled Asphalt

The design requirements for RCP aggregates are essentially the same as natural aggregates. The strength of the material must be determined using the methods outlined in Chapter 5 and Section 7.3, and an assessment must be made of the drainage characteristics, as discussed in Section 7.2. With full-depth reclamation, all of the asphalt pavement sections and a predetemined amount of underlying materials are treated with recyling agents to produce a stabilized base course, and is well covered in FHWA-SA-98-042 (Kandhal, and Mallick, 1997) . The advantages of this process are establishing high production rate and maintaining the geometry of the pavement or shoulder reconstruction. The primary drawbacks are aggregate size, depth limitation and depth control, and need for specialized equipment. With the sizing, RAP can often only be effectively screened down to a maximum size of 50 mm (2 in.). If a significant amount of contaminated base course (*i.e.,* containing significant amount of fines) is removed with the asphalt, the hydraulic properties of the aggregate could also be poor.

Recycled Concrete

Again, the design requirements for RCM aggregates are essentially the same as natural aggregates. Recycled concrete has been used by a number of states as base materials since the 1980s. However, several states have identified three significant issues, including

- the formation of tufa (calcium deposits) clogging drains and filter materials;
- alkaline (high pH) run-off; and,
- freeze thaw degradation.

As a result, these states are now primarily using the recycled concrete, mixed with natural soils, as embankment fill.

7.7.2 Recycled Waste Materials

A number of recylced waste materials have been used in permanent construction, practically all of which where covered in Section 7.6.7 since they have a lighter weight than conventional aggregate. Other applications not reviewed in Section 7.6.7 include the use of recycled materials as a replacement for base materials (*e.g.,* slag and bottom ash) and, in some cases (*e.g.,* glass and tire shreds) drainage aggregate. As indicated in Section 7.6.7, the materials must be evaluated with respect to the same property requirements as the material they will replace. The pavement support value (*e.g.,* resilient modulus or CBR) should be determine based on lab tests reviewed in Chapter 5. Field trails using FWD tests to confirm the as constructed properties are also recommended. Durability is a critical issue with many of these materials, and, obviously, an assessment of environmental issues must be made.

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CHAPTER 8.0 CONSTRUCTION AND DESIGN VERIFICATION FOR UNBOUND PAVEMENT MATERIALS

8.1 INTRODUCTION

Thus far, this reference manual has described the processes detailing the pre-construction phase – site characterization and design. The last phase of work required to complete a roadway, the construction phase, is the emphasis of this chapter. As such, construction specifications are described, quality control/quality assurance concepts are reviewed, and innovative measures of design verification are provided. Issues surrounding the preparation of the pavement foundation, focused primarily upon cut and fill soil construction, prepared subgrade, chemically stabilized soils, and unbound engineered aggregate layers (base/subbase) are detailed. Lastly, a review of monitoring techniques for the finished product is provided, an important consideration with the current move toward performance specifications and warranties.

Projects come in all shapes and sizes, each presenting unique challenges. For instance, a new roadway alignment could be conceptualized in a flat topography, requiring little earthwork, or conversely, in a hilly or mountainous topography that requires large cuts – excavation and/or rock blasting – and placement of deep fills. In either case, the engineer typically must make do with the local soils and design for the site conditions encountered. Fundamental to the pavement construction is the preparation of the pavement foundation (*i.e.,* the subgrade) to meet the pavement design support requirements. The designer has made assumptions based on the subsurface exploration program as to the support conditions and the field requirements anticipated to meet those support conditions (*i.e.,* the anticipated adequacy of existing support after grading or modifications required to achieve that support). It is now up to construction to achieve these requirements. Proper treatment of the subgrade during construction will assure expedient construction of the pavement section, enhance pavement performance over its life, and ensure that the pavement design intent is carried through in the construction phase (Ohio DOT, 2002).

8.2 SPECIFICATIONS

The development of the specifications is usually conducted in the pre-construction phase as part of the design. The specification dictates the quality for the pavement section construction, with the intent of tying the design to the finished product (design intent ⇔ performance). There is no good practice other than what is specified in the contract.

Most agencies have developed as part of the construction process a set of standards and specifications. These documents may contain guideline specifications (*e.g., AASHTO Standard Specifications for Transportation Materials*, AASHTO, 2004) modified by the local agency to address local conditions, materials available, construction techniques commonly employed, and their local experience.

The specifications can generally be categorized as *method, result, or performance* specifications. An example of each type is provided below:

Method Specification – This form of specification describes in detail the equipment and procedures (process) used to achieve a desired result (*e.g.,* a compactive effort of 4 passes using a 35-ton sheepsfoot roller – Caterpillar C825 or equivalent – shall be made on each 8-inch lift of loose soil spread on the grade).

Result Specification – The result specification is normally shorter and easier to write than the method specification. This form of specification states what property must be achieved, allowing the contractor some liberty to innovate the process to satisfy the intended result (*e.g.,* a dry density of 95% of the maximum dry density – as determined by AASHTO T99, standard Proctor – shall be obtained for each lift of soil placed on the grade).

Performance Specification – A specification for key materials and construction quality characteristics that have been demonstrated to correlate significantly with long-term performance of the finished work (*e.g.,* the pavement shall support 1 million ESALs without developing fatigue cracks or rut depths exceeding 6 mm (0.25 in.)).

Performance specifications may be presented in one of three forms, including (after Chamberlin, 1995)

- *Performance specifications* – which directly define the condition of the road, the response of the road to load, and/or the condition of the pavement materials at a given period of time,
- *Performance-based specifications* – which describe desired levels of fundamental engineering properties that are predictors of performance; or
- *Performance-related specifications* - which describe the desired level of key materials and construction quality characteristics that have been demonstrated to correlate significantly with long-term performance of the finished work.

The main intent of each type of specification with respect to geotechnical factors is to confirm the adequacy and/or improve the engineering behavior of the soil or aggregate material by modification of moisture content and densification, or compaction, of the soil or aggregate. While the *result specification* is more common, the *method specification* can be utilized where the result is probable based on local experiences, or where the result is difficult to measure (*i.e.,* density of coarse rock fill). This form of specification takes responsibility away from the contractor and places it on the shoulders of the owner and his engineer. The *result specification* will typically encourage the contractor to utilize the most efficient and economical means to achieve the requirements.

Pavement *performance specifications* may be appropriate for design/build and warranty contracts. However, it is obvious that the above pavement *performance specifications* cannot be used to control the quality of aggregate or subgrade materials used in the construction. The pavement is an interdependent layered system consisting of different materials, all of which affect performance. During the service life of the pavement, the material properties can change from those measured during construction. The performance required by the example above is also affected by the thickness of the layers, which is a design element. The main challenge with performance specifications is the determination of performance measures, as discussed later in Section 8.5.

No specification type can cover all situations, and each type has relevance depending on the circumstance (*e.g.,* Design Build or Design-Build-Let contracting methods). The specification, regardless of whether *method*, *result,* or *performance*, should emphasis material properties of raw materials (soil classification, limits for maximum particle size, grain size distribution, Atterberg limits, and other properties typically used for aggregates in base or subbase layers, such as toughness (durability) and soundness, among others).

Each specification type should contain a provision for corrective action measures to be taken when unsuitable conditions (*i.e.,* weak, soft, wet, yielding materials) are encountered. The corrective measures should include

- method of detection (proofroll, QC/QA test, etc.),
- depth of anticipated treatment,
- type of treatment (drainage, undercut and replace, installation of geosynthetics, chemical modification/stabilization), and
- quick resolution determining whose responsibility (pay item) it is to implement the corrective measure (Owner or Contractor).

Once established, site preparation, excavation, hauling, placing, compaction, and grading objectives can commence.

8.3 QUALITY CONTROL AND QUALITY ASSURANCE

Good quality control/quality assurance (QC/QA) practices are essential to obtain satisfactory results in a construction project. QC/QA can be a single plan developed by the Owner to review the construction process. A third party or the agency often performs the quality control (QC) field observations. Alternatively, the quality control (QC) may refer to a written plan submitted by the contractor, which is reviewed and approved by the owner/engineer. This document clearly demonstrates how the contractor will control the processes used to produce or purchase materials used in construction, as well as control the processes for proper installation in order to meet the requirements set forth by the owner/engineer. The QC Plan will typically include tests (QC tests) performed on the materials intended for use at a prescribed frequency, as summarized in Table 8-1, as well as tests to indicate that the intent of the specification is being satisfied (field compaction monitoring and control, again at a prescribed frequency). Quality assurance (QA) is documentation that the contractor is following the QC Plan, and most likely will consist of some random inspections and testing to verify QC observations and results.

Table 8-1. Typical material properties measured for construction.

Test	Frequency
Material Source(s) [1]	
Classification	1 per material type
Atterberg Limits	1 per material type
Grain Size	1 per material type
Moisture-Density (Proctor)	1 per material type
Abrasion [2]	1 per material type
Soundness [2]	1 per material type
Field Installation	
Moisture Content	per QC Plan [3]
Density	per QC Plan [3]
Stiffness Assessment (*e.g.,* proof rolling)	per QC Plan [3]

NOTES:
1: different natural (in-situ or borrow) soil or quarry aggregate.
2: values typically required for quarry aggregate used as base.
3: frequency intervals identified in the QC Plan.

8.3.1 QC/QA: Tradition Methods

The construction specification establishes the framework for QC/QA. With a *method specification*, the quality control (QC) individual would document the equipment utilized and continuously monitor its activities during operation. The assurance may be by certification of QC tests and reports along with intermittent inspection. With a *result specification*, the QC individual would perform frequent testing at the start of the process, testing for changed conditions, and some testing for verification. The assurance testing would typically be a prescribed number of tests for a specific quantity of materials at random locations. Statistical processing of the test data may be used to determine the amount of payment if pay factors are included in the contract. A good practice for quality control is the development and use of a checklist for monitoring and inspecting the construction of the pavement system, similar to the one shown in Table 8-2.

Initial observations include confirming that clearing and grubbing operations have been adequately accomplished and that the prepared surface is suitable for placement of embankment/fill. The "suitability" is often confirmed through proof rolling.

Proof Rolling

The objective of proof rolling is to point out soft or yielding material. The technique can be implemented at any point during construction of the embankment, preparation of the subgrade (top 300 mm (12 in.)), and completion of base and/or subbase layers. In fact, as described later in Section 8.4, proof rolling observations can be made as material is being excavated, hauled, placed, and compacted using the equipment used to perform each of these tasks.

Many agencies have developed vehicle configuration specifications, including weight and tire pressures, for performing proof rolling operations, and have established a policy on methodology and threshold criteria for acceptable deflections, as well as those requiring remediation. For example, Ohio uses proof rolling for all projects types (new, rehabilitation, and reconstruction). This practice is good for detecting soft zones that may have passed the density requirements of the project, but not necessarily the moisture content, and can detect problems that could extend many feet below the tested surface. Once detected, seasoned experience can often estimate the depth of probable weakness; however, penetration rods and hand augers can be used with more objectivity than the eye guestimate. Once detected and properly delineated (aerial extent and depth), remediation actions are typically employed (remove/undercut and replace, installation of underdrains, installation of geosynthetics, chemical stabilization) that best suit the conditions encountered. The remediation alternative selected typically is a result of a cost or schedule constraint. Many agencies have reported historically large change order work dealing with soft subgrade, and have subsequently included likely remediation alternatives in the bidding process to establish a competitive rate for this work.

Table 8-2. Field monitoring checklist.

1. Read the specifications and become familiar with - site preparation - material requirements - construction procedures - grade/slope requirements - drainage requirements - tolerances of each of above requirements - testing requirements - acceptance/rejection criteria
2. Review the construction plans and become familiar with - lines, grades, and layer thickness requirements - temporary and permanent drainage features, locations, and details - details for utility construction, special requirements - demolition (if rehabilitation or reconstruction project) - corrective action requirements for weak, yielding, unstable materials (undercut/replace, chemical stabilization, geosynthetics) - construction sequence
3. Review material requirements, equipment requirements, and approved submittals.
4. Check site conditions. Observe - clearing, grubbing requirements, and activities – document the final condition in accordance with the specification - haul patterns - response to load (deflection under heavy equipment traffic) - assess need for drainage system - perform foundation acceptance – testing as required (*e.g.,* proof rolling, DCP, etc.)
5. On site monitoring and testing. - observe and document hauling, placement, and spreading adequacy (segregation, loose lift thickness, moisture content reasonableness) - review truck tickets and ensure that material sources are from approved sources (generally for engineered products such as aggregate base or flowable fill, bedding material, and off-site borrow, etc.) - randomly sample for material compliance - assess for moisture content reasonableness (not overly wet or dry in conformance with the specification) - document compaction efforts (size and type of equipment, # of passes) - test for compaction (determine conformance to specifications with respect to density and moisture – Proctor check point, if necessary) and assess stability - non-conformance corrective action – proof roll and assess extent/depth of affected area; determine and document responsibility (owner vs. G.C.) and begin corrective action - measure and document stabilizing agent type and rate of application, mixing adequacy and depth, compactive effort, moisture reasonableness and compaction compliance results (for stabilization/modification layers or treatments). Sampling and testing of treated layers and determination of strength in the laboratory may also be required.

Tests

Test methods used for in-place quality control and acceptance of individual flexible pavement layers and of new and rehabilitated flexible pavement systems have changed little in past decades. Such quality control and acceptance operations typically rely on nuclear density measurements (Figure 8-1), sand cone, balloon or drive tube methods, and the results of moisture content determined by a variety of methods, including nuclear gauge, speedy moisture, and hot plate or oven drying. These tests are typically performed on embankment construction (fill soils), finished subgrade, and unbound base layers, while some are applicable to measuring the quality of chemically stabilized materials, as described later in this section.

The old school of thought used compaction testing to calibrate construction methods. After the methods were calibrated, observation became as important as testing for quality control. Samples were taken at select locations based on observations. Today there is more of an emphasis on statistical characterization of constructed materials. Sample locations have become more random. Quality assurance specifications often give the contractor the responsibility of sampling and testing for process control. Testing by the owner includes some verification of the contractor's test results, and testing for acceptance and payment. The amount of payment may be determined by the statistical evaluation of test values resulting in pay factors (and no test reports, no pay).

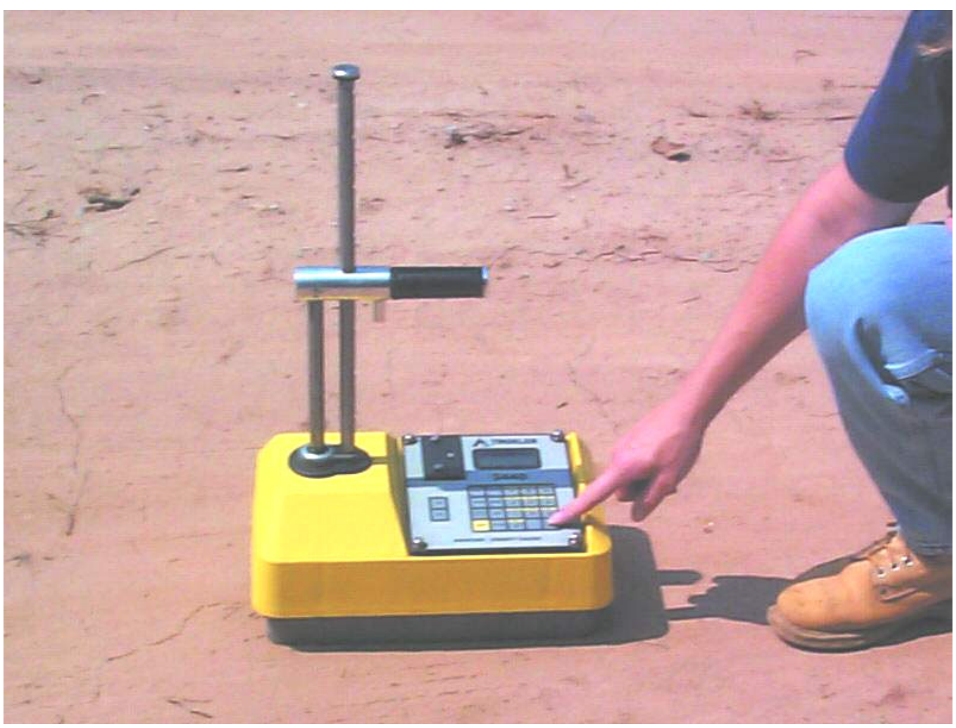

Figure 8-1. Nuclear densometer (*photo courtesy of Troxler*).

As part of a good QC Plan, process control and measurement of the control can be a valuable tool. A test result, or trend of a measured value, may not directly demonstrate compliance or non-compliance, but tracking the measured value over time may help explain why another process is out of compliance. In the following example, a soil with a standard proctor maximum dry density of 15.7 kN/m^3 (100 pcf) at an optimum moisture content of 20% is being placed in a single lift along a 300-m (1,000-foot) length of roadway embankment. The specification requires that a minimum of 95% of the maximum density be obtained, at or near the optimum moisture content. In order to simplify this example, it is assumed that the material is uniform in classification, and is being hauled by scrapers from a cut zone nearby. QC tests have been recorded and are graphically shown in Figure 8-2. This figure illustrates that the density is adequate along the first 120 m (400 feet) of placement, then trends toward an ‘out-of-tolerance’ or ‘out-of-control’ situation. The QC Plan may prompt the contractor to exert more compactive effort on this ‘out-of-control’ area, or change compactors; however, the moisture data suggests that the moisture content may be the ‘out-of-control’ parameter, which is, in turn, causing the density to move ‘out-of-control’. By recognizing what part of the process is defective, the contractor begins spreading the cut soil in thin lifts, allows some drying to occur prior to compacting, and again returns to a product considered satisfactory at the 250-m (820-foot) mark.

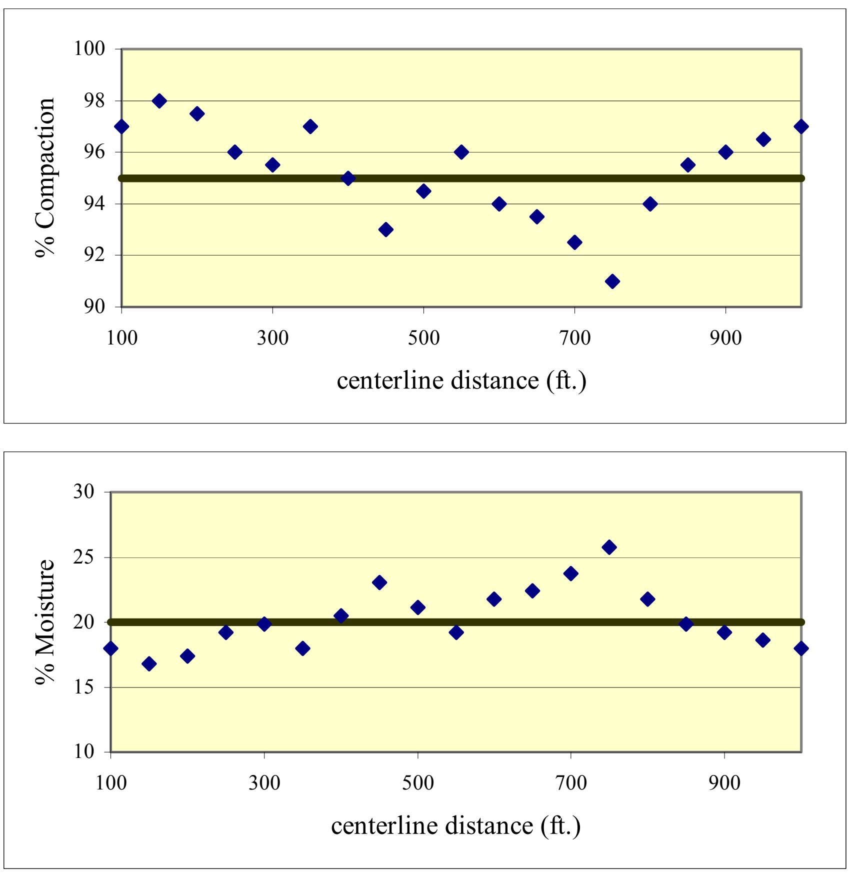

Figure 8-2. Process control, field density and moisture. (3.3 ft = 1 m)

This form of process control (density or degree of compaction) is applicable to embankment, subgrade, and unbound aggregate base construction. In addition, gradation of aggregate base materials is an important process control measurement. Depending on location of sampling (on grade, from haul trucks, from stockpiles) segregation and contamination may be detected using this measure. It is also an important measure used to ensure that the quarry process (crushing and grading) is in control.

Chemical Stabilization

In addition to density and moisture measurements, several additional controls are required and considered good practice when stabilization techniques are employed. An excellent source for QC/QA requirements can be found in the Soil Stabilization for Pavements manual (Army, Air Force, 1994). Briefly, these elements include

- Pulverization and scarification – An assessment of the material to be treated is required, and generally includes random sampling and testing using a field sieve (25-mm (1-in.) and 4.75-mm (No. 4) sieves).
- Stabilizing agent content – The amount of modifier added to the soil should be measured, and generally includes sampling the discharge using a canvas of known area placed on the soil to be treated, or calculating the area over which a known tonnage has been spread. For lime stabilization, pH can provide a good indication that the correct dosage has been achieved, as discussed in Appendix E.
- Uniformity of mixing – Visual observation is made to ensure that uniform mixing has been accomplished throughout the full depth of treatment. The use of phenolphthalein indicator solution has been used effectively for lime treatments to indicate depth of treatment. This solution – a light spray applied to the sides of a hand-excavated hole in the treated soil – will react with the lime, turning a brilliant pink color.
- Compaction and moisture control – Covered in the previous section.
- Curing – Curing is essential to assure that the modified soil mixture will achieve the final properties desired. The use of moist curing (light sprinkling of water) or membrane curing (application of a bituminous coating) is common. Regardless of method, the entire area must be properly sealed, and documentation of this activity is required.

Again, most QC programs only measure the compaction of the earthwork operation. While this methodology is valuable, density does not necessarily translate into performance. This type of QC testing is still very important in that it provides for a uniformity in the contractor's work, and can control the moisture of a given soil type at or near its optimum moisture content. This moisture control is important in order to minimize volume change characteristics, and none of the technologies described above have this capability.

8.3.2 QC/QA: Emerging Technologies

Lost in most projects requiring earthwork operations is the *design intent*. While the industry has accrued decades of experience founded upon successful engineered and constructed projects, we still observe the occasional premature failure that could have possibly been avoided. Reviewing, site soils are improved for three reasons:

- there are large quantities available;
- the natural state is inadequate to support the intended structure; and
- it is cost-effective.

For embankment fills, shear strength (slope stability) and consolidation (settlement potential) are used in the engineering analyses for *design intent*. As the elevation gets closer to the final grade (below the granular layers and surface layers – or within the anticipated depth or zone of repeated loading influence) resistance to deformation – siffness or resilient modulus – is used in the engineering analyses for *design intent*. These parameters are measured in the laboratory on soils sampled in the soil exploratory phase of the design. In essence, the design parameters are rationalized as an anticipated, educated estimate of what will finally be obtained in the field. But rather than measure these design parameters in the field, we commonly accept the contractor's work based on a *result specification* measurement of density. While this may be common and traditional, it does not verify if the *design intent* has been met.

As an example, the following scenario represents a hypothetical design situation where a 0.4-km (¼ -mile) roadway, 2 lanes wide is to be constructed along a gently rolling topography. Based on soil borings drilled and samples obtained, classified and tested, Table 8-3 was compiled for five uniquely different soils (based on visual description and engineering classification) ranging from a sandy soil to a highly plastic clay. The five samples are identified as Samples A-E.

In this hypothetical example, it is assumed that each of these materials will be equally represented along the length and width of the project, and are semi-infinite in depth. Further, the design assumes each material will be field compacted to 95% of the soil maximum dry density at or very near its optimum moisture content. Lastly, it is assumed that the soil mass will remain at this condition for the performance period. Each of these assumptions is idealistic, not realistic. However, it can be demonstrated that the *design intent* for this hypothetical example can be verified in the field during construction. In this example (Boudreau, 2003), the *design intent* is a roadbed stiffness of 20 Mpa (7200 psi) based on a stress state of 36 kPa (5.3 psi) vertical and 11 kPa (1.6 psi) horizontal (as estimated by the pooled subgrade constitutive model $M_r = 9041\sigma_v^{-0.19526}\sigma_h^{0.19643}$ with σ_v, σ_h in English units).

Table 8.3. Summary of design soil properties for example problem (pre-construction).

Physical Property		A	B	C	D	E
Liquid Limit, LL		21	NL	35	64	36
Plastic Limit, PL		16	NP	14	29	27
Plasticity Index, PI		5	---	21	35	9
P_4 (%)		94	100	100	100	100
P_{10} (%)		92	100	100	96	96
P_{200} (%)		47	20	59	82	48
Maximum Dry Density,[1] γ_{max}	(kN/m^3)	18.8	18.2	16.9	14.9	17.8
	(pcf)	(119.8)	(115.9)	(107.8)	(94.7)	(113.3)
Optimum Moisture Content, w_{opt} (%)[1]		12.0	11.8	17.2	25.6	15.0
AASHTO Classification		A-4	A-2-4(0)	A-6(9)	A-7-6(32)	A-4
Unified Soil Classification		SC	SM	CL	CH	SC
Resilient Modulus Parameters[2]	K1	10 387	6246	10 274	10 362	7938
K2		-0.015483	-0.00836	-0.41797	-0.18345	-0.21171
K3		0.23229	0.30028	0.08425	0.12762	0.23770

1. Maximum dry density and optimum moisture content, as determined by AASHTO T-99 (standard Proctor).
2. For modulus equation: $M_r = K1S_V^{K2}S_3^{K3}$ with S_V and S_3 in English units. Laboratory test specimens prepared to 95% of maximum dry density at optimum moisture content (as determined by AASHTO T99).

Based on the information provided, a conventional pavement cross section resulting from the subgrade support conditions, determined from the pre-construction laboratory test program summarized in the table above (analyses performed per AASHTO 1993 Design Guide using estimates for traffic and other inputs per the Boudreau reference cited) is 140 mm (5.5 in.) of asphalt concrete on 200 mm (8 in.) of a crushed aggregate base. The question is, if the contractor satisfies the *result specification* for subgrade construction - for this example, the contractor must achieve 95 percent compaction at or very near optimum moisture content - and the layers above (140 mm (5.5 in.) of asphalt concrete and 200 mm (8 in.) of a crushed aggregate base) are constructed with approved materials and constructed to satisfy the *result specification* for these layers, will the pavement perform as designed and expected? The answer deserves some examination.

First, the *design intent* for the subgrade is stiffness and strength; the measure of acceptance is density. Fundamentally, these two measures are uniquely different; one measure does not necessarily confirm the other measure. It is possible that the contractor has met the compaction specification on the wet side of optimum. The important measure is one of stiffness – in this hypothetical example, 50 MPa (7200 psi).

There are a number of ways to more precisely measure whether the *design intent* has been satisfied. These measures could include field CBR and plate load tests, dynamic cone penetration (DCP) tests, correlation studies, and/or laboratory tests performed on undisturbed tube samples obtained at finished grade. More recently, nondestructive testing (NDT) methods, including lasers, ground-penetrating radar, falling weight deflectometers (FWD), mini or portable lightweight FWD (LWD) cone penetrometers, GeoGauge (providing direct stiffness measurements), and infrared and seismic technologies, have been significantly improved and have shown potential for use in the quality control and acceptance of flexible pavement construction. As mentioned in Section 8-4, another technology in development consists of instrumented compaction equipment. This and the others mentioned above require field verification studies prior to any endorsement of the technology. The thrust of NCHRP Project 10-65 is to explore many of these technologies for this specific application (Von Quintus et al., 2004). It is anticipated that some of these techniques will eventually be incorporated into *performance specifications* as the industry gains more knowledge and accrues more experience with them. Many of these techniques were briefly reviewed in Chapter 4 (see Tables 4-2 through 4-6) and are described in greater detail below.

With the advent of the much anticipated NCHRP 1-37A Pavement Design Guide and extended warranty period of performance, there is an ever increasing need to measure layer stiffness properties by owner agencies, an activity that is not presently a typical component in the acceptance of a completed project.

Proof Rolling

A practical approach that many agencies use is the concept of proof rolling, as discussed in the Section 8.3.1. Although this approach is observer-dependant, many agencies use the technique not to measure *design intent* (deformations anticipated at stress levels typical under repeated load traffic protected by layers of material would result in deformations undetectable to the human eye during a proof rolling exercise), but to evaluate gross deficiencies including soft, yielding, or pumping subgrade. The objective of this type of process is to correct problem areas prior to the placement and compaction of stronger, more expensive materials (these soft zones will surely be detected during finishing operations of the stronger layer materials in the form of roller cracks).

With newer more sophisticated technology, including lasers, digital video, and image analysis, it is possible to take proof rolling to a new level of direct stiffness measurements. Small deformations can now be monitored as the proof roller moves across the site. Although this is somewhat of a research topic at this time, the concept is fairly straightforward to develop. In fact, Wisconsin DOT has developed a prototype deflection measurement system for use with a loaded dump truck, using ultrasonic sensors and a micro-controller, in order to continuously and objectively proof roll subgrade soils. Wisconsin DOT concluded that a threshold value of 38 mm (1.5 in.) of deflection indicated "failed" areas that required corrective action, and also found value in analyzing the ratio of the 0-offset and 0.6 m-offset (24 in.) sensors to determine depth of weakened zones (Wisconsin DOT, 2002).

Field CBR or Plate Load Tests

These technologies were developed several years ago and were employed as a measure for verifying *design intent*. Each included mobilization of equipment (moderate to heavy plates, loading rams, calibrated proving rings or load cells, and dial indicators or electronic deflection measurement devices) crew and heavy reaction vehicle (typically readily available on an earthwork construction project in the form of a scraper or track-mounted excavator/shovel). These tests are often the standard for quality programs in Europe, but have not typically been utilized in the U.S., based on their relative cost, time involved to set up and perform the test at a specified location, and accuracy issues. The field CBR test could measure the in-situ CBR of finished subgrade in order to verify *design intent* for flexible pavements, and the plate load could directly measure the modulus of subgrade reaction, or k-value, for rigid pavements. The plate load test is the standard practice in Europe for all pavement types.

Each of these measures is time-consuming; thus, only a few locations could be tested per day, oftentimes impeding the earthwork contractor's progress. In cut zones, these tests measure soil properties that are not controlled by the contractor, thus it is often difficult to expect the contractor to achieve a predetermined CBR or k-value threshold without paying for corrective measures.

It is noted that this type of testing is common and traditional in many European countries, using special customized equipment to make this type of testing more automated and more productive than that described above.

Dynamic Cone Penetration (DCP) Tests

The DCP technology consists of a steel shaft with an instrumented penetration head conforming to a precise configuration, as was described in Table 4-9. The instrumentation is capable of measuring resistance per increment of advancement and used with correlations to estimate stiffness of the materials. The benefits of this form of measurement are that the device can be quickly and efficiently mobilized to the project site (can be hand-carried or mounted inside a vehicle) and can measure to depths beyond surficial soils. The drawbacks include discrete point evaluations – leaving zones between points unknown, and the fact that the information gathered is used to correlate stiffness and strength. Thus, is only as accurate as the correlation models used. However, an added value is that the DCP can readily indicate soil support via correlations for construction activities. For example, if the estimated in-situ CBR has a value of 6 or less, the soils are expected to rut and deflect under construction operations. If the estimated CBR values are between 6 and 8, the soils are considered marginally suitable for construction support (Illinois DOT, 1982).

Resilient Modulus Testing

The design for the example introduced at the beginning of this section was based upon several soil samples characterized in the laboratory. The soils selected for characterization were those anticipated in the uppermost zones of the finished subgrade. In order to verify *design intent*, it would seem logical that samples of the earthwork contractor's finished work be sampled and characterized in a similar fashion. This can be accomplished by extending short Shelby tubes into the compacted soil and returning to the laboratory for resilient modulus testing. The testing can occur on extruded tube samples the same day they are obtained from the field. Thus, final reporting can be available the next day. This form of sampling and testing has the benefit of comparing actual results with those used for design purposes. Additionally, these measures are direct; therefore they are not reliant upon correlations. Lastly, because these samples are physical in nature, the density and moisture content will be measured and can be compared with the QC test results for accuracy of the QC testing program.

Falling Weight Deflectometer (FWD) Tests

A very mobile device and one that can be utilized to examine the stress dependency of the embankment or roadbed soils, a falling weight deflectometer is basically a trailer-mounted piece of equipment, which drops a weight transmitted through a hard rubber-type pad to the surface (as covered in detail in Table 4-2). The van pulling the trailer is equipped with a computer data acquisition system that measures the load and offset surface deflections. For field control, there are also portable or lightweight LWD units (as shown in Figure 8-3), allowing an individual to carry the unit around in any vehicle.

This technology, with sophisticated computer models, can directly measure the roadbed deflection from which modulus values can be estimated in order to verify *design intent*. The device is relatively quick (less than 4 minutes is required per location to measure the properties), thus numerous locations can be measured per hour. There are also several new developments with units mounted on sleds or skis such that continuous coverage along the length of a project is possible.

Figure 8-3. Lightweight deflectometer (*photo courtesy of Dynatest*).

GeoGauge

A recent development is the GeoGauge, a lightweight unit capable of measuring stiffness at discrete points. The Federal Highway Administration is currently evaluating this technology in the form of a Pooled-Fund Study. This device has many perceived benefits, including the capability to measure the stiffness of a composite soil mass directly and quickly such that numerous discrete points can be evaluated per hour.

Seismic Methods

The Portable Seismic Pavement Analyzer (PSPA) and a derivative modified for base and subgrade measurement, the Dirt Seismic Pavement Analyzer (DSPA), are currently being used on a trial basis by the Texas Department of Transportation for QC/QA purposes (Nazarian, 2002). The operating principal of the PSPA is based on generating and detecting stress waves in a medium. If used appropriately, analyses of the stress waves can be used to determine the modulus of the layered material, as well as assess the thickness of the layer (aggregate base). These techniques are being utilized with very promising results during construction on a few projects in Texas, and are being considered for quality control on pavement warranty projects in Texas and New Mexico. This method of measurement and analysis are very similar to the principals used in spectral analysis of surface waves (SASW, Table 4-6).

Automatic Controlled Variable Roller Compaction and Documentation System (Intelligent Compaction)

While each of the aforementioned techniques have perceived advantages and disadvantages, none of the techniques described above has the capability to continuously evaluate *design intent* along the entire length of a project. The use of instrumented compaction equipment would appear to have some potential for continuously monitoring conditions along a length of a project.

The real-time automatic controlled variable roller compaction and documentation system (a.k.a. intelligent compaction) allows for optimization of compaction rates and real-time quality control. The system works by using accelerometers to monitor the speed of the dynamic wave through the soil, induced by the vibratory rollers, in order to measure the dynamic stiffness of the soil, which generally increases with higher compaction. Efficient fill densification is achieved via automatic adjustment of compaction energy and the measurement/documentation feedback, eliminating time wasted on compacting areas that are already adequately compacted. This energy variability and efficiency is achieved by the use of two counter rotating weights in the drum, rather than the conventional single, one-directional eccentric weight. The weights rotate in opposite directions and only come together in a common direction in the downward vertical inclination. This eliminates unwanted and wasteful movements in the lateral and upward directions that occur with conventional compaction drums. Internally, the entire counterweight assembly is rotated to adjust the direction of the point where the two weights act together. If the onboard monitoring system determines the soil is compacted to a satisfactory level, it will automatically reduce the vertical component of force at the specific time and location.

In addition, the ability to monitor density improvement during compaction both speeds up and improves the aerial extent of quality control. Most importantly, the ability of instrumented compaction equipment to provide 100% quality control coverage enables the use of performance based approaches to specifications, and the effective implementation of warranties and guarantees for both earthwork and pavements.

This method does require proprietary specialized monitoring equipment, but the equipment and process are not patented. The equipment is readily available in the U.S., and requires nominal operator training.

8.3.3 Risk Acceptance

Warranties for materials and workmanship are common in the construction industry, with most performance bonds covering such items for 1 year following completion of a project.

However, the new emphasis on warranties for highway construction involves the guarantee of the long-term performance of highways. Typically, a long-term warranty is considered to cover a period from 2 – 5 years. It is beyond the generally accepted standard warranty period of 1 year. This creates a very difficult situation, and one that involves a very high degree of uncertainty with our current state of practice and technology in the United States highway construction industry (Hancher, 1994 - NCHRP Synthesis 195).

This shift in philosophy has basically brought about a shift in project risk. Traditionally, the owner has assumed nearly all the performance risk by developing the design, specifying materials, and either specifying the results (density) or method to achieve the desired result, and measuring and accepting the contractor's work. The contractor is at risk for gross failures resulting from noncompliance with contract requirements detected in the first year of performance, while the owner assumes responsibility for failures and maintenance following the initial year of performance. By extending the initial warranty period from 1 year to a period up to 5 years, the owner has shifted some of this inherent risk to the contractor. As a result, the contractor has been tasked with becoming a more integral part of the design and construction process.

Extending the period of performance and assigning performance risk to various parties has lead to more sophisticated approaches with respect to life-cycle performance monitoring, early detection of potential problems, cost analyses, and budget optimization. For a contractor to be willing to accept more risk, he or she should have a more active role in design.

8.4 CONSTRUCTION AND CONSTRUCTION MONITORING

Construction of the pavement involves grading to provide a uniform support layer(s) at the appropriate elevation. In modern construction, either major earthwork or reconstruction, sophisticated equipment is available to excavate, haul, add water, aerate (decrease water), spread, and compact to achieve this purpose. The objective of this type of operation is to achieve a structure with specific design intent. Most earthwork projects have the additional objective that transforms existing or natural topography to an acceptable and safe vertical and horizontal alignment. A common goal is to achieve this new alignment with a balance of site materials. Projects that require more soil in fill areas than can be produced from cut zones will require additional materials from off-site borrow sources. Projects that generate more material from cut zones than can be placed in fill areas will be wasted.

The construction process is described in this section, along with the requirements for monitoring each phase of the construction activities, as outlined in Section 8.3 and summarized in Table 8-2.

For new construction, subgrade preparation will typically require grubbing and grading (either cut or fill) to meet subgrade elevation requirements. In either case, clearing and grubbing is very important to remove vegetation, debris, and any organic, soft, or otherwise unsuitable materials from the surface of the site, either at subgrade level or before placing fill. For reconstruction, the old roadway surface will be removed (possibly recycled), possibly along with the base and subbase layers, if they are not suitable for support (*i.e.,* intermixed with large amounts of fine-grained soil). Observation of heavy equipment operations on the site at this phase provides the first indication of the subgrade adequacy. Rutting and deflection during initial earthwork operations indicates an immediate problem. This may not be a problem if significant undercut or stabilization was anticipated prior to construction. However, if soft materials are encountered at subgrade or initial fill elevation and were not anticipated, immediate action should be taken. In either case, the conditions must be improved.

In order to improve the soil conditions at a site, a couple of options are common:

1. Remove marginal soils and replace with select material of higher quality.
2. Stabilize in-place soil (using one of the many techniques discussed in Chapter 7).
3. Improve site drainage.

Often the removal and replacement alternative is used because it appears to be the easiest. This is generally true for small areas or areas where spot locations are identified. However, undercutting may not be the most effective or even desirable for large areas. Excess water in soil is the principal cause of unstable conditions, and reducing the soil water content either by dewatering or stabilization methods may offer a better solution to expedite construction. Hauling or loading on a wet subgrade may continually disturb the section. Excavation in wet conditions often leads to more excavations. The variation of soil types and saturation levels should be evaluated by subsurface investigation to determine the vertical and longitudinal extent of the problem before a decision is made on which method to use. Excavation, drainage, and confirmation of stabilization will be reviewed in greater detail in the next section on construction techniques.

Proper consideration of existing conditions should be given prior to earthwork construction. For example, areas that are to receive fill soils should be cleared and grubbed (*i.e.,* vegetation, organic soils, and weak or otherwise unsuitable soils removed). Long-term performance issues are often traced to inadequate removal of unsuitable materials. Removal of surface soils containing organic matter is important, not only for settlement, but these soils are often moisture sensitive, losing significant strength when wet, and are easily disturbed under construction activities. Site QC/QA personnel should monitor and confirm that these organic soils have been adequately removed. Documentation should including a visual description along with photos of the cleared surface. If additional excavation will not be made, the surface should be checked at this point for compliance with specification requirements. This will often require proof rolling or other testing such as DCP. Unsuitable materials also often find their way back into filling operations. Therefore, the control of unsuitable materials should also be documented. Other special earthworks should also be observed and documented, including embankments properly sloped to prevent slides, keyways (a.k.a. shear keys) constructed to avoid toe slope failures, and erosion protection techniques to maintain long-term stability. Large embankments can cause settlement of the natural soils, thus analyses examining settlement potential of underlying soils is essential. If settlement is anticipated, techniques such as removal-and-replacement or surcharge embankments can be utilized to mitigate the problem (See FHWA NHI-00-045, 2000).

In unusual circumstances, a roadway alignment may be constructed over large voids or weakened soil zones, such as caves, faults or collapsed, abandoned mines, and sinkholes. Detection of these underlying potential problems prior to construction is ideal. Geophysical techniques such as ground penetrating radar (GPR) or spectral analyses of surface waves (SASW), again as described in Chapter 4, may be used in conjunction with historical information and experience to detect or predict the likelihood of encountering these described problems. Mitigation may include excavation and backfill operations, injection grouting, or grout columns using tubular fabric forms. For very localized areas and in karstic regions where voids are random but anticipated to be small, there has also been some success with bridging such areas with thickened reinforced concrete slabs or reinforced base and subbase using geogrids or welded wire mesh. However, for these techniques to be successful, the areal extent of the subsidence area must be clearly defined.

8.4.1 Drainage

If conditions exist during construction that indicate the need for underdrains (*e.g.,* wet, saturated conditions) or the cleaning of the existing underdrains outlets, then this work should start as soon as possible (after Ohio DOT, 2002). Some examples of these conditions are as follows:

1. existing underdrains with clogged outlets on rehabilitation projects
2. free water in the subgrade
3. saturated soils of moderately high permeability, such as sandy silt and silty clay of low plasticity

4. groundwater seepage through layers of permeable soil
5. water seeping in the test pits
6. water seeping from higher elevations in cut locations
7. water flowing on the top of the rock undercuts

Significant subgrade stability improvement can be obtained by cleaning out the existing underdrain outlets on rehabilitation projects and by adding construction underdrains on new construction projects. The FHWA/NHI course manual on Pavement Subsurface Drainage Design is a useful reference. Once the underdrain systems are in place and functioning, the drainage system can typically reduce subgrade pumping problems within a few days, but may take longer depending on the characteristics of the in-situ materials. Soils that are subject to densification and are not free draining (percent saturation exceeding 80 – 90%) within 1 m (2 – 4 ft) of the surface are not expected to support construction traffic. This order of magnitude of saturation is frequently observed to be the limit for compaction stability when developing moisture-density curves in the laboratory. Saturated soils with more than 10% fines (minus 0.075 mm (No. 200) sieve) are not expected to be drainable with respect to supporting construction traffic. Moisture reduction by evaporation (*e.g.,* disking and aeration) may be more feasible than gravity drainage for these types of soils.

For rehabilitation projects, the Contractor should be instructed to unclog the underdrain outlets immediately, attempting to perform this work in the timeframe listed above. If the project consists of several phases, then the Contractor should perform the outlet cleaning for the entire project at the same time. Because of the timeframes involved, construction underdrains should not be used for rehabilitation projects.

For new construction projects, subgrade stability can be achieved by constructing the plan or construction underdrains as soon as the water problem is found (see Figure 8-4). New construction projects can allow a longer period of time for the underdrain system to work. At the beginning of construction, and certainly before winter shut down, are opportune times for this work.

The plan underdrains should be placed only when they will not be contaminated by further construction. If contamination is a concern, then sacrificial or temporary construction underdrains should be used on the project.

Construction underdrains are usually placed in the centerline of the roadway. They may also be placed in the ditch line, if the water is coming in from a cut section at a higher elevation. The porous backfill is extended to the subgrade elevation. The outlets for the construction underdrain are the same pipe material and backfill as regular underdrains. The underdrains can be outlet to any convenient location. Some potential outlet locations are catch basins, manholes, pipes, or ditches. The project should not be concerned with the contamination in the upper portion of construction underdrain backfill. Construction underdrains are sacrificial underdrains that will continue to work throughout the life of the contract and afterwards even though the upper portion is contaminated.

For rock or shale cuts, the design underdrains should extend at least 150 mm (6 in.) into the existing rock formation. If the underdrains are too high, the water will accumulate at the rock and soil interface and cause subgrade instability.

Figure 8-4. Underdrain installation (*photo courtesy of Ohio DOT*).

8.4.2 Excavation

Most construction projects will consist of some amount of excavation, or removal of in-situ soil to some design elevation or grade line. Excavation is typically accomplished using scrapers (also referred to as pans) or shovels, which are among the list of heaviest equipment used in modern earthwork. Observations made at this stage of construction are considered the first line of QC documentation. Site personnel should observe vertical movements below the construction equipment during excavation. Moderate to large deflections are the first indication that weak soils exist and some corrective action may be necessary.

Scrapers have the ability to remove material from grade and spread at another location nearby. Some scrapers may need to be pushed by another scraper or by a bulldozer in order to advance while cutting into the zone of soil to be removed. Other scrapers are equipped with an elevator system (Figure 8-5) that allows the excavated material to be readily loaded without the assistance of a push from behind.

Scrapers are commonly used in cut-fill earthwork operations, where the majority of the soil excavated is placed along another portion of the project. Anticipated site conditions that consist of wet or saturated soils due to water table elevations may necessitate the use of other forms of excavation equipment in order to minimize disturbance to the underlying in-situ soils. A common piece of equipment is the track-mounted excavator (shovel) like the one illustrated in Figure 8-6. Materials removed or excavated by this means require transfer to a secondary piece of equipment for hauling off site, or to another location along the alignment where fill soils are required.

8.4.3 Hauling and Placement

While scrapers (a.k.a. pans) transport their payload from a cut zone to a fill area, shovels require a haul truck to be utilized. There are several types of hauling vehicles, including end dump, side dump, bottom dump (or belly dump), and articulated dump trucks, as illustrated in Figure 8-7.

Figure 8-5. Self-loading scraper (*photo courtesy of Caterpillar*).

Figure 8-6. Track-mounted excavator (*photo courtesy of Komatsu*).

Figure 8-7. Articulated dump truck (*photo courtesy of Komatsu*).

Some projects will require off-site materials to be hauled in because of an imbalance of site materials. These borrow materials will typically be hauled in trucks and dumped near their intended final location. Depending on the dumping method, these piles may require spreading using a bulldozer or motor-grader (shown in Figures 8-8 and 8-9, respectively). Again, observation of this activity can indicate soft or unsuitable areas that will require special treatment. When excessive rutting is noted, haul routes should be changed so as to minimize the depth of disturbance. An assessment by the engineer should be made as soon as practical to determine if underdrains are needed. Often, well-placed and well-timed construction underdrains can mitigate the problem, and hauling over the previously unstable location may improve the stability by adding compaction to the draining soils.

Some projects may restrict hauling on existing paved roads in order to eliminate damage to existing local roadways. In this case, it is possible to utilize a conveyor system to transport the borrow material to the site, like the one shown in Figure 8-10.

During the hauling operation, material to be used as fill should be sampled and tested by QC/QA personnel for compliance with the specification requirements (*e.g.,* soil type, gradation, etc.) Laboratory moisture-density tests (a.k.a. Proctors) should also be preformed for correlation with field density testing.

Figure 8-8. Bulldozer (*photo courtesy of Komatsu*).

Figure 8-9. Motor grader (*photo courtesy of Caterpillar)*

Figure 8-10. Earth-moving conveyor system (*Atlanta Airport – 5th Runway Embankment Placement*).

8.4.4 Field Compaction

Compaction can be defined as the densification of soils by the application of mechanical energy, oftentimes requiring a modification of water content. The purpose of compaction is generally to enhance the strength or load carrying capacity of the material, while minimizing long-term settlement potential. By adjusting the moisture content to a value at or near a moisture content considered optimum – as described below – reduced volume changes and increased strength can be achieved.

Significant advances have been made in the science and technology of earth structures in the last century. In the early 1900s, soils were placed in embankments by end dumping from wagons, with little attempt to compact. Structures that were placed by hand using baskets had, at a minimum, foot traffic to unintentionally "compact" the soil. It was observed that this foot traffic actually strengthened the soil, thus creating the concept of mechanical stabilization. Different types of field compaction equipment are appropriate for different types of soils. Steel-wheel rollers, the earliest type of compaction equipment, are suitable for cohesionless soils. Vibratory steel rollers have largely replaced static steel-wheel rollers because of their higher efficiency. Sheepsfoot rollers, which impart more of a kneading compaction effort than smooth steel wheels, are most appropriate for plastic cohesive soils. Vibratory versions of sheepsfoot rollers are also available. Pneumatic rubber-tired work well for both cohesionless and cohesive soils. A variety of small equipment for hand compaction in confined areas is also available.

Recommended field compaction equipment for various soil types is summarized in Table 8-4. The effective depth of compaction of all field equipment is usually limited, so compaction of thick fills must be done in a series of lifts, with each lift thickness typically in the range of 150 – 300 mm (6 – 12 in.) with greater depths possible (up to 0.7 m (2 ft)) through the use of specialized high energy equipment and the right type of soil conditions (*e.g.,* free-draining granular soils). The soil type, degree of compaction required, field compaction energy (type and size of compaction equipment and number of passes), and the contractor's skill in handling the material are key factors determining the maximum lift thickness that can be compacted effectively. Control of water content in each lift, either through drying or addition of water plus mixing, may be required to achieve required compacted densities and/or to meet specifications for compaction water content.

Proof rolling with heavy rubber-tired rollers is often used to achieve additional compaction beyond that from normal compaction and, more important, to identify any remaining soft areas. The proof roller must be sized to avoid causing bearing capacity failures in the materials that are being proof rolling. Proof rolling is not a replacement for good compaction procedures and inspection. QC/QA personnel need to be present on site to watch the deflections under the roller in order to identify soft areas. Construction equipment, such as loaded scrapers and material delivery trucks, can also be used to help detect soft spots along the highway alignment. Details on the determination of suitability using proof rolling methods were provided in Section 8.3.

Table 8-4. Recommended field compaction equipment for different soils
(*after Rollings and Rollings, 1996*).

Soil	First choice	Second choice	Comment
Rock fill	Vibratory	Pneumatic	—
Plastic soils, CH, MH (A-7, A-5)	Sheepsfoot or pad foot	Pneumatic	Thin lifts usually needed
Low-plasticity soils, CL, ML (A-6, A-4)	Sheepsfoot or pad foot	Pneumatic, vibratory	Moisture control often critical for silty soils
Plastic sands and gravels, GC, SC (A-2-6, A-2-7)	Vibratory, pneumatic	Pad foot	—
Silty sands and gravels, SM, GM (A-3, A-2-4, A-2-5)	Vibratory	Pneumatic, pad foot	Moisture control often critical
Clean sands, SW, SP (A-1-b)	Vibratory	Impact, pneumatic	—
Clean gravels, GW, GP (A-1-a)	Vibratory	Pneumatic, impact, grid	Grid useful for over-sized particles

It is very difficult to achieve satisfactory compaction if the lift is not on a firm foundation. Figure 8-11 shows a typical stress distribution under a rubber-tired pneumatic roller for two different foundations. The first case corresponds to a homogeneous deposit with a constant modulus of elasticity equal to 170 MPa (25,000 psi), which is representative of a good quality granular material. The second case corresponds to a 150 mm (6-in.) thick lift of 170 MPa (25,000 psi) granular material over a subgrade soil having a modulus of 35 MPa (5,000 psi), which is representative of a soft clay having a CBR of around 3 or 4. As is clear from the figure, the stresses induced by the roller in the second case are much lower than in the first. High levels of compaction will be difficult to achieve in the thin lift over the weak subgrade, and the high stresses in the lower soil may produce shear failure and excessive rutting, especially if proof rolling is performed. Thus, it is easy to see the importance of monitoring cut surfaces through proof rolling and measuring compaction of each lift during fill placement.

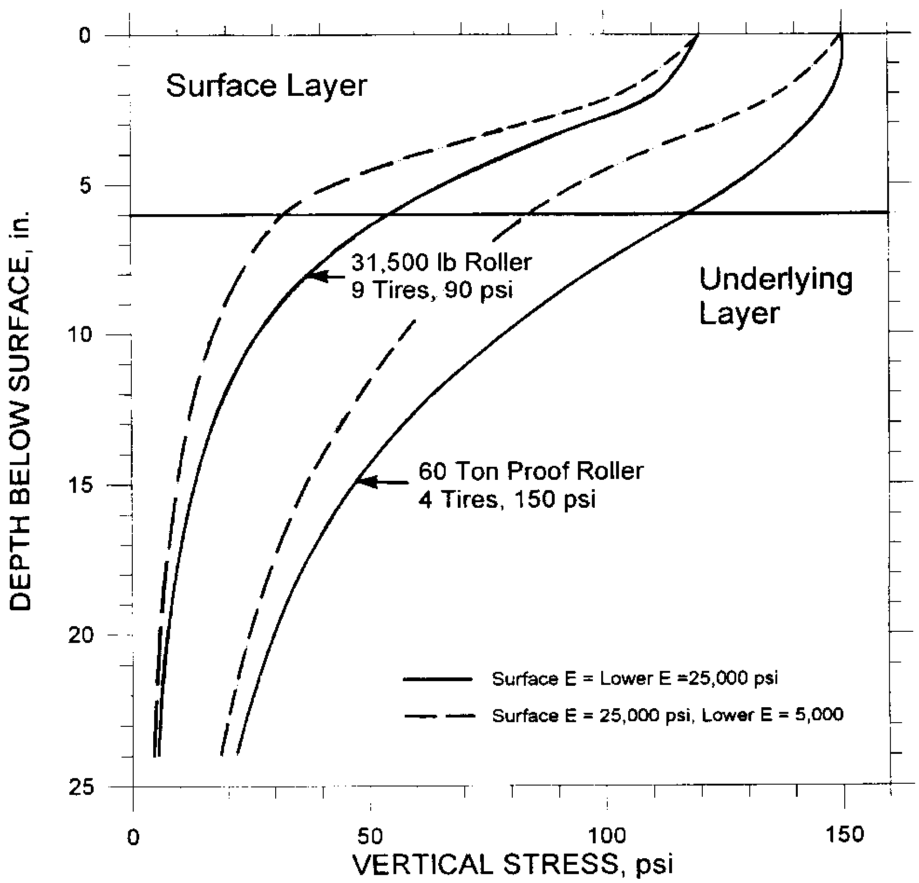

Figure 8-11. Stress distributions under rollers over different foundations (*after Rollings and Rollings, 1996*). (1 ton = 8.9 kPa, 1 psi = 6.9 kPa)

The most common measure of compaction is density. Field moisture and densities can be measured using a variety of standard methods. Field density is correlated to moisture-density relationships measured in the lab (AASHTO Test Procedures T99 and T180). Moisture-density relationships for various soils are discussed in Chapter 7, and the lab tests are covered in Chapter 5. Optimal engineering properties for a given soil type occur near its compaction optimum moisture content (w_{opt} or OMC), as determined by the laboratory test standard. At this state, a soil's void ratio and potential to shrink (if dried) or swell (if inundated with water) is minimized.

In controlling compaction, the appropriate moisture-density laboratory method (*e.g.,* standard or modified Proctor) should be matched to the equipment typically used in the local region. Higher energy equipment should be controlled with compaction tests based on high energy (*i.e.,* modified in lieu of standard Proctor). There is a trend to lower moisture content tolerances with consideration for the higher energy equipment; however, this method could result in lower compactive efforts. The reason for this move is that the high-energy compaction equipment is causing an apparent pumping to occur when the soil is above its optimum moisture. However, this method could ultimately lead to premature failures as the subgrade saturates over time (*e.g.,* loss in stiffness/strength, or potential volume change). It is considered good practice to compact at the optimum moisture content for the material used. If some deviation occurs, it is better to be on the wet side, rather than dry.

Compaction, or mechanical stabilization (*i.e.,* water content adjustments and densification) is the most common and least expensive of all soil improvement techniques. Perhaps the most common problem arising from deficient construction is related to mechanical stabilization. The intent of mechanical stabilization is to maximize the soil strength (and minimize the potential volume change) by the proper adjustment of moisture and the densification at or near the ideal moisture content, as discussed in Section 8.4.4. Without proper quality control and quality assurance (QC/QA) measures, some deficient work may go unnoticed. This is most common in utility trenches and even bridge abutments, where it is difficult to compact because of vertical constraints. This type of problem can be avoided or at least minimized with a thorough plan and execution of the plan as it relates to QC/QA during construction, as reviewed in detail in Section 8.3. This plan should pay particular attention to proper moisture content, proper lift thickness for compaction, and sufficient configuration of the compaction equipment utilized (weight and width are the most critical). Failure to adequately construct and backfill trench lines will most likely result in localized settlement and cracking at the pavement surface.

There are several compaction devices available in modern earthwork, and selection of the proper equipment is dependent on the material intended to be densified. Generally, compaction can be accomplished using pressure, vibration, and/or kneading action. Heavy equipment, as measured by ground pressure, is utilized to accomplish the compaction process. Some heavy equipment have low ground pressures, such as tracked vehicles like bulldozers, and rubber-tire equipment like front end loaders and motor graders. The low ground pressures imparted by these types of vehicles are not effective at compacting soils; however, tracked vehicles do provide some limited compaction of granular, cohesionless materials by means of vibration.

A *smooth drum roller* is perhaps the most common of all compaction devices, capable of applying pressure across the width of its drum. Smooth drum rollers can consist of a single drum (Figure 8-12) or dual drum. Most drum rollers are equipped with oscillary vibrators to increase the energy transmitted to the surface of the layer being compacted. These smooth drum rollers are best suited for granular, relatively non-cohesive soils. Some agencies have used smooth drum rollers to finish subgrades prior to base construction, and have even employed them as a proof rolling instrument.

Figure 8-12. Smooth drum roller (*photo courtesy of Bomag*).

The *sheepsfoot or studded rollers* like the one shown in Figure 8-13 are typically used on cohesive soils. These rollers are very similar to the smooth drum roller, however, many rounded or rectangular protrusions (or feet) are attached to the drum. These protrusions provide for a very high contact pressure in a small zone of soil. By spacing these protrusions apart, very high vertical stresses, as well as horizontal stresses, are achieved, thus creating a kneading action that compacts from the bottom up. During compaction, the roller literally “walks out” of the lift once compaction is achieved. This kneading or shearing action has the ability to produce a soil structure that maximizes a cohesive soil’s strength at high density levels. Some sheepsfoot or studded rollers are also equiped with oscillatory vibrators to increase the effectiveness across a broader range of soil.

Pneumatic or *rubber-tire rollers* have also been utilized to compact materials. These compactors are typically used as an alternate for compacting a variety of soil types (see Table 8-4). They are particularly effective for non-cohesive silty soils. Some agencies have used them successfully in embankment placements and have also employed them as a proof rolling instrument. Hauling vehicles (scrapers and loaded dump trucks) have been used for compaction purposes.

The latest compaction equipment are *high-energy impact rollers*, which use shaped (*e.g.,* triangular ellipsoids or hexagonal), as opposed to round drums, as shown in Figure 8-14. The high energy imparted by these systems allows them to achieve compaction at a faster rate and to greater depths. A comparison of different types of compaction equipment based on vertical settlement with number of passes is shown in Figure 8-15, demonstrating the superior effectiveness both in terms of number of passes and influence depth of high-energy equipment.

Most of the research on this equipment has been performed in Europe, and unfortunately the availability is limited at this time in the U.S. The Europeans are also experimenting with hydraulic and pneumatic impact hammers to achieve compaction at greater depths, especially in rubblized fills (Dumas, et al, 2003). This technique uses a 5-tonnes (5.5 ton), 1-m (3.3 ft) drop hydraulic pile hammer to drive a large foot into the ground. This technology eliminates excavation and allows for compaction of shallow layers (or soils with low moisture content) up to 3 m (9 ft) thick. The technique was initially developed by the British and U.S. military for rapid airfield repair.

Figure 8-13. Sheepsfoot roller (*photo courtesy of Bomag*).

Figure 8-14. Impact roller.

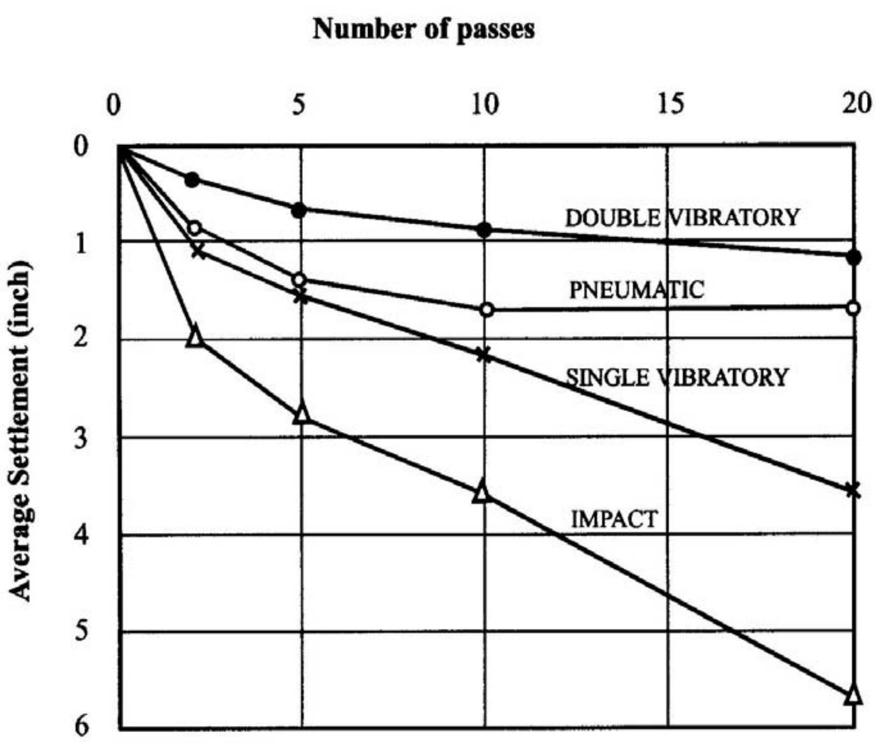

Figure 8-15. Compaction efficiency. (1 in. = 25 mm)

Another significant development in compaction equipment is the use of instruments in the compaction drums to measure the response of soil (*e.g.,* stiffness). The equipment is computerized, allowing for real-time monitoring of foundation response and automated feedback controlling vibration amplitude and frequency, and vehicle speed. While *intelligent compaction equipment* was originally developed for contractors to improve their efficiency in achieving compaction with a minimum number of passes, it has direct and significant application potential for controlling and monitoring compaction effectiveness for pavement performance, as discussed in Section 8.3 on QC/QA.

Other means exist in which to promote deep densification, including dynamic compaction and vibroflotation. These processes are discussed in the FHWA/NHI *Ground Improvement Techniques* reference manual (FHWA NHI-04-001) and, given the right conditions, can be used to densify soils at depths of over 9 meters (30 ft). Each is limited to successes achieved in deep, loose non-cohesive soils, such as sands and gravels.

As previously discussed, with the advent of newer higher energy compaction equipment, agencies should carefully evaluate their current specifications to meet these changing demands.

The final phase of subgrade construction is the confirmation of surficial support prior to placement of the base/subbase layers. One or more of the methods outlined in Section 8.3 should be utilized (*e.g.,* proof rolling, DCP, FWD).

8.4.5 Stabilization

In certain instances, when stabilization is a more economical means of constructing a pavement section with the desired support characteristics, use of chemical admixtures, such as lime, flyash or cement, is common. These mixtures are typically designed in a controlled laboratory environment in order to establish volumetric properties, such as admixture design content, maximum density, moisture content, and strength, as discussed in Chapter 7.

Chemical Stabilization/Modification. In the special case where lime (or other pozzolanic modifier such as cement or flyash) is to be utilized to enhance the load carrying capability of the soil, the additional effort of introducing the modifier to the soil and mixing prior to compaction is required.

The basic construction steps for chemical stabilization of subgrade soils are (1) pozzolan delivery and distribution; (2) mixing; (3) compacting; and (4) curing. Pozzolans can be applied to a soil either dry or as a slurry. In the case of dry lime, the lime may be either in the form of dry hydrated lime, which is very fine-grained and, thus, may pose dust control problems, or dry quicklime, which is granular and much less dusty.

The pozzolanic material specified for stabilization or modification is distributed along the road alignment, either via bags that are spread manually, by pneumatic trucks with spreader bars, or by dump trucks with controlled tailgate openings. Lime slurries can be mixed in a central mixing plant or in various types of portable mixing systems. A typical lime slurry mixture would consist of 0.9 tonne (1 ton) of lime mixed with 1900 liters (500 gal) of water to produce 550 tonnes (600 tons) of slurry with 31% lime solids (Transportation Research Board, 1987).

Adequate mixing of the pozzolanic material with the soil is critical; poor mixing is the leading cause of unsatisfactory stabilization results. Subgrade soils can be mixed on site with the pozzolan by disking, repeated blading, or by traveling rotary or pug-mill mixing equipment.

Figure 8-16. Roadway stabilizer/mixer (*photo courtesy of Bomag*).

Mixing is usually done in thin lifts and often with multiple passes, with the lift thickness and number of passes dependent upon the soil type and the mixing equipment being used. A two-stage mixing process is sometimes used for highly plastic materials; the reduced plasticity and coarser texture that develops during curing for several days after the initial mixing makes the soil more workable for final mixing and compaction.

Compaction of chemically stabilized soil mixtures follows standard procedures. However, with respect to lime stabilization, the addition of lime will generally decrease the maximum density and increase the optimum water content at a given compaction energy, which may cause problems determining the percentage of specified density achieved by the field compaction. Compaction curves of the in-situ lime-soil mixture at the time of compaction may be required to determine the appropriate density values for field compaction control.

Curing at temperatures above 4.4^o C (40^o F) and with adequate moisture is essential for the pozzolanic reactions underlying the long-term strength gains in lime-stabilized soils. A cure period of 3 – 7 days is typically employed, with adequate moisture maintained either through moist curing (*e.g.,* truck sprinklers) or by applying an asphalt seal over the surface.

Similarly, modified soil (lime, cement and/or flyash) will require special QC/QA considerations, as discussed in Section 8.3. Again, a final evaluation of the stabilized subgrade surface should be made by one or more of the methods described in Section 8.3 (*i.e.,* proof rolling, DCP, FWD).

8.4.6 Base and Subbase Construction

In the case of aggregate base construction, material is hauled to the site and is typically placed directly on grade, spread to a uniform specified thickness, and compacted. Care must be exercised to minimize segregation of aggregate blends. Good practices to prevent or minimize segregation include eliminate the number of transfer points prior to final grade placement (avoidance of intermediate stockpiling), and minimize the amount of spreading and movement once on grade. Asphalt pavement spreaders have been successfully utilized to distribute aggregate base materials on grade. Use of such equipment allows good control of specified thickness and reduces the potential of segregation caused by traditional spreading techniques such as motor-grader or dozer operations. This practice can also positively affect the overall project profile smoothness objectives. Regardless of the method of placement, care should be exercised to avoid the potential to contaminate the aggregate with site soil. Contamination is typically introduced when wet soils adhere to construction traffic tires or tracks, and are "cleaned off" when traversing over newly placed aggregate layers. Again, QC/QA personnel should document observations and test results, indicating conformance or non-conformance with the specification.

Base and subbase layers are typically aggregate materials containing moderate (dense graded) to little (open graded or drainable) fines. Compaction is typically achieved utilizing vibratory smooth drum compactors described previously in Section 8.4.4. Failure to achieve proper compaction may be a result of several factors, either individually or in combination:

- Lack of substrate support (Should have been detected and corrected prior to placement of layer.)
- Improper size of compactor
- Excessive moisture (Perhaps from rainfall. Site surface runoff should be promoted. Excess moisture usually can be dried by blading and allowing excessive moisture time to evaporate.)
- Segregation

Correction in the form of drying and recompacting should work in the majority of cases. If problems persist, removal and replacement may be warranted. If the problem is deeper than the base or subbase layer, subexcavation and replacement or some form of chemical stabilization technique may be required, as discussed in Section 8.4.5.

Chemical Stabilization. Mixing of lime or cement with coarse aggregates for base and subbase layers is often done in a central mixing plant. Although the central mixing plant is required primarily for gradation control, it also enables good control of the lime-aggregate or cement-aggregate proportions and mixing. Again, testing during construction should closely parallel that described for soil stabilization/modification.

8.4.7 Pavement Drainage Systems

For construction of pavement drainage systems, design should acquaint agency construction personnel with the impact of construction on the design results. Care during construction to build the pavement drainage designed section without compromising the effectiveness of design is essential to the pavement's long-term performance. Key performance elements for construction personnel to remember include

- Good pavement starts with a good foundation. A stable platform is required for construction of the permeable base.
- Quality of aggregate and its ability to meet gradation requirements is essential to meeting design performance.
- An awareness that the introduction of fines into the permeable base during construction could result in the premature failure of the pavement.

- Unstabilized drainable base tends to displace under traffic.

In addition to these key elements, construction personnel (contractor and inspector) should be aware of how each construction activity can impact the performance of the pavement drainage system.

Subgrade Preparation As with all road sections, the foundation surfaces are required to be level, somewhat smooth and constructed to required grades. With drainable pavement sections, constructing and maintaining the required subsurface grades until pavement construction is essential in maintaining positive pavement drainage. Grades that are too flat, local depressions resulting from soft areas, and/or depressions from equipment trafficking can lead to ponding of water below the pavement structure and subsequent loss of foundation support.

Separation Layers For granular separation layers, the gradation of materials must be carefully checked against design requirements. Material that is more open than specification requirements may allow migration of fines and contamination of the permeable base. Good compaction of the separation layer is essential to the placement of the permeable base. The subbase should be observed for rutting during compaction and subsequent trafficking. Subbase surface rutting may be an indication of subgrade rutting and requires immediate attention (*e.g.,* by reducing equipment loads or increasing the lift thickness). "A separator is not a substitute for proper subgrade preparation" (FHWA-SA-92-008).

For construction of geotextile separation layers, material and certification should be checked against design specification requirements to make sure the proper materials have been received and used. The smooth subgrade surface is desirable. It is recommended that sharp rock protrusions or loose rocks (usually greater than 20 mm (¾ in.) in size) be removed to avoid damage to the geotextile, unless such conditions have been anticipated and heavy-weight (greater than 250 g/m^2 (7.4 oz/yd^2)) geotextiles have been specified.

Edgedrains Proper grading is essential for edgedrains to be effective. Undulating drain lines are not acceptable, as water will accumulate in depressed areas. Good practice dictates that drains must be properly connected to the permeable base and to outlets. Outlets are required to be set at the proper grades and ditch lines graded according to drainage requirements. Drain lines are to be carefully marked and care maintained throughout construction to avoid crushing the pipe with construction equipment (*e.g.,* concrete trucks and other heavy vehicles/equipment are not to be allowed to travel over drain lines). Drains are sometimes constructed after pavement construction to avoid this problem. In this case, temporary drainage is required for the permeable base to prevent a bathtub effect from water trapped in the porous base.

As discussed in Section 7-2, the filter (geotextile or aggregate) has to be carefully placed at the design location around all sides of the backfill, not in contact with the permeable base.

The edgedrains are required to be backfilled with material at least as permeable as the permeable base. Most states use a graded aggregate, while some states use free-draining sand. In either case, the drainage backfill should be placed below the invert of the pipe, and compacted to better support the pipe, reduce the risk of crushing the pipe, and to prevent subsequent subsidence that could affect the road. As with the trench line, the pipe must be placed at the proper grade on a level surface. Drainage backfill is placed to the final elevation and protected from fouling until the pavement section is complete. Maintaining an open drainage aggregate is critical during the remaining construction period. A shovel full of fines could clog the drain. Construction traffic should not be allowed to traverse over the drain line. The drain line could be covered with a geotextile to help prevent fouling during construction. Also, outlets must properly drain during this phase to provide temporary drainage during construction. Ditch lines should be continuously checked and maintained, as erosion sediments could back up and foul essential features. Headwalls for outlets should be installed and outlets marked so they will not be disturbed by subsequent construction.

The edgedrain system should be inspected and tested for proper operation toward the end of construction, before final acceptance. An acceptance criteria based on performance parameters must be established, otherwise signs of poor construction practices will most likely not be identified until major structural damage is done and the pavement life has been shortened. Inspection techniques can consist of simply pouring water on the drainage layer in an upstream section of the drain and measuring the outflow against the anticipated rate. The most effective method for post-construction evaluation is video equipment (*e.g.,* Iowa borescope and other mini-cameras). Several states do not accept edgedrains until video inspection indicates that they have not been damaged during construction. The design of the edgedrain system should have included pipe access installed at the "upstream" end of the drain line to gain access for camera inspection, effectiveness testing, and subsequent maintenance flushing activities.

Drainable Base Materials Unstabilized permeable base requires close control of the material gradation and attention to activities that might cause segregation. An asphalt spreader box is usually required to reduce segregation. Unstabilized base tends to weave and rut under traffic.

Asphalt-stabilized permeable base usually contains AASHTO No. 67 or No. 57 crushed aggregate plus 2 – 2.5% asphalt by weight. Higher asphalt cement percentages may be required when a less-open gradation is used. Some states prohibit the use of bank run gravel aggregate because of the rounded faces. Stabilized aggregate should be placed at 90 – 120^o C (200 – 250^o F) but not rolled until it is below 65^o C (150^o F). Vibratory rollers are usually not allowed, and the number of roller passes is usually between 1 and 3 (FHWA-SA-92-008).

Cement-stabilized permeable base usually contains 2 to 3 bags for No. 67 and No. 57 crushed aggregate. As with asphalt-stabilized base, higher amounts may be required for less-open graded aggregate. Cement-stabilized base could be cured similar to pavement. Test strips are recommended to determine appropriate curing and compaction methods (FHWA-SA-92-008).

Care is required to protect the permeable base from fines contamination (*e.g.,* from dirty construction equipment, adjacent backfilling operations, erosion sediments, etc.). While the drainable base can generally support light construction loads, it should not be used as a haul road. Equipment that would cause rutting (*e.g.,* concrete and loaded dump trucks), dirty equipment, or equipment transporting fines should not be allowed to traverse over the permeable base. Good practice dictates that traffic be restricted to low speeds with minimal turning allowed. Traffic should not be allowed until complete drainage of the base and subbase has been confirmed.

Based on a survey of state agencies (Christopher and McGuffy, 1997), good construction of subsurface drainage systems appears to depend on a number of factors:

- The contractor (and inspector) should be knowledgeable in drain installation principles and practices.
- Someone with knowledge of drainable pavements must be on site at startup.
- Water needs a continuous, unobstructed path to drain, both during and after construction.
- A positive slope is required.
- Any discontinuity in flow path can destroy the system's effectiveness.
- The pavement (or shoulder) is supported by the system; therefore, compaction is essential.
- Construction activities for other work in the area can destroy good drainage installations.

8.5 PERFORMANCE MONITORING

8.5.1 Pavement Management Systems

Pavement management systems have been utilized as tools to document and track pavement performance. These systems typically rely on the assessment of the pavement wearing surface, in the form of distress surveys performed at periodic intervals, in order to not only illustrate how the pavement is performing, but to predict how the pavement may perform into the future. Through the use of these tools, agencies have been able to detect performance problems early, and correct the problems with routine maintenance during the pavement life-cycle. These tools assist agencies to best manage maintenance and capital budgets across their broad network of pavements, and can be utilized efficiently at the project level to optimize pavement performance for individual construction projects. These tools become very important at the project level when considering performance risk, particularly with extended performance periods.

A major disadvantage with the conventional distress survey input for a pavement management system, particularly with respect to pavement layers associated with unbound materials, is that problems are not detected until failure occurs. Problems caused by moisture intrusion into the subgrade and unbound base/subbase layers weaken the pavement system. If gone undetected, a pavement's life can be dramatically shortened. In order to circumvent this problem, agencies and particularly design-build teams, have seen the benefit of augmenting a solid pavement management system (distress survey) with structural surveys (NDT using one of the many geophysical testing techniques previously documented in Chapter 4, and described in further detail in the following sections).

8.5.2 Geophysical

Geophysical measurements detect differences or anomalies in material properties. However, these properties require interpretation as conditions relevant to pavement performance. As discussed in Chapter 4, geophysical testing can be used to locate voids beneath pavement sections for both construction and long-term performance monitoring. Periodic monitoring of a region with known problems such as solution caves or other karstic features can be a significant asset in evaluating the effectiveness of grouting programs to solve problems during construction and evaluate any long-term developments that could lead to future problems. The following two case histories provide a demonstration of effective use of geophysics in both short- and long-term monitoring programs.

As indicated in Chapter 4, the Finnish government performs resistivity testing on subgrades along with other in-situ and geophysical tests to develop a complete map of the subgrade system, including moisture and corresponding settlement and frost heave profiles. These anticipated profiles are then used to define the performance requirements for roadway warranties. The allowable settlement for a 30-year service period and a 5-year warranty period is calculated based on this well-documented and detailed site investigation (Tolla, 2002).

Widening and realignment of State Route 69 traverses an area of Tertiary-age travertine bedrock near Mayer, Arizona. During the design phase of the project, subsurface exploration encountered small voids within the right-of-way. A moderate-sized cave structure in the area was mapped by local speleologists. Arizona Department of Transportation (ADOT) was concerned that cave structures of unknown size might be found within a few feet of the new roadway subgrade. As a result, highway construction specifications contained special provisions requiring geophysical surveys to identify cave structures that could adversely affect the roadway and expose the traveling public to possible subgrade failure hazards. ADOT's concern was realized during construction when a D-9 Caterpillar tractor broke through a cave roof and dropped about 1.8 m (6 ft) into the void. A geophysical survey conducted of the cave-affected alignment identified 130 cave-type anomalies, and recommendations were provided to ADOT and the contractor to remediate the cave-affected highway section. Survey monuments were established for monitoring roadway performance and potential subgrade settlement (Euge et al., 1998).

8.5.3 Falling Weight Deflectometer

Much research has been conducted by FHWA in the past decade, particularly as part of the Long-Term Pavement Performance (LTPP) study. Although typically utilized as a tool to measure structural capacity of a pavement system for the primary purpose of designing strengthening and overlay thickness requirements, the FWD can be utilized to monitor the subgrade performance, as well as base/subbase performance. This type of program can be established by first measuring the deflection profile of a newly completed or rehabilitated pavement section at numerous discrete points (baseline data). These measurements (particularly deflections away from the loaded plate) can provide useful information about the deeper layers in the pavement system. Measurements made at annual or seasonal periods and compared with baseline data may indicate when potential problems exist. A loss in stiffness in a deep subgrade, or intermediate base or subbase layer, may indicate a poor drainage condition exists, one which can be readily corrected prior to premature pavement system failures by either constructing underdrains, maintaining existing underdrains, or altering the site hydrology in a way that better promotes site drainage.

8.5.4 Drainage Inspection (*e.g.,* video logging)

Performance monitoring of drainage systems is essential for both acceptance of the constructed facility and for maintaining a preventive maintenance program (NCHRP Synthesis 285). Probably the most significant development in edgedrain inspection has been the use of small diameter, optical tube video cameras with closed circuit video systems. Video cameras allow the inside of the edgedrain system to be logged, and expose the weaknesses in construction and inspection procedures. Iowa was one of the first states to effectively use video inspection *(Steffes et al., 1991)*. Random inspection of drains with video cameras has exposed many problems including

- rodent nests in the drain,
- varied sag from main line to outlet,
- polyethylene tubing and connector failures,
- break from stretch or puncture, and
- geocomposite drain J-buckling.

As was discussed in Chapter 4, significant effort to evaluate the use of video cameras as an inspection tool and demonstrate the technology was undertaken by the Federal Highway Administration. In evaluation of 269 outlet pipes that were inspected, 35% of the laterals could not be inspected because they were crushed or clogged, and the condition of the mainline could not be investigated. Of the mainlines that were evaluated, 17% were blocked or clogged. These findings clearly indicated that there were serious inadequacies in the edgedrain design, construction, and maintenance practice. The study also showed that the video inspection of edgedrains was a viable tool for determining the existing condition of edgedrains and had a definite role in providing construction quality assurance.

The Federal Highway Administration program to promote this technology appears to have had a significant impact. Over 17 states reported to have used a video camera. Many agencies own their own video camera, with a cost for the system ranging from \$13,000 – \$40,000. Some agencies retain consultants to perform video inspections. Video cameras have proven to be a valuable tool for many of the agencies in identifying problems and exposing weaknesses in construction and inspection procedures. Many states currently do or will shortly require video inspection for construction acceptance. Several agencies have reported that they have improved from an edgedrain failure rate of up to 40% to a failure rate of less than 5% by improving their QC/QA program, including the use of video cameras. Several agencies have incorporated their video camera into their preventive maintenance program, with periodic monitoring during routine inspection.

8.5.5 Instrumented Geosynthetics

Geosynthetics provide a convenient delivery system for performance monitoring instruments. Instrumentation, including strain gauges for deformation and stress measurements, pore pressure transducers to monitor soil suction, dielectric sensors to monitor moisture change, and thermistors to measure temperature change, can all be installed in the factory, delivered to the site, and hooked directly into a remote data acquisition system with telecommunications (no wires). Geosynthetics are currently available in Europe with an array of strain gages embedded in the product for monitoring subsidence (*e.g.,* from karst conditions and abandoned mines). This allows performance monitoring with practically no disruption to construction. Care is still required to avoid damage to the instruments during placement of the section and the initial fill over that section.

8.6 POST-CONSTRUCTION ISSUES AND SPECIAL CASES

The installation of structural features (*e.g.,* storm water lines and manholes, culverts, roadway drainage lines, etc.) adjacent to or beneath pavements can also lead to problems during or following construction. Proper installation of such structures is critical and close inspection during construction is critical. For example, a precast concrete pipe is installed as a storm drain. Each segment of the pipe is grouted, and the pipe is grouted into a junction box that also serves as a storm drain surface inlet (surface grate). The pipe is located on a 100-mm (4-in.) sand bedding at the bottom of a trench excavation. Following installation of the pipe, the trench is backfilled adequately, and the pavement is constructed. Imagine though, that one of the pipe joints was not adequately grouted, or post-construction settlement occurred (*e.g.,* due to inadequate embankment or bedding compaction) causing differential movement such that one of the joints separates. Because of the amount of storm water carried by the pipe during the most intense rainfall, the turbulent flow of water has begun to pipe soil backfill from around the pipe, and has swept it to the junction box and further down stream. The progression of this piping and erosion will eventually lead to pavement subsidence.

This type of pavement failure, subsidence of underlying strata, can be manifested by a mechanism as described above or by a similar mechanism – water movement through voids, piping or eroding fines over time to cause larger voids that eventually collapse. These failures, described as *sinkholes*, generally are catastrophic in nature, and costly to repair (construction and delay costs). A key element in the installation of piping systems is proper compaction beneath and around the pipe. Granular fill should always be used to form a haunch below the pipe for support. Some state agencies are using flowable fill or controlled low-strength material (CLSM) as an alternative to compacted granular fill (NCHRP Project 24-12). Without this support feature, the weight above the pipe will cause it to deform laterally, creating settlement above the pipe and often pipe collapse. Even if a sinkhole does not appear, leaks of any water bearing utility will inundate the adjacent pavement layers reducing their support capacity. Several agencies have used CLSM around pipes in the pavement section.

Pavement problems also occur when improper fill is used in the embankment beneath the pavement system. Placement of tree trunks, large branches and wood pieces in embankment fill must not be allowed. Over time, these organic materials decay, causing localized settlement and, eventually, voids in the soil. Again, water entering these voids can lead to collapse and substantial subsidence of the pavement section. Likewise, placement of large stones and boulders in fills creates voids in the mass, either unfilled due to bridging of soil over the large particles or filled with finer material that cannot be compacted with conventional equipment. Soil above these materials can pipe into the void space creating substantial subsidence in the pavement section. These issues can be mitigated with a well-crafted specification that will not allow these types of materials to be used, and full execution of the project QC/QA plan. (*e.g.,* Uniformity Coefficient, $C_u = D_{60}/D_{10} > 15$, <u>and</u> Coefficient of Curvature, $C_c = D_{30}^2/D_{10}D_{60} > 5$).

Special cases may require large stone (*e.g.,* blast rock or surge stone) to be used as fill. If such material must be used as fill, then select graded granular material and/or geotextiles should be placed above and below the large stone to form separator layers. For the use of a granular separation layer, the upper layer should be well compacted to choke off the voids in the stone. If possible, this layer should be flooded with a hose to confirm its compatibility. The gradation of the granular material must be such that it will not move into the void space and must also meet the filter criteria for the finer-grained fill material in the embankment. These conditions are met if the following gradation criteria are satisfied:

- D_{85} graded granular blanket $\geq 0.2\ D_{50}$ large stone fill
- D_{85} graded granular blanket $\leq 5\ D_{85}$ embankment fill
- D_{15} graded granular blanket $\geq 5\ D_{15}$ embankment fill

An alternative is to use high-survivability geotextiles (AASHTO M288 Class 1) placed immediately above and below the large stone layers to act as separators and prevent soil from moving into the void spaces in these materials.

Transitions between cut zones and fill zones can also create problems, particularly related to insufficient removal of weak organic material (clearing and grubbing), as well as neglect of subsurface water movements.

A specific transition also occurs at bridge approaches. These problems are typically related to inadequate compaction, typically a result of improper compaction equipment mobilized to the site, lack of supervision and care (*e.g.,* lift placement greater than compaction equipment can properly densify).

Many problems arise only after construction is completed and some amount of service life has been consumed. Detection of problems attributed to the geotechnical aspects of the pavement can be identified by interpretation of distresses observed. Examples of problems associated with flexible and rigid pavements are highlighted in Tables 8-5 and 8-6, respectively.

Pavement design and construction is an ongoing voyage. The industry has encountered numerous untold problems, and has found logical and economical solutions to mitigate these problems in the design stage, the construction stage, and the performance stage. Local agencies have developed their own strategies to solve anticipated problem and deal with unanticipated problems. This chapter was intended to summarize and discuss the majority of these issues.

Table 8-5. Geotechnical related post-construction problems in flexible pavements.

Problem/Distress Observed	Probable Cause(s)	Corrective Action
Longitudinal crack in wheel path (fatigue)	1. weak subgrade 2. insufficient pavement thickness	overlay
Surface rut in wheel path	1. over-stressed subgrade 2. post-construction densification of asphalt layer(s)	- leveling course in ruts and overlay - plane and overlay (*i.e.,* mill & fill)
Staining in surface cracks – color of stains consistent with local soil	drainage problems, wet subgrade – fines contamination in base, if present	- reconstruction - thick overlay (extend life somewhat, but mask the problem)
Intermittent depressions, subsidence	1. erratic compaction control 2. buried organic matter 3. piping in subsurface voids (*e.g.,* around utilities)	localized demolition and patching
Edge cracking	frost susceptibility and drainage issues	Construct wider shoulder; use materials that are not frost susceptible above the depth of frost penetration.

Table 8-6. Geotechnical related post-construction problems in rigid pavements.

Problem/Distress Observed	Probable Cause(s)	Corrective Action
Staining in surface cracks and/or joints – color of stains consistent with local soil	drainage problems, wet subgrade – fines contamination in base, if present, pumping fines	- reconstruction - thick overlay (extend life somewhat, but mask the problem)
Corner break	pumping of fines resulting in void or loss of support beneath slab	localized demolition and patching or reconstruction depending on extent of problem (mud-jacking and under sealing may be a good option if voids are detected prior to breaks)
Faulting at joints, subsidence of utility patches	1. erratic compaction control 2. buried organic matter 3. piping in subsurface voids (*e.g.,* around utilities) 4. pumping of fines resulting in void or loss of support beneath slab	localized demolition and patching or reconstruction depending on extent of problem (mud-jacking and under sealing may be a good option if voids are detected prior to faults, diamond grinding may be required for smoothness)

Example 8.1. A class exercise will be constructed around problems that are common to the agency. Each team will be given a specific pavement subgrade scenario and asked to identify the most appropriate soil improvement method(s) and describe the reasons for their selection. Other teams will critique the selection(s). This exercise would then be followed by slides summarizing the advantages and disadvantages of all of the soil improvement methods.

8.7 REFERENCES

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APPENDIX A. TERMINOLOGY

The following provides a definition of the pavement components, along with other terms common to the geotechnical aspects of pavements as contained in this manual. (Definitions were taken from NCHRP 1-37A, where available). The terms and definitions are organized under five general headings:

- Primary Pavement Components
- Geotechnical Pavement Components
- Non-Geotechnical Components
- Design Terminology
- Pavement Distress and Failure Terminology

Primary Pavement Components

subgrade - The top surface of a roadbed upon which the pavement structure and shoulders are constructed.

subbase - The layer or layers of specified or selected materials of designed thickness placed on a subgrade to support a base course. Note that the layer directly below the PCC slab is now called a base layer, not a subbase layer.

base - The layer or layers of specified or select material of designed thickness placed on a subbase or subgrade to support a surface course. The layer directly beneath the PCC slab is called the base layer.

surface course - One or more layers of a pavement structure designed to accommodate the traffic load, the top layer of which resists skidding, traffic abrasion, and the disintegrating effects of climate. The top layer of flexible pavements is sometimes called the "wearing" course.

Geotechnical Pavement Components

The **geotechnical components** of a pavement system as covered in this manual include unbound granular base, unbound granular subbase, the subgrade or roadbed, aggregate and geosynthetics used in drainage systems, graded granular aggregate and geosynthetic used as separation and filtration layers, and the roadway embankment foundation. Terms related to these components are defined as follows.

aggregate base (AB) - A base course consisting of compacted mineral aggregates. Also, granular base (GB), unbound granular base.

aggregate subbase (ASB) - A subbase course consisting of compacted mineral aggregates. Also, granular subbase, unbound granular subbase.

asphalt-treated permeable base (ATPB) - A base containing a small percentage of asphalt cement to enhance stability.

asphalt-treated permeable base (ATPB) - A permeable base containing a small percentage of asphalt cement to enhance stability. Also, asphalt-treated open-graded base (ATOGB), asphalt-treated base-permeable (ATB-Perm).

cement-treated base (CTB) - A base course consisting of mineral aggregates blended in place or through a pugmill with a small percentage of Portland cement to provide cementitious properties and strengthening. Also, aggregate cement, cement-stabilized graded aggregate (CSGA), cement-stabilized base (CSB).

cement-treated permeable base (CTPB) - An open-graded aggregate base treated with Portland cement to provide enhanced base strength and reduce erosion potential.

crushed stone base - A base course of designed thickness and constructed of graded and mechanically crushed mineral aggregate compacted above a subbase course or subgrade. Also, aggregate base (AB), graded aggregate base (GAB), and crushed aggregate (CA).

crushed stone subbase - A subbase course of designed thickness and constructed of graded and mechanically crushed mineral aggregate compacted above a subgrade.

dense-graded aggregate (DGA) - A mechanically crushed, well graded aggregate having a particle size distribution such that when it is compacted, the resulting voids between the aggregate particles, expressed as a percentage of the total space occupied by the material, are relatively small.

drainable granular subbase - A subbase constructed of compacted and crushed open-graded aggregate.

geogrid (GG) - a geosynthetic formed by a regular network of tensile elements with apertures of sufficient size to interlock with surrounding fill material, used primarily as reinforcement of base and subbase layers and in stabilization of soft subgrade layers. Also used in overlays for asphalt reinforcement.

geosynthetic - a planar product manufactured from a polymeric material used with soil, rock, earth, or other geotechnical-related material as an integral part of a civil engineering project, structure, or system.

geotextile (GT) - a permeable geosynthetic made of textile materials, used as a separator between base, subbase and subgrade layers, used as filters in drainage features, and used in stabilization of soft subgrade layers. Also used in asphalt overlays as a membrane absorption and/or waterproofing layer.

gravel - Coarse aggregate resulting from natural disintegration and abrasion of rock or processing of weakly bound conglomerate. In geotechnical engineering, the particles of rock that range in size from 76.2 mm (3-in. U.S. sieve) to 4.75 mm (No. 4 U.S. sieve). To be classified as a gravel in the Unified Classification System (UCS), at least 50% of the material must be in this range. (Identification and classification of soils is covered in Chapter 5.)

gravel base - An unbound base course constructed of compacted gravel. May or may not be graded and/or crushed.

gravel subbase - An unbound subbase course constructed of compacted gravel. May or may not be graded and/or crushed.

gravel subgrade - A subgrade where a natural gravel has been used as the roadbed surface or where the native soil has been blended with a gravel additive (a.k.a. gravel-treated subgrade for the second case).

lime-treated subgrade - A prepared and mechanically compacted mixture of hydrated lime, water, and soil supporting the pavement system.

lime-flyash base (LFB or LFA) - A blend of mineral aggregate, lime, flyash, and water, combined in proper proportions and producing a dense mass when compacted.

modified or treated base - The addition of cement or asphalt (typically less than 5%) to unbound base with the primary purpose of improving the stability for construction (*i.e.,* no improvement anticipated for stiffness or structural support).

open-graded aggregate base (OGAB) - A crushed mineral aggregate base having a particle size distribution such that when compacted the interstices will provide enhanced drainage properties. Also, granular drainable layer, untreated permeable base (UPB).

permeable base (PB) - A base course constructed of treated or untreated open-graded aggregate. Also, free-draining base.

prefabricated geocomposite edge drain (PGED) - An edgedrain consisting of an extruded plastic drainage core covered with a geotextile filter (also known as panel drains or fin drains).

roadbed - The graded portion of a highway between top and side slopes, prepared as a foundation for the pavement structure and shoulder.

roadbed material - The material below the subgrade in cuts and in embankment foundations, extending to such depth as affects the support of the pavement structure.

soil aggregate - Natural or prepared mixtures consisting predominantly of stone, gravel, or sand that contain a significant amount of minus 75-µm (No. 200) silt-clay material.

soil cement - A mechanically compacted mixture of soil, Portland cement, and water, used as a layer in a pavement system to reinforce and protect the subgrade or subbase. Also, cement-treated subgrade (CTS).

stabilized granular base - A base course with an unspecified stabilizing material, usually asphalt cement or Portland cement.

stabilized permeable base - A permeable base with an unspecified stabilizing material, usually asphalt cement or Portland cement. Also, bound drainable base.

subgrade - the top surface of a roadbed upon which the pavement structure and shoulders are constructed with the purpose of providing a platform for construction of the pavement and to support the pavement without undue deflection that would impact the pavements performance (NCHRP 1-37A). In this manual, the natural and/or prepared soil materials beneath the pavement structure that deform under pavement loading or otherwise have an influence on the support of the pavement (a.k.a. roadbed, pavement foundation).

Unbound base/subbase - compacted mineral aggregate layer that may be either untreated or treated, but has not been modified sufficiently to provide an increase in stiffness or strength for design.

Non-Geotechnical Components

As indicated in Section 1.1, the **non-geotechnical components** are the surficial pavement layers, including asphaltic concrete, Portland cement concrete, and bound aggregate layers. Terms related to these components are defined as follows:

asphalt concrete (AC) - A controlled mixture of asphalt cement and graded aggregate compacted to a dense mass. Also, hot-mixed asphalt (HMA), hot-mixed asphalt concrete (HMAC), bituminous concrete (BC), plant mix (PM).

asphalt concrete base (ACB) - Asphalt concrete used as a base course. Also, asphalt base course (ABC), asphalt-stabilized base, hot-mixed (ASB-HM), asphalt-treated base (ATB), bituminous aggregate base, bituminous concrete base (BCB), bituminous base (BB), hot-mixed asphalt base (HMAB).

asphalt concrete pavement (ACP) - A pavement structure placed above a subgrade or improved subgrade and consisting of one or more courses of asphalt concrete or a combination of asphalt concrete and stabilized or unstabilized aggregate courses.

asphalt concrete surface (ACS) - Asphalt concrete used as a surface course. Also, dense-graded asphalt concrete (DGAC).

continuously reinforced concrete pavement (CRCP) - Portland cement concrete pavement with no transverse joints and containing longitudinal steel in an amount designed to ensure holding shrinkage cracks tightly closed. Joints exist only at construction joints and on-grade structures.

flexible pavement - A pavement structure that maintains intimate contact with and distributes loads to the subgrade and depends on aggregate interlock, particle friction, and cohesion for stability.

jointed plain concrete pavement (JPCP) - Jointed Portland cement concrete pavement containing no distributed steel to control random cracking; may or may not contain joint load transfer devices.

jointed reinforced concrete pavement (JRCP) - Jointed Portland cement concrete paving containing distributed steel reinforcement to control random cracking and usually containing joint load transfer devices.

lean concrete base (LCB) - A base course constructed of mineral aggregates plant mixed with a sufficient quantity of Portland cement to provide a strong platform for additional pavement layers and placed with a paver.

plain concrete - PCC without reinforcing steel.

Portland cement concrete (PCC) - A composite material consisting of a Portland or hydraulic cement binding medium and embedded particles or fragments of aggregate.

rigid pavement - A pavement structure that distributes loads to the subgrade, having as one course a Portland cement concrete slab of relatively high-bending resistance.

Design Terminology

In the context of current design practice, pavement designers and geotechnical specialists must communicate using **design terms** with consistent definitions. Terms related to design as used in this manual include

analysis period - (a.k.a. performance period) The time period used for comparing design alternatives. An analysis period may contain several maintenance and rehabilitation activities during the life cycle of the pavement being evaluated.

average annual daily traffic (AADT) - The estimate of typical traffic on a road segment for all days of the week over the period of a year.

average annual daily truck traffic (AADTT) - The estimate of typical truck traffic on a road segment for all days of the week over the period of a year.

axle load - The sum of all tire loads on an axle.

axle load spectrum - The full spectrum (distribution) of single, dual, triple, and other axle loads applied to a pavement structure by a given traffic stream.

bound base - The addition of a sufficient amount of cement or asphalt to change the long term stiffness and structural characteristics of unbound base to that of lean concrete.

design life - The length of time for which a pavement structure is being designed, including the time from construction until major programmed rehabilitation.

elastic layer theory - A mathematical process wherein the layers of a pavement structure are all assumed to behave elastically.

equivalent single axle load (ESAL) - A numerical factor that expresses the relationship of a given axle load to another axle load in terms of the relative effects of the two loads on the serviceability of a pavement structure. Often expressed in terms of 18,000-pound (80 kN) single axle loads.

finite element analysis - The finite element method is one wherein rigorous mathematical solution, often employing complex differential equations, of an engineering problem is approximated algebraically. The geometry of the problem is described by discrete elements of finite dimensions that are analyzed through the application of engineering mechanics principles. Results of the finite element analyses are aggregated to approximate the exact mathematical solution.

international roughness index (IRI) - A pavement roughness index computed from a longitudinal profile measurement using a quarter-car simulation at a simulation speed of 50 mph (80 km/h).

life-cycle cost analysis (LCCA) - An economic assessment of an item, area, system, or facility and competing design alternatives considering all significant costs of ownership over the economic life (which encompasses several analysis periods), expressed in equivalent dollars.

longitudinal profile - The perpendicular deviations of the pavement surface from an established reference parallel to the lane direction, usually measured in the wheel tracks.

mechanistic-empirical (M-E) - A design philosophy or approach wherein classical mechanics of solids is used in conjunction with empirically derived relationships to accomplish the design objectives.

pavement performance - Measure of accumulated service provided by a pavement (*e.g.,* the adequacy with which it fulfills it purpose). Often referred to as the record of pavement condition or serviceability over time or with accumulated traffic.

performance period - The period of time that an initially constructed or rehabilitated pavement structure will last (perform) before reaching its terminal condition when rehabilitation is performed. This is also referred to as the design period.

present serviceability index (PSI) - An index derived by formula for estimating the serviceability rating from measurements of physical features of the pavement.

present serviceability rating (PSR) - A mean rating of the serviceability of a pavement (traveled surface) established by a panel rating under controlled conditions. The usual scale for highways is 0 to 5, with 5 being excellent.

reliability - The probability that a given pavement design will last for the anticipated design performance period.

rideability - A subjective judgment of the comparative discomfort induced by traveling over a specific section of highway pavement in a vehicle.

serviceability - The ability at time of observation of a pavement to serve traffic (autos and trucks) that uses the facility.

single axle load - The total load transmitted by all wheels whose centers may be included between two parallel transverse vertical planes 1 m (40 in.) apart, extending across the full width of the vehicle.

tandem axle load - The total load transmitted to the pavement by two consecutive axles whose centers may be included between parallel vertical planes.

traffic growth factor - A factor used to describe the annual growth rate of traffic volume on a roadway.

transverse profile - The vertical deviations of the pavement surface from a horizontal reference perpendicular to the lane direction.

user costs - Those costs realized by the users of a facility. In a life cycle cost analysis, user costs could take the form of delay costs or of changes in vehicle operating costs associated with various alternatives.

weigh-in-motion (WIM) - The process of estimating a moving vehicle's gross weight and the portion of that weight that is carried by each wheel, axle, or axle group, or combination thereof, by measurement and analysis of dynamic forces.

wheel load - The sum of the tire loads on all tires included in the wheel assembly comprising a half axle.

zero-stress temperature - temperature (after placement and during curing) at which the concrete layer exhibits zero thermal stress (at temperatures less than this, concrete exhibits tensile stress).

Pavement Distress and Failure Terminology

Distress refers to conditions that reduce serviceability or indicate structural deterioration. Failure is a relative term. In the context of this manual, failure denotes a pavement section that experiences excessive rutting or cracking that is greater than anticipated during the performance period or that a portion of the pavement is structurally impaired at any time during the performance period with incipient failure anticipated from the local distress. There are a number of ways that a pavement section can fail.

alligator cracking - Interconnected or interlaced cracks forming a pattern that resembles an alligator's hide. Also, map cracking.

blowup - An upward eruption of a PCC pavement slab near a crack or joint.

crack - A fissure or discontinuity in the pavement surface not necessarily extending through the entire thickness of the pavement.

fatigue cracking - Cracking of the pavement surface as a result of repetitive loading; may be manifested as longitudinal or alligator cracking in the wheel paths for flexible pavement and transverse cracking (and sometimes longitudinal cracking) for jointed concrete pavement.

faulting - Elevation or depression of a PCC slab in relation to an adjoining slab, usually at transverse joints and cracks.

liquefaction - the process of transforming any soil from a solid state to a liquid state, usually as a result of increased pore pressure and reduced shearing resistance (ASTM, 2001). Spontaneous liquefaction may be caused by a collapse of the structure by shock or other type of strain and is associated with a sudden but temporary increase of the prefluid pressure.

longitudinal cracking - Pavement cracking predominantly parallel to the direction of traffic.

pumping - The ejection of foundation material, either wet or dry, through joints or cracks, or along edges of rigid slabs resulting from vertical movements of the slab under traffic, or from cracks in semi-rigid pavements.

punchouts - A broken area of a CRCP bounded by closely spaced cracks usually spaced less than 1 m (3 ft).

random cracking - Uncontrolled and irregular fracturing of a pavement layer.

raveling - A pavement distress characterized by the loss of surface material involving the dislodgment of aggregate particles and degradation of the binder material.

reflective cracking - Cracks in asphalt or concrete surfaces of pavements, occurring over joints or cracks in the underlying layers.

rutting - Longitudinal depression or wearing away of the pavement in wheel paths under load.

spalling - The cracking, breaking, or chipping of pavement edges in the vicinity of a joint or crack.

thermal cracking - Cracks in an asphalt pavement surface, usually full width transverse, as a result of seasonal or diurnal volume changes of the pavement restrained by friction with an underlying layer.

transverse cracking - Pavement cracking predominantly perpendicular to the direction of traffic.

warping - Deformation of a PCC slab caused by a moisture or temperature differential between the upper and lower surfaces.

APPENDIX B: MAIN HIGHWAY PROJECT

B.1 INTRODUCTION

The Main Highway, the example project for the design exercises in this manual, involves the reconstruction and upgrading of an existing county road in the upper northeastern United States. This appendix summarizes the geotechnical and other data available for the design of this project.

A topographic map showing the project horizontal alignment is shown in Figure B-1. The length of the entire reconstruction project is 1.9 miles. The project subsurface investigation consisted of 10 test pits, 10 power auger borings, 5 hand auger borings, and 21 hand rod soundings. Seventeen soil samples were collected and tested in the laboratory.

For the purposes of the design examples in this manual, only a 1500-foot-long subsection of the entire project will be considered. This subsection between Sta. 255+00 and 270+00 is indicated in Figure B-2. Subsequent sections of this appendix summarize the relevant design data for this subsection.

B.2 SUBSURFACE CONDITIONS

A soils map for the site is shown in Figure B-2. Detailed plan and profile views of the subsection of interest are shown in Figure B-3 through Figure B-6. The locations of the various borings, test pits, and soundings are also indicated on these figures. Logs from borings within the subsection of interest are summarized in Figure B-7 through Figure B-14. Similar observations from the test pits are provided in Figure B-15 through Figure B-17.

B.2.1 General Conditions

The dominant soil type along this project is anticipated to consist of clay silt, with some remnants of the existing base material from the existing county road. The natural subgrade soils are all plastic and have been classified as an AASHTO A-4 or A-6 soil, with a frost rating of IV or III. Several undesirable soil conditions may be encountered along this project. These include soil support, drainage, and slumping.

Soil Support. The group index value is used as a general guide to the load bearing characteristics of a soil. It is a function of the liquid limit, plasticity index, and the amount of material passing the No. 200 sieve. Zero indicates good subgrade material, whereas a group index value of 20 or more indicates a poor subgrade material. Group index values obtained on the moist clay silts along this project ranged between 16.3 and 27.3, with an average of 21.6. CBR tests run on three samples (3779[1], 90021, 90022[1]) produced soil support values of 2.6, 2.8, and 2.2, respectively. Based on this, it is anticipated that these soils will have a low bearing capacity resulting in poor soil support. It is anticipated that a standard 30 inch section will not provide adequate structural support. Due to the poor bearing capacity of the clay soils, it has been determined that an 18 inch lift of granular material will be required in addition to the structural aggregate base member. This 18 inch granular lift will provide a working platform for construction equipment to operate upon.

Drainage. Drainage of base and subgrade soils is extremely important along this project due to the presence of clay silt soils along the subgrade. The strength of the clay soils along this project will be dramatically affected by the presence of water. These soils have a high volume change between wet and dry states and will shrink and swell with changes in moisture content. Clay silt soils have a high dry strength but lose much of this strength upon absorbing water. Unfortunately, these soils are poorly drained and may absorb water by capillary action, resulting in low bearing capacity.

Slumping. The clay silt soils along this project have a tendency to slump with slopes greater than 2:1. At times, depending on seasonal conditions, these soils may even slump at slopes shallower than 2:1. It is anticipated that stone ditch protection and stone protection of some backslopes will be required. Vegetation of all exposed soil areas will be very important.

Additional relevant comments and observations from the geotechnical investigation report for this project are as follows:

- It is recommended that construction activities requiring heavy equipment operation on the native subgrade soils not be attempted during early to mid spring due to anticipated moist, soft soils.
- No substantial embankments or cuts are proposed along the project. However, some small embankments or cuts (less than about 5 ft) are proposed over/through the clay and silty clay native soils.

[1] From locations outside of study section.

- Bedrock was not encountered along the project and thus no rock excavation or shallow rock conditions are anticipated.

B.2.2 Detailed Conditions along Study Section (Sta. 255+00 to 270+00)

This section consists of several cut and fill areas with a maximum cut of 4.0 feet and a maximum fill of 3.4 feet along the proposed centerline. Field explorations within this section consist of 2 power auger borings, 3 test pits, and 3 hand auger borings.

Earth excavation is anticipated to consist of existing base material (403) and clay silt (90021, 3777). The base material is classified as an AASHTO A-1-b soil, with a frost class of I. The clay silts are plastic and have been classified as A-6 soils, with a frost class of III. Depending on field conditions with respect to moisture, the clay silts along the surface may be moist to wet and softer than the underlying clay silts.

Subgrade soils are anticipated to consist of existing base material (403) and clay silt (90021, 3777). It is anticipated that most of the clay silts encountered at subgrade within the proposed cut sections may be a little firmer than the overlying clay silts. However, this is based upon the seasonal conditions at the time of the field explorations, and these conditions could change dramatically by the time of construction.

As previously discussed in other sections, drainage is extremely important with respect to the load bearing characteristics of the clay silt subgrade soils. The existing poor pavement conditions in this section can be attributed to the lack of adequate drainage and poor subgrade soils. It has been recommended that all proposed ditching be deepened to 2 feet below proposed subgrade wherever possible.

B.2.3 Laboratory Test Data

The sample log for all test specimens along the study subsection (Sta. 255+00 to 270+00) is given in Table B-1. The following laboratory test information is available:

- Gradation curves for several samples, including some from outside the study section (Figure B-18 and Figure B-19).
- Compaction curves and CBR value for the clayey silt subgrade (Figure B-20 and Figure B-21).
- Compaction curves for the granular working pad (Figure B-22).

- Laboratory resilient modulus values for the clayey silt subgrade (Table B-2) and the granular base material (Table B-3). Table B-4 summarizes the corresponding stress-dependent resilient modulus parameters k_1, k_2, and k_3 for the NCHRP 1-37A level 1 M_R relation:

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(1 + \frac{\tau_{oct}}{p_a}\right)^{k_3}$$

in which

M_R = subgrade resilient modulus (same units as p_a)

θ = bulk stress = $\sigma_1+\sigma_2+\sigma_3$ (same units as p_a)

τ_{oct} = octahedral shear stress = $\frac{1}{3}\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_1-\sigma_3)^2}$ (same units as p_a)

p_a = atmospheric pressure (to make equation dimensionless)

k_1,k_2,k_3 = material properties with constraints $k_1>0$, $k_2 \geq 0$, $k_3 \leq 0$

B.2.4 Field Test Data

Several sets of FWD tests were performed at various times during the years prior to construction. These tests were used to backcalculate the subgrade resilient modulus and pavement effective modulus. These are summarized in Table B-5. The subgrade resilient modulus M_R is calculated from the FWD results using the standard AASHTO equations:

$$M_R = \frac{0.24P}{d_r r}$$

$$r \geq 0.7a_e$$

$$a_e = \sqrt{\left[a^2 + \left(D\sqrt[3]{\frac{E_p}{M_R}}\right)^2\right]}$$

in which

M_R = back-calculated subgrade resilient modulus (psi)

P = applied load (pounds)

d_r = deflection at a distance r from the center of the load (inches)

r = distance from the center of the load (inches)

a_e = radius of the stress bulb at the subgrade-pavement interface (inches)

a = load plate radius (inches)

D = total pavement thickness above the subgrade (inches)

The effective pavement modulus E_p is determined from:

$$d_0 = 1.5pa\left\{\frac{1}{M_R\left[\sqrt{1+\left(\frac{D}{a}\sqrt[3]{\frac{E_p}{M_R}}\right)}\right]^2}+\frac{1-\frac{1}{\sqrt{1+\left(\frac{D}{a}\right)^2}}}{E_p}\right\}$$

in which

d_0 = deflection measured at the center of the load plate, adjusted to a standard temperature of 68°F (inches)

p = load plate pressure (psi)

and the other terms as previously defined. Subgrade resilient modulus for design purposes is usually less than the value back-calculated from FWD data. The AASHTO design guide recommends a design subgrade modulus equal to 33% of the FWD value for flexible pavements and 25% of the FWD value for rigid pavements.

B.3 ENVIRONMENTAL CONDITIONS

The site is located in the northern United States in a cold and wet climate. Freezing Index and frost penetration estimates for the project site are summarized in Table B-6.

B.4 TRAFFIC

Traffic estimates for use in design are summarized in Table B-7. The average traffic level is approximately 750 ESALs per day.

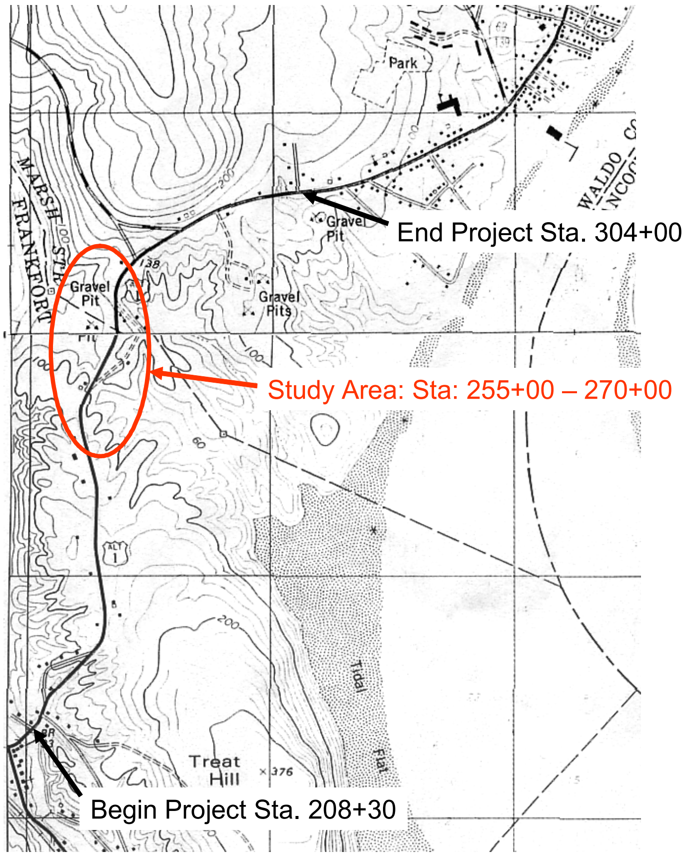

Figure B-1. Project alignment.

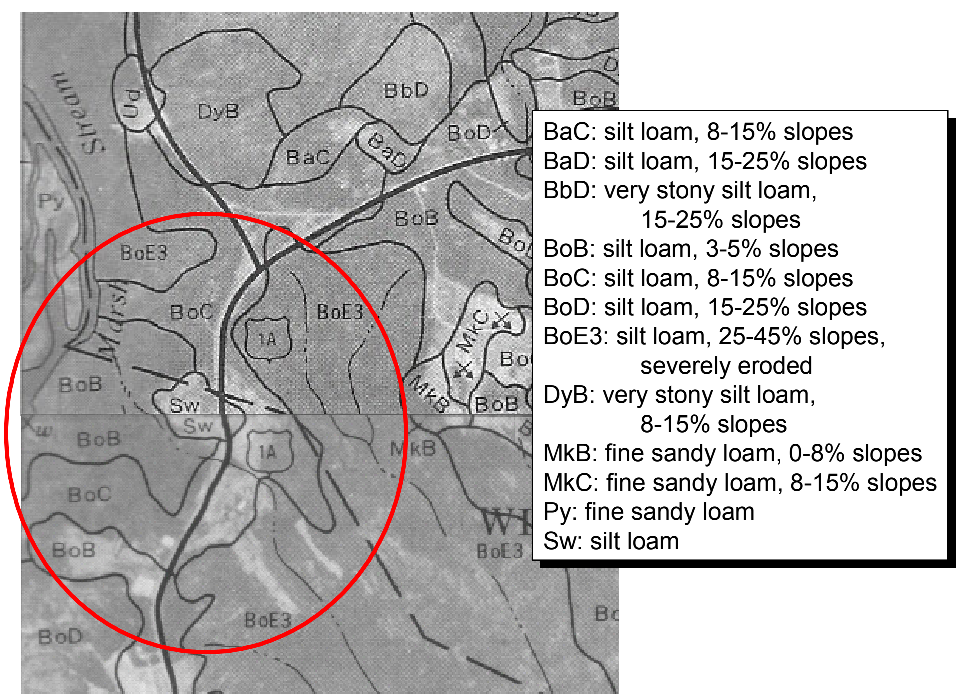

Figure B-2. Soils map for project site.

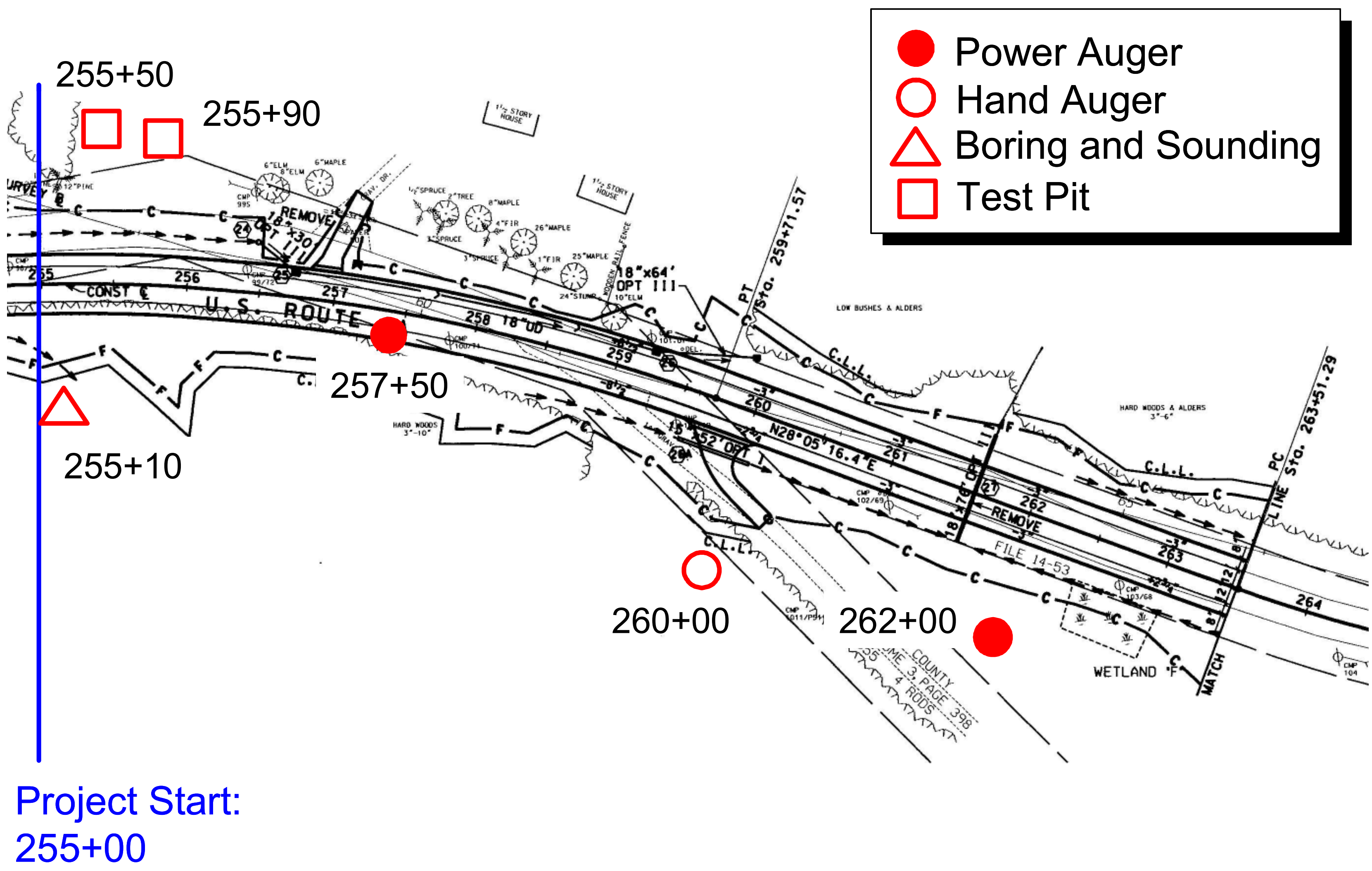

Figure B-3. Horizontal alignment with subsurface exploration locations: Sta. 255+00 to 263+50.

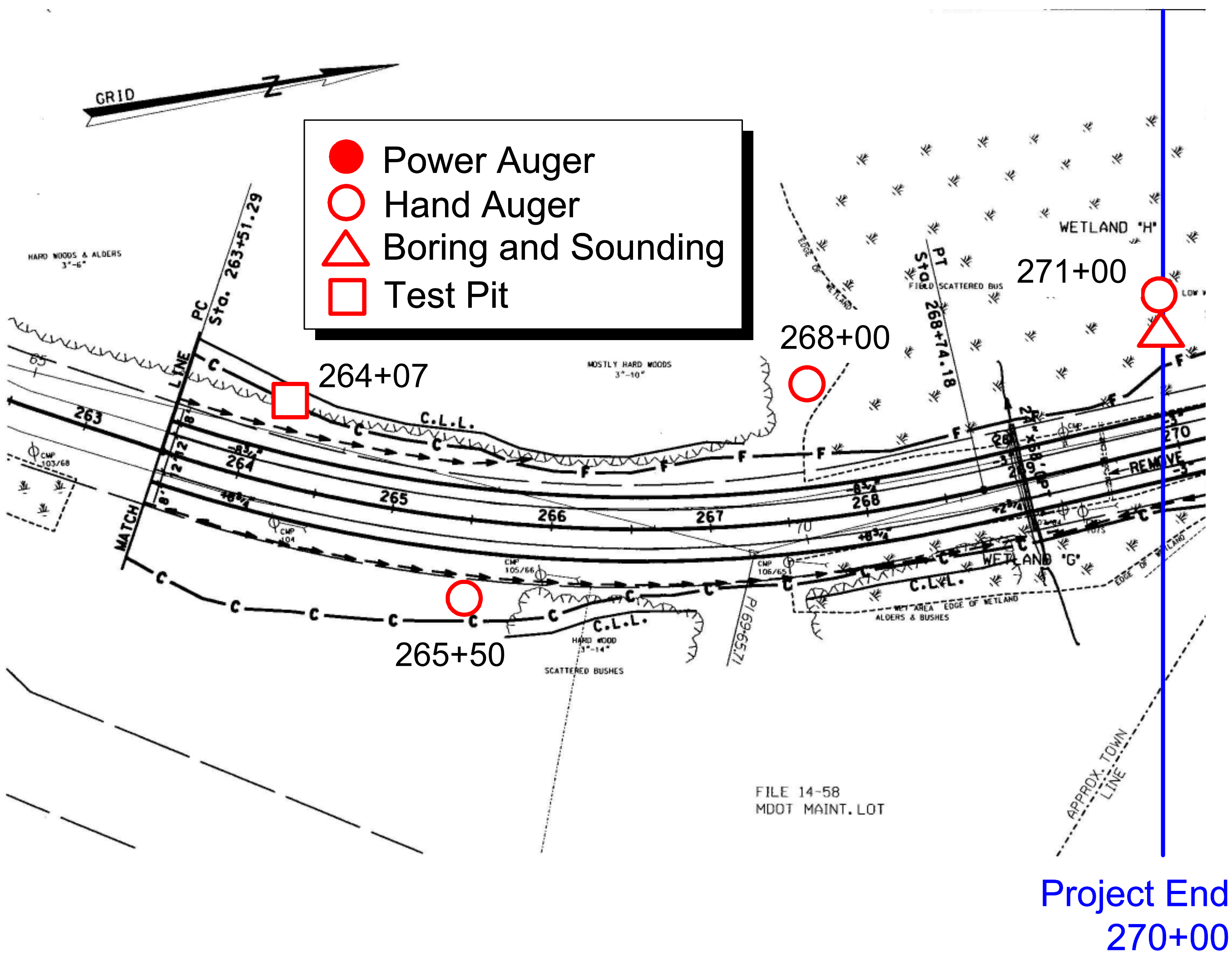

Figure B-4. Horizontal alignment with subsurface exploration locations: Sta. 263+50 to 270+00.

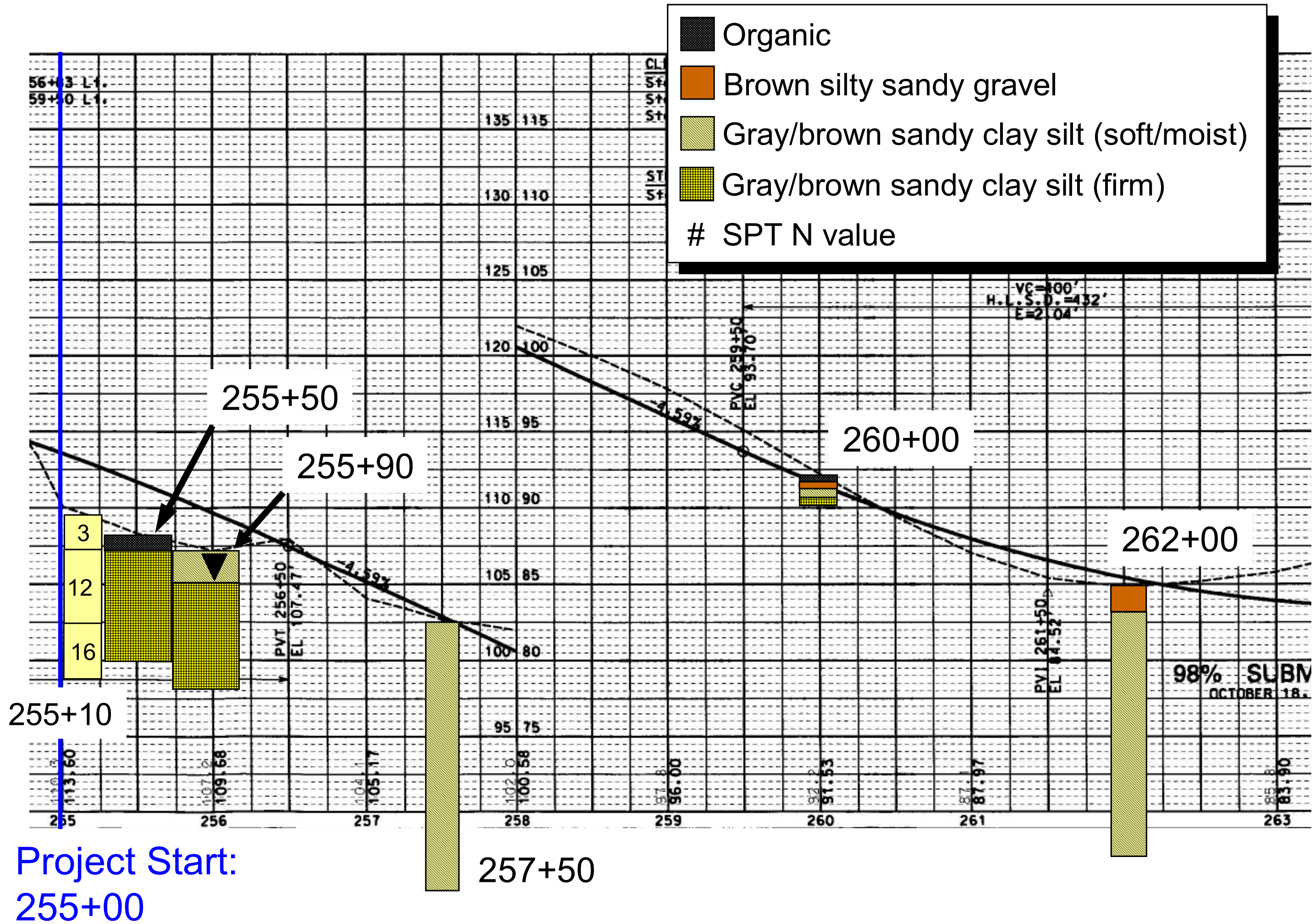

Figure B-5. Vertical alignment with subsurface exploration findings: Sta. 255+00 to 263+50.

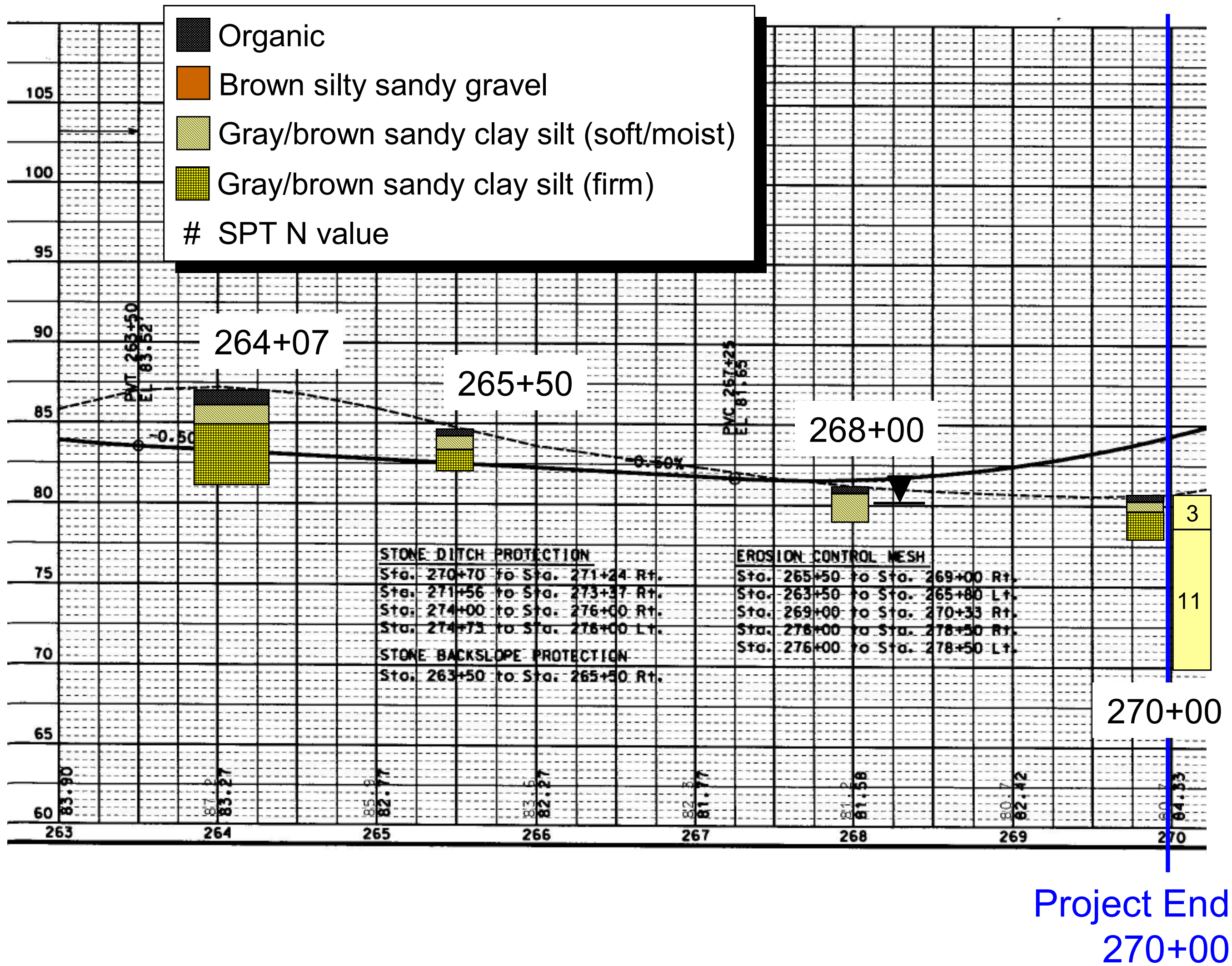

Figure B-6. Vertical alignment with subsurface exploration findings: Sta. 263+50 to 270+00.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	**Log of Boring 255+10** Sheet 1 of 1

Date(s) Drilled 5/10/95	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Boring and Sounding	Drill Bit Size/Type	Total Depth of Borehole 12
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 109.5
Groundwater Level and Date Measured	Sampling Method(s) None	Hammer Data
Borehole Backfill	Location Station 255+10, Offset 30R	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
110	0			2					Note: Advacent bank washed out below culvert.
				6					
				14					
				12					
				14					
105	5			12					
				12					
				15					
				16					
				16					
100	10			16					
				27					
							Bottom of Boring at 12 feet bgs. No refusal.		
95	15								
90	20								
85	25								
80	30								

Figure B-7. Boring log, station 255+10.

Project: Main Highway **Project Location: Hometown** **Project Number: FHWA NHI 132040**	**Log of Boring 257+50** Sheet 1 of 1

Date(s) Drilled **5/10/95**	Logged By **Joe Engineer**	Checked By **Jane P. Manager**
Drilling Method **Power Auger**	Drill Bit Size/Type	Total Depth of Borehole **19.5 feet bgs**
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation **102.5**
Groundwater Level and Date Measured	Sampling Method(s) **Bulk**	Hammer Data
Borehole Backfill	Location **Station 257+50, Offset 5R**	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
102	0				A-6		Moist brown sandy clay silt		
97	5								
		Bulk	90021					24	
92	10								
87	15								
82	20						Bottom of Boring at 19.5 feet bgs. No refusal.		
77	25								
72	30								

Figure B-8. Boring log, station 257+50.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	**Log of Boring 260+00** Sheet 1 of 1

Date(s) Drilled 7/11/95	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Hand Auger	Drill Bit Size/Type	Total Depth of Borehole 1.9
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 92
Groundwater Level and Date Measured	Sampling Method(s) None	Hammer Data
Borehole Backfill	Location Station 260+00, Offset 40R	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
92	0						Moist rooted organic matter		
					A-7-5		Moist rooted silty pebbly sand		
					A-4		Rooted moist gray clay silt w/ some organics (similar to sample 232)		
					A-4		Moist mottled sandy silt (similar to sample 233)		
							Firm moist mottled clay silt (similar to sample 233)		
							Bottom of Boring at 1.9 feet bgs. No refusal.		
87	5								
82	10								
77	15								
72	20								
67	25								
62	30								

Figure B-9. Boring log, station 260+00.

Project: Main Highway **Project Location: Hometown** **Project Number: FHWA NHI 132040**	**Log of Boring 262+00** Sheet 1 of 1

Date(s) Drilled **5/10/95**	Logged By **Joe Engineer**	Checked By **Jane P. Manager**
Drilling Method **Power Auger**	Drill Bit Size/Type	Total Depth of Borehole **19.5**
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation **85**
Groundwater Level and Date Measured	Sampling Method(s) **None**	Hammer Data
Borehole Backfill	Location **Station 262+00, Offset 30R**	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
85	0						Brown silty sandy gravel		
					A-4		Moist gray brown sandy clay silt (similar to sample 3777)		
80	5								
75	10								
70	15								
65	20						Bottom of Boring at 19.5 feet bgs. No refusal.		
60	25								
55	30								

Figure B-10. Boring log, station 262+00.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	**Log of Boring 265+50** Sheet 1 of 1

Date(s) Drilled 7/11/95	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Hand Auger	Drill Bit Size/Type	Total Depth of Borehole 2.3
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 84.5
Groundwater Level and Date Measured 1.3 bgs	Sampling Method(s) Bulk	Hammer Data
Borehole Backfill	Location Station 265+50, Offset 30R	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
84	0		232		A-7-5		Moist rooted organic matter.	92	
			234		A-4		Moist rooted clay silt with some organics. (ATD)		
					A-4		Moist pebbly sandy clay silt.		
							Moist mottled sandy gray silt (similar to sample 233).		
							Bottom of Boring at 2.3 feet bgs. No refusal.		
79	5								
74	10								
69	15								
64	20								
59	25								
54	30								

Figure B-11. Boring log, station 265+50.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	**Log of Boring 268+00** Sheet 1 of 1

Date(s) Drilled 7/11/95	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Hand Auger	Drill Bit Size/Type	Total Depth of Borehole 2.8
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 81
Groundwater Level and Date Measured	Sampling Method(s) None	Hammer Data
Borehole Backfill	Location Station 268+00, Offset 30L	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
81	0				A-7-5		Moist rooted organic matter		
					A-4		Rooted moist gray clay silt (less organic than sample 232)		
							Firm moist mottled sandy clay silt (similar to sample 233)		
							Bottom of Boring at 2.8 feet bgs. No refusal.		
76	5								
71	10								
66	15								
61	20								
56	25								
51	30								

Figure B-12. Boring log, station 268+00.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	**Log of Boring 270+00** Sheet 1 of 1

Date(s) Drilled 7/11/95	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Hand Auger	Drill Bit Size/Type	Total Depth of Borehole 2.4
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 80.5
Groundwater Level and Date Measured	Sampling Method(s) Bulk	Hammer Data
Borehole Backfill	Location Station 270+00, Offset 35L	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
80	0				A-7-5		Moist rooted organic matter.		
			233		A-4		Rooted moist gray clay silt w/ some organics (similar to sample 232).	23	
							Mottled moist clay silt (firm)		
							Bottom of Boring at 2.4 feet bgs. No refusal.		
75	5								
70	10								
65	15								
60	20								
55	25								
50	30								

Figure B-13. Boring log, station 270+00.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	**Log of Boring 270+10** Sheet 1 of 1

Date(s) Drilled 7/11/95	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Boring and Sounding	Drill Bit Size/Type	Total Depth of Borehole 12
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 81
Groundwater Level and Date Measured	Sampling Method(s) None	Hammer Data
Borehole Backfill	Location Station 270+10, Offset 30L	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
81	0			2					
				4					
				12					
				9					
				10					
76	5			9					
				11					
				9					
				14					
				12					
71	10			10					
				9					
							Bottom of Boring at 12 feet bgs. No refusal.		
66	15								
61	20								
56	25								
51	30								

Figure B-14. Boring log, station 270+10.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	Log of Boring 255+50 (Pit) Sheet 1 of 1

Date(s) Drilled 11/5/92	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Test Pit	Drill Bit Size/Type	Total Depth of Borehole 10 feet bgs
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 108
Groundwater Level and Date Measured	Sampling Method(s) None	Hammer Data
Borehole Backfill	Location Station 255+50, Offset 30L	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
108	0						Loose brown sandy clay silt.		
							Firm gray brown clay silt. Note: Soil becomes slightly moist with depth, but no seepage visible in test pit.		
103	5								
98	10						Bottom of test pit at 10 feet bgs.		
93	15								
88	20								
83	25								
78	30								

Figure B-15. Test pit log, station 255+50.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	Log of Boring 255+90 (Pit) Sheet 1 of 1

Date(s) Drilled 6/14/90	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Test Pit	Drill Bit Size/Type	Total Depth of Borehole 10 feet bgs
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 103
Groundwater Level and Date Measured 2.5 bgs	Sampling Method(s) None	Hammer Data
Borehole Backfill	Location Station 255+90, Offset 35L	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
103	0						Moist soft gray brown clay silt w/some sand, pebbles at shallow depth.		
							(ATD) ▽		
							Firm gray brown clay silt.		
98	5								
93	10						Bottom of test pit at 10 feet bgs.		
88	15								
83	20								
78	25								
73	30								

Figure B-16. Test pit log, station 255+90.

Project: Main Highway **Project Location: Hometown** **Project Number: FHWA NHI 132040**	**Log of Boring 264+07 (Pit)** Sheet 1 of 1

Date(s) Drilled **6/14/90**	Logged By **Joe Engineer**	Checked By **Jane P. Manager**
Drilling Method **Test Pit**	Drill Bit Size/Type	Total Depth of Borehole **7**
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation **87**
Groundwater Level and Date Measured	Sampling Method(s) **Bulk**	Hammer Data
Borehole Backfill	Location **Station 264+07, Offset 20L**	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
87	0						Rooted clay silt topsoil.		
							Soft moist gray brown clay silt.		
					A-4		Firm gray brown clay silt.		
			3777					24	
82	5								
							Bottom of test pit at 7 feet bgs.		
77	10								
72	15								
67	20								
62	25								
57	30								

Figure B-17. Test pit log, station 264+07.

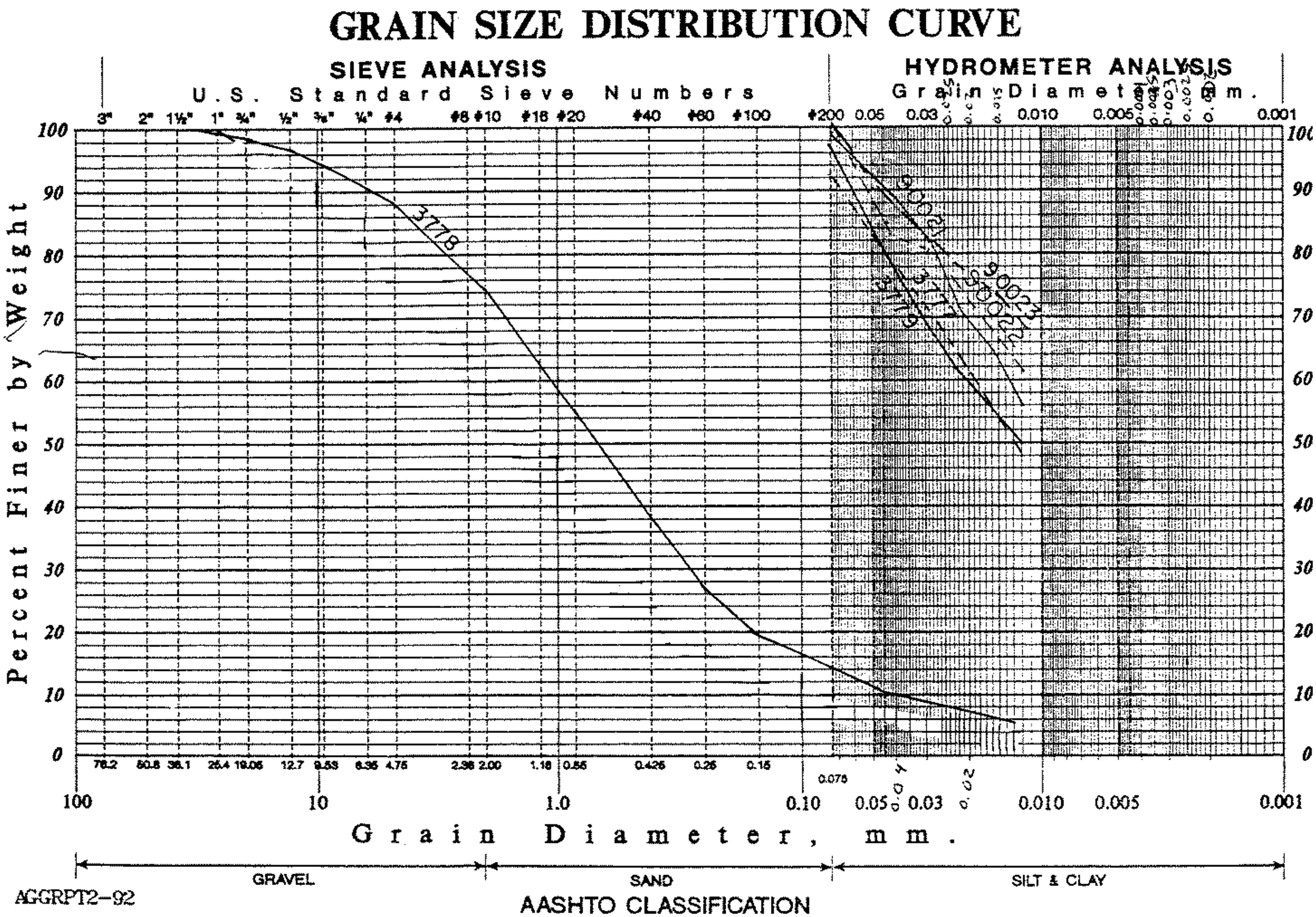

Figure B-18. Grain size distribution curves.

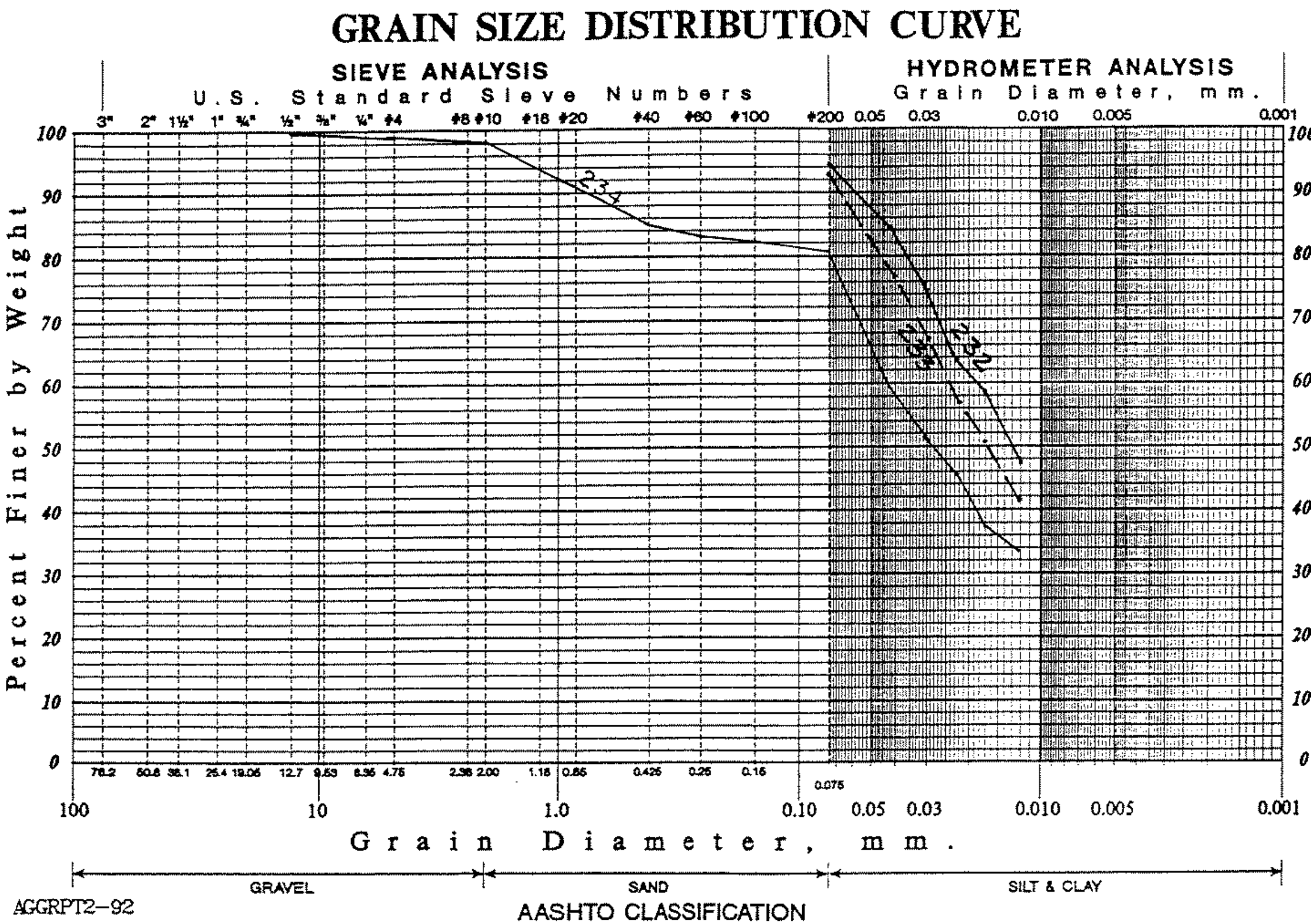

Figure B-19. Grain size distribution curves.

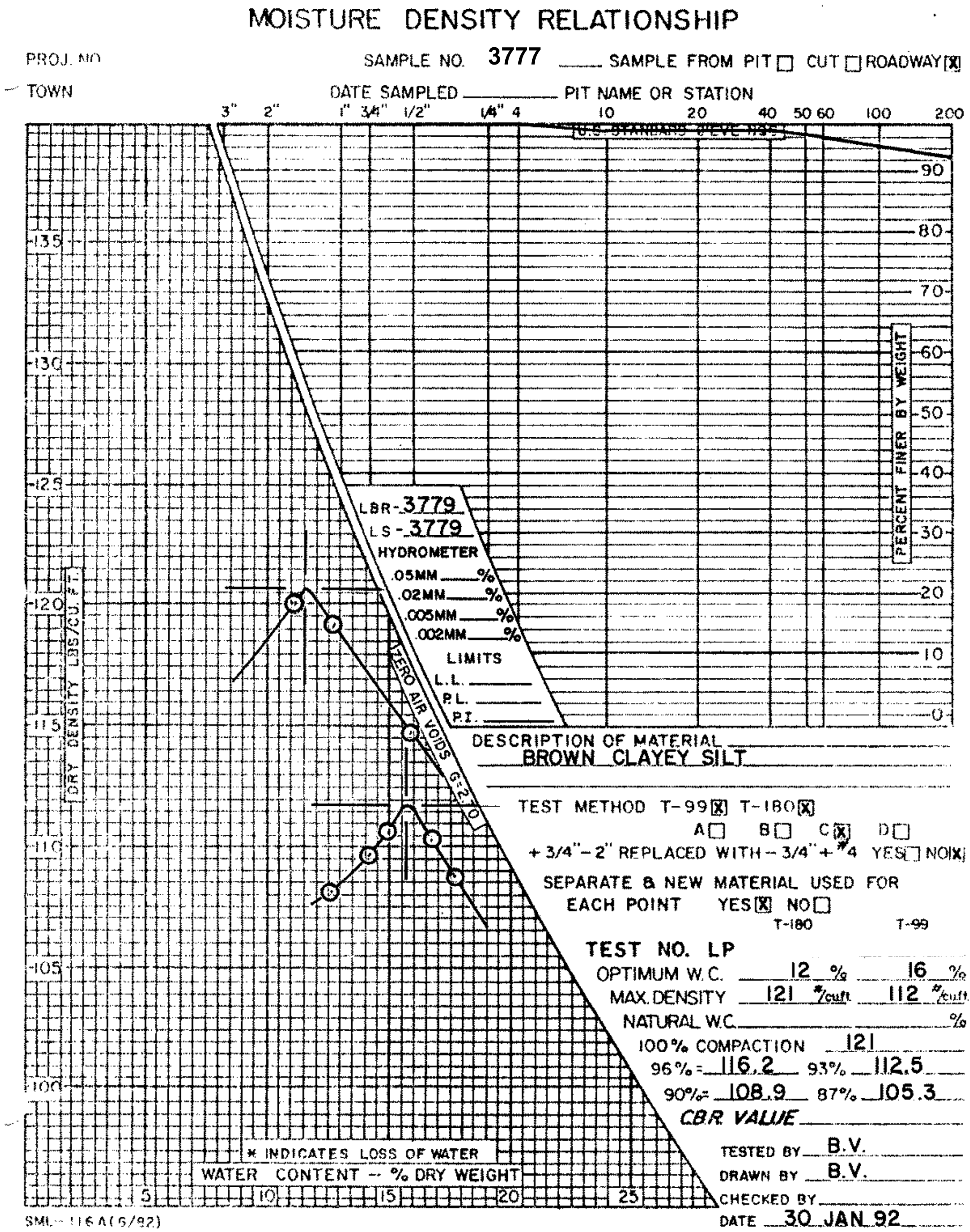

Figure B-20. Compaction curves for clayey silt subgrade soil (sample 90021).

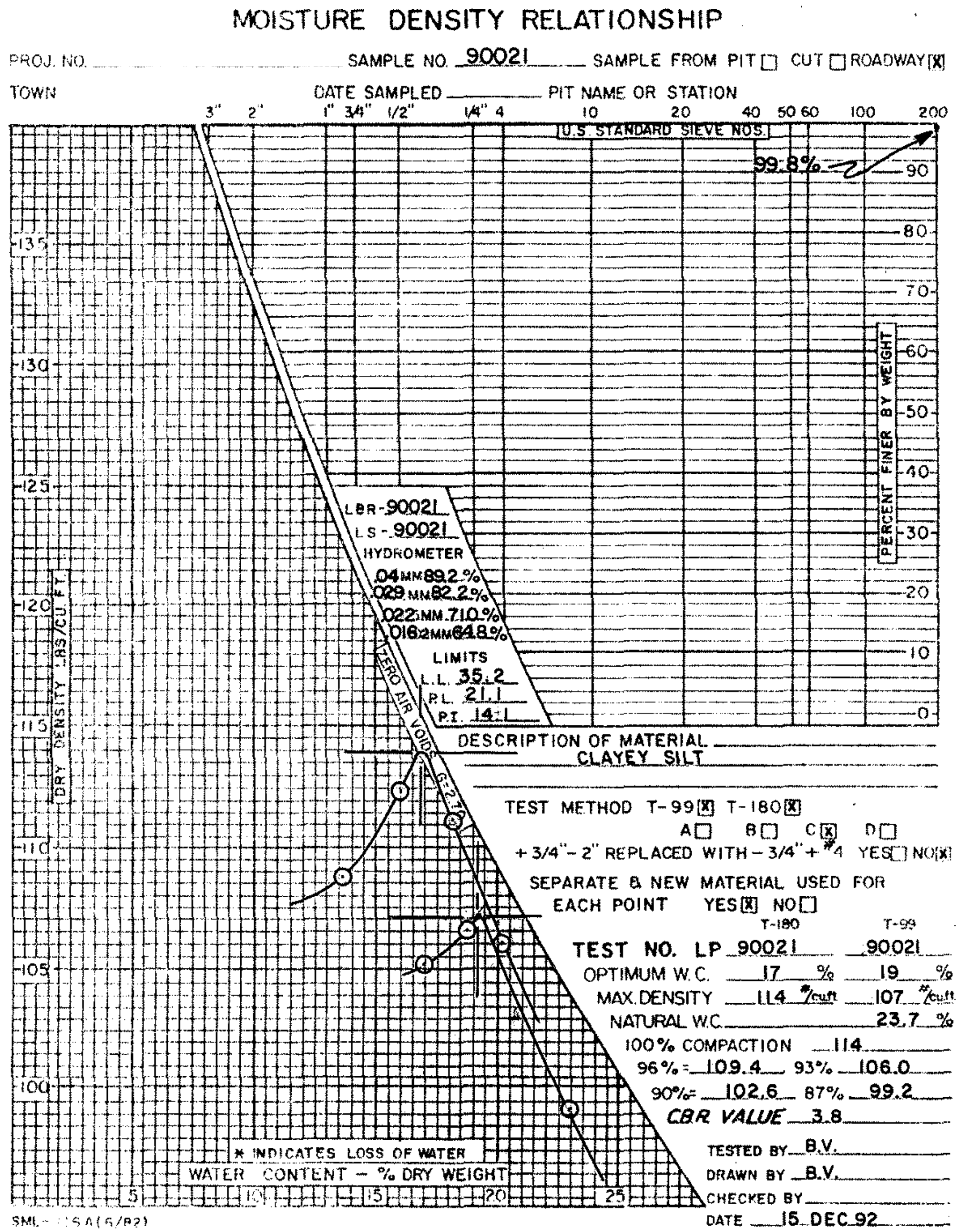

Figure B-21. Compaction curves for clayey silt subgrade soil (sample 90021).

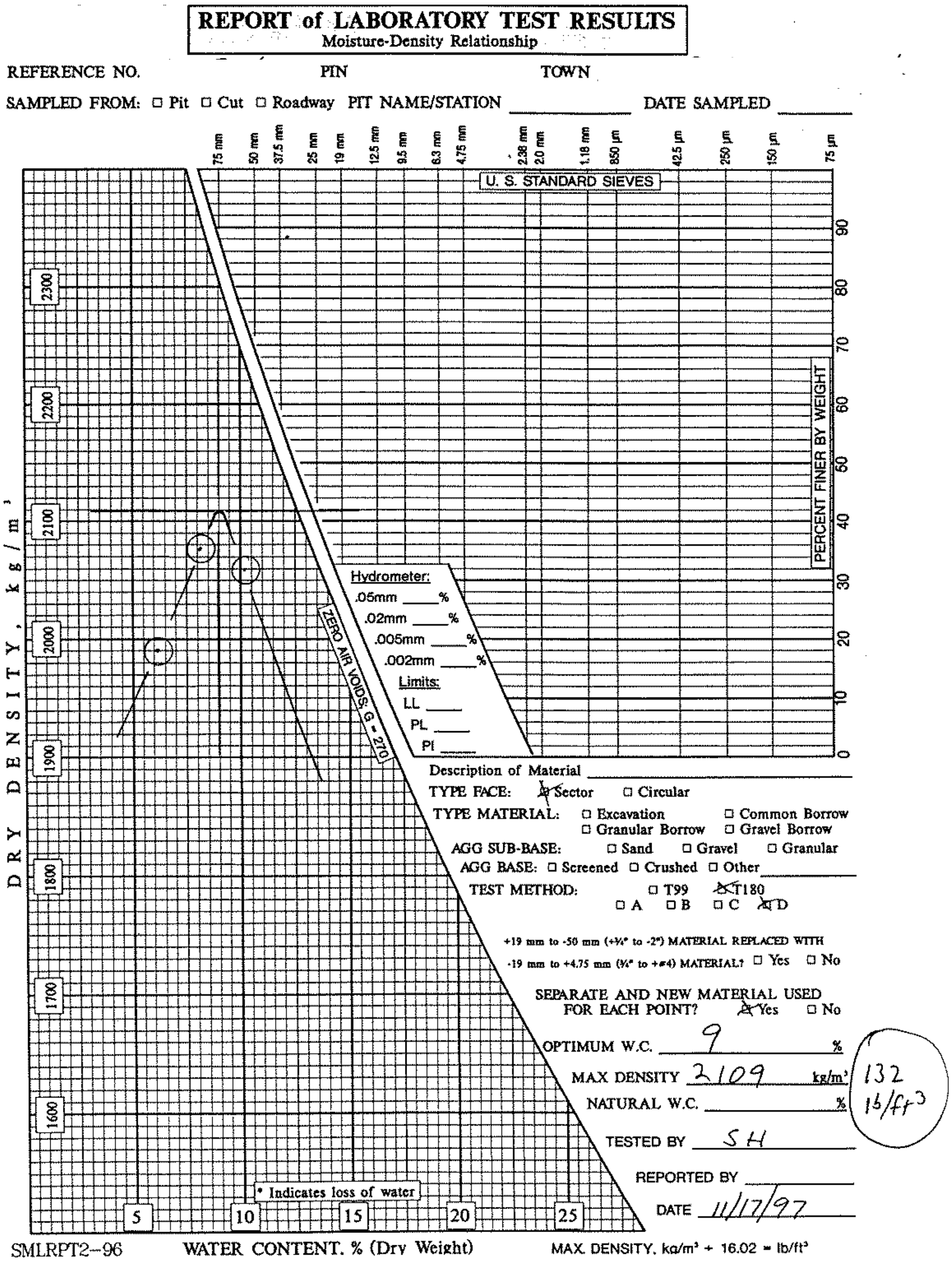

Figure B-22. Compaction curves for granular working platform.

Table B-1. Sample log.

STATION	OFFSET	DEPTH	SAMPLE No.	W.C	L.L.	P.I.	IGN.	pH.	CLASSIFICATION	
									AASHTO	FROST
257+50	5R	8.0 - 9.2	90021	24	35.2	14.1		6.5	A-6	III
264+07	20L	4.0 - 5.0	3777	24	27.5	9.1		6.5	A-4	IV
265+50	30R	0.3 - 0.8	234	24	26.1	5.6			A-4	IV
265+50	30R	0.8 - 1.8	232	92	65.1	29.4	16.6		A-7-5	III
270+00	35L	1.2 - 2.2	233	23	22	6.4			A-4	IV
Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This Classification is followed by the "Frost Susceptibility Rating"from zero (non-frost susceptible) to Class IV (highly frost susceptible). The 'Frost Susceptible Rating"is based upon the Corps of Engineers Classification Systems.										

Table B-2. Laboratory resilient modulus data for clay silt subgrade (sample 3777).

Replicate	Contact Stress (psi)	Confining Pressure (psi)	Cyclic Stress (psi)	θ (psi)	τ_{oct} (psi)	M_R (psi)
1	0.620	8.032	0.946	25.660	0.738	1450
	0.743	6.099	0.839	19.880	0.746	1417
	0.307	4.107	0.806	13.433	0.525	1320
	0.151	2.043	0.757	7.037	0.428	1152
	0.636	8.032	1.431	26.164	0.974	1094
	0.455	5.969	1.390	19.752	0.870	1046
	0.307	3.972	1.316	13.537	0.765	971
	0.184	2.019	1.209	7.450	0.656	888
2	0.603	8.045	0.880	25.618	0.699	1703
	0.447	6.186	0.880	19.886	0.626	1675
	0.299	4.139	0.831	13.547	0.532	1519
	0.167	2.207	0.773	7.560	0.443	1337
	0.620	8.001	1.464	26.087	0.982	1289
	0.447	6.082	1.431	20.124	0.885	1240
	0.307	4.059	1.324	13.808	0.769	1125
	0.167	2.010	1.225	7.423	0.656	1013
3	0.603	8.088	0.806	25.674	0.664	1351
	0.463	6.030	0.789	19.342	0.591	1341
	0.307	4.036	0.724	13.140	0.486	1177
	0.151	2.028	0.674	6.909	0.389	1046
	0.603	8.025	1.357	26.036	0.924	1025
	0.463	6.083	1.316	20.027	0.839	970
	0.332	4.097	1.250	13.873	0.746	898
	0.200	2.012	1.143	7.381	0.633	808

Table B-3. Laboratory resilient modulus data for granular base material.

Replicate	Contact Stress (psi)	Confining Pressure (psi)	Cyclic Stress (psi)	θ (psi)	τ_{oct} (psi)	M_R (psi)
1	0.60	3.07	0.72	10.55	0.62	17623
	1.17	5.96	2.51	21.55	1.74	19768
	2.04	9.98	4.89	36.87	3.27	27007
	3.07	15.08	8.07	56.36	5.25	34378
	4.02	20.08	11.35	75.60	7.25	43204
	0.60	3.06	2.17	11.96	1.31	10960
	1.16	6.00	5.54	24.69	3.16	16819
	2.04	9.97	10.54	42.48	5.93	23760
	3.00	14.91	17.48	65.21	9.66	33003
	4.01	20.02	23.99	88.07	13.20	42197
	0.61	3.03	4.84	14.55	2.57	10496
	1.20	6.07	12.30	31.71	6.36	17847
	2.04	10.03	23.44	55.58	12.01	27008
	3.03	14.98	35.69	83.66	18.25	37326
	4.05	20.10	46.70	111.03	23.92	44990
	0.63	2.97	7.56	17.09	3.86	10734
	1.19	6.06	20.15	39.52	10.06	20036
	2.06	10.00	35.35	67.41	17.64	28956
	3.02	15.04	51.98	100.11	25.93	37193
	4.01	19.99	67.65	131.64	33.78	44559
	0.61	2.99	14.79	24.37	7.26	12506
	1.22	5.95	35.32	54.39	17.23	21959
	2.08	10.03	58.14	90.30	28.38	29538
	3.00	15.04	84.55	132.69	41.27	38762
	4.05	20.03	108.67	172.83	53.14	49332
2	0.61	3.02	1.29	10.97	0.90	7386
	1.23	6.03	2.63	21.96	1.82	11375
	2.03	10.04	4.91	37.06	3.27	20089
	3.00	15.02	7.74	55.82	5.07	28547
	4.03	20.06	10.84	75.04	7.01	38724
	0.50	3.06	2.59	12.26	1.46	6802
	1.21	6.06	5.89	25.28	3.35	12349
	2.08	10.01	10.13	42.23	5.75	20448
	3.06	15.04	16.02	64.20	9.00	30563
	4.08	19.93	21.30	85.17	11.96	39487
	0.59	3.03	4.90	14.57	2.59	7355
	1.21	5.85	11.73	30.48	6.10	13987
	2.00	10.00	21.36	53.37	11.01	23784

Replicate	Contact Stress (psi)	Confining Pressure (psi)	Cyclic Stress (psi)	θ (psi)	τ_{oct} (psi)	M_R (psi)
	3.04	15.05	32.28	80.49	16.65	34193
	4.02	20.06	42.26	106.47	21.82	41623
	0.60	3.02	7.81	17.46	3.97	8253
	1.20	6.06	18.79	38.16	9.42	16909
	2.01	10.02	32.66	64.72	16.34	26331
	3.03	14.93	47.85	95.66	23.98	35554
	4.06	20.01	62.46	126.55	31.36	43168
	0.61	2.97	14.35	23.88	7.05	11096
	1.21	6.05	32.38	51.73	15.84	20945
	2.02	9.93	53.30	85.12	26.08	30242
	3.02	15.02	76.57	124.65	37.52	39709
	4.09	19.97	101.06	165.06	49.57	51042

Table B-4. Stress-dependent resilient modulus parameters.

Soil	k_1	k_2	k_3
Clay silt subgrade	170	0.450	-16.388
Granular base	662	1.010	-0.585

Table B-5. Field backcalculated values for subgrade modulus and effective pavement modulus.

Section	M_R (psi)	E_p (psi)
255+00 - 261+50	3000	25714
261+50 - 268+00	3857	27142
268+00 - 269+00	2571	27142

Table B-6. Freezing index and frost penetration estimates (assuming 32 inches of pavement and base).

	Total Frost Penetration		Frost Penetration into Subgrade	
Freezing Index	Nongranular Subgrade	Granular Subgrade	Nongranular Subgrade	Granular Subgrade
Mean 1200	46 in.	65 in.	14 in.	33 in.
Design 1700	53 in.	85 in.	21 in.	15 in.

Table B-7. Design traffic.

Vehicle Class	Est. AADT (2 way)	Est. AADT Percentage	ESAL Factor	Design ESALs
4	90	1.51%	0.4700	21.15
5	297	4.99%	0.3000	44.55
6	361	6.04%	1.3300	240.7
7	111	1.86%	3.3600	186.48
8	118	1.96%	0.8600	50.74
9	127	2.13%	1.0900	69.22
10	79	1.32%	2.8800	113.76
11	0	0.00%	1.0000	0.00
12	0	0.00%	1.0000	0.00
13	0	0.00%	3.7500	0.00
Light Vehicles	4727	80.19%	0.0008	1.89
All Vehicles	5910	100.00%		727.85

APPENDIX C: 1993 AASHTO DESIGN METHOD

C.1 INTRODUCTION

The *AASHTO Guide for Design of Pavement Structures* (AASHTO, 1993) is the primary document used to design new and rehabilitated highway pavements. Approximately 80% of all states use the AASHTO pavement design procedures, with the majority using the 1993 version. All versions of the AASHTO Design Guide are empirical design methods based on field performance data measured at the AASHO Road Test in 1958-60.

Chapter 3 of this manual describes the evolution of the various versions of the AASHTO Design Guide. Geotechnical inputs to the 1993 AASHTO design procedure are detailed in Chapter 5. Chapter 6 provides some design examples using the 1993 AASHTO procedures.

The overall approach of the 1993 AASHTO procedure for both flexible and rigid pavements is to design for a specified serviceability loss at the end of the design life of the pavement. Serviceability is defined in terms of the Present Serviceability Index, PSI, which varies between the limits of 5 (best) and 0 (worst). Serviceability loss at end of design life, ΔPSI, is partitioned between traffic and environmental effects, as follows (see also Figure 3.8):

$$\Delta PSI = \Delta PSI_{TR} + \Delta PSI_{SW} + \Delta PSI_{FH} \tag{C.1}$$

in which ΔPSI_{TR}, ΔPSI_{SW} and ΔPSI_{FH} are the components of serviceability loss attributable to traffic, swelling, and frost heave, respectively. The structural design procedures for swelling and frost heave are the same for both flexible and rigid pavements; these are detailed in Appendix G of the 1993 AASHTO Guide. The structural design procedures for traffic are different for flexible and rigid pavement types. These procedures are summarized below in Sections C.2 and C.3, respectively. For simplicity, only the design procedures for new construction are summarized here. The design procedures for reconstruction are similar, except that characterization of recycled materials may be required. See the 1993 AASHTO Guide for details of additional procedures (*e.g.,* determination of remaining structural life for overlay design) relevant to rehabilitation design.

C.2 FLEXIBLE PAVEMENT STRUCTURAL DESIGN

Design Equation

The empirical expression relating traffic, pavement structure, and pavement performance for flexible pavements is:

$$\log_{10}\left(W_{18}\right) = Z_R S_0 + 9.36\log_{10}\left(SN+1\right) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40+\frac{1094}{\left(SN+1\right)^{5.19}}} + 2.32\log_{10}\left(M_R\right) - 8.07 \tag{C.2}$$

in which:

W_{18}	=	number of 18 kip equivalent single axle loads (ESALs)
Z_R	=	standard normal deviate (function of the design reliability level)
S_0	=	overall standard deviation (function of overall design uncertainty)
ΔPSI	=	allowable serviceability loss at end of design life
M_R	=	subgrade resilient modulus
SN	=	structural number (a measure of required structural capacity)

The first five parameters typically are the inputs to the design equation, and *SN* is the output. Equation (C.2) must be solved implicitly for the structural number *SN* as a function of the input parameters. The structural number *SN* is defined as:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \tag{C.3}$$

in which D_1, D_2, and D_3 are the thicknesses (inches) of the surface, base, and subbase layers, respectively, a_1, a_2, and a_3 are corresponding structural layer coefficients, and m_2 and m_3 are drainage coefficients for the base and subbase layers, respectively. Equation (C.3) can be generalized for additional bound and/or unbound layers. Note that there may be many combinations of layer thicknesses that can provide satisfactory *SN* values; cost and other issues must be considered to determine the optimal final design.

Design Inputs

Analysis Period

Performance period refers to the time that a pavement design is intended to last before it needs rehabilitation. It is equivalent to the time elapsed as a new, reconstructed, or rehabilitated pavement structure deteriorates from its initial serviceability to its terminal serviceability. The term "analysis period" refers to the overall duration that the design strategy must cover. It may be identical to the performance period. However, realistic performance limitations may require planned rehabilitation within the desired analysis period, in which case, the analysis period may encompass multiple performance periods. Analysis period in this context is synonymous with design life in the 1993 AASHTO Guide. AASHTO recommendations for analysis periods for different types of roads are summarized in Table C-1.

Table C-1. Guidelines for length of analysis period (AASHTO, 1993).

Highway conditions	Analysis period (years)
High-volume urban	30 – 50
High-volume rural	20 – 50
Low-volume paved	15 – 25
Low-volume aggregate surface	10 – 20

Traffic

Traffic is one of the most important factors in pavement design, and every effort should be made to collect accurate data specific to each project. Traffic analysis requires the evaluation of initial traffic volume, traffic growth, directional distribution, and traffic type.

The AASHTO Design Guide is based on cumulative 18 kip (80 KN) equivalent single-axle loads (ESALs). Detailed traffic analysis is beyond the scope of this reference manual. However, ESALs may be estimated using the following equation:

$$ESAL = (ADT_0)(T)(T_f)(G)(D)(L)(365)(Y) \tag{C.4}$$

in which:

ADT_0 = average daily traffic at the start of the design period
T = percentage of trucks in the ADT
T_f = truck factor, or the number of 18 kip ESALs per truck

G	=	traffic growth factor
D	=	directional distribution factor
L	=	lane distribution factor
Y	=	design period in years

AASHTO (1993) and standard pavement engineering textbooks (e.g, Huang, 2004) provide details on the determination of all of these parameters and estimation of design ESALs.

Reliability

Design reliability is defined as the probability that a pavement section will perform satisfactorily over the design period. It must account for uncertainties in traffic loading, environmental conditions, and construction materials. The AASHTO design method accounts for these uncertainties by incorporating a reliability level R to provide a factor of safety into the pavement design and thereby increase the probability that the pavement will perform as intended over its design life. The levels of reliability recommended by AASHTO for various classes of roads are summarized in Table C-2.

The reliability level is not included directly in the AASHTO design equations. Rather, it is used to determine the standard normal deviate Z_R. Values of Z_R corresponding to selected levels of reliability are summarized in Table C-3.

The AASHTO design equations also require specification of the overall standard deviation S_0. For flexible pavements, values for S_0 typically range between 0.35 and 0.50, with a value of 0.45 commonly used for design.

Table C-2. Suggested levels of reliability for various functional classifications (AASHTO, 1993).

Functional classification	Recommended level of reliability	
	Urban	Rural
Interstate and other freeways	85 – 99.9	80 – 99.9
Principal arterials	80 – 99	75 – 95
Collectors	80 – 95	75 – 95
Local	50 – 80	50 – 80

Note: Results base on a survey of AASHTO Pavement Design Task Force.

Table C-3. Standard normal deviates for various levels of reliability.

Reliability (%)	Standard normal deviate (Z_R)	Reliability (%)	Standard normal deviate (Z_R)
50	0.000	93	-1.476
60	-0.253	94	-1.555
70	-0.524	95	-1.645
75	-0.674	96	-1.751
80	-0.841	97	-1.881
85	-1.037	98	-2.054
90	-1.282	99	-2.327
91	-1.340	99.9	-3.090
92	-1.405	99.99	-3.750

Serviceability

Serviceability is quantified by the Present Serviceability Index, PSI. Although PSI theoretically ranges between 5 and 0, the actual range for real pavements is between about 4.5 to 1.5.

The initial serviceability index p_o corresponds to road conditions immediately after construction. A typical value of p_o for flexible pavements is 4.2. The terminal serviceability index p_t is defined as the lowest serviceability that will be tolerated before rehabilitation or reconstruction becomes necessary. A terminal serviceability index of 2.5 or higher is recommended for design of major highways. Thus, a typical allowable serviceability loss due to traffic for flexible pavements can be expressed as:

$$\Delta PSI = p_t - p_o = 4.2 - 2.5 = 1.7 \qquad \text{(C.5)}$$

Subgrade Resilient Modulus

Pavement subgrade quality is defined in terms of its resilient modulus M_R. The resilient modulus M_R is a basic material property that can be measured directly in the laboratory, evaluated in-situ from nondestructive tests, or estimated using various empirical relations as detailed in Chapter 5. The 1993 AASHTO Design Guide also incorporates a procedure for considering seasonal fluctuations in M_R to determine a seasonally averaged value for use in design. This procedure is summarized in Section 5.4.3.

Layer Properties

The material properties required for each layer are the structural layer coefficients a_i and, for unbound materials, the drainage coefficients m_i. Methods for evaluating the a_i and m_i values for unbound materials are detailed in Sections 5.4.5 and 5.5.1, respectively. The chart in Figure C-1 can be used to estimate the structural layer coefficient for asphalt concrete in terms of its elastic modulus at 68°F. Values of a_1 between 0.4 and 0.44 are typically used for dense graded asphalt concrete.

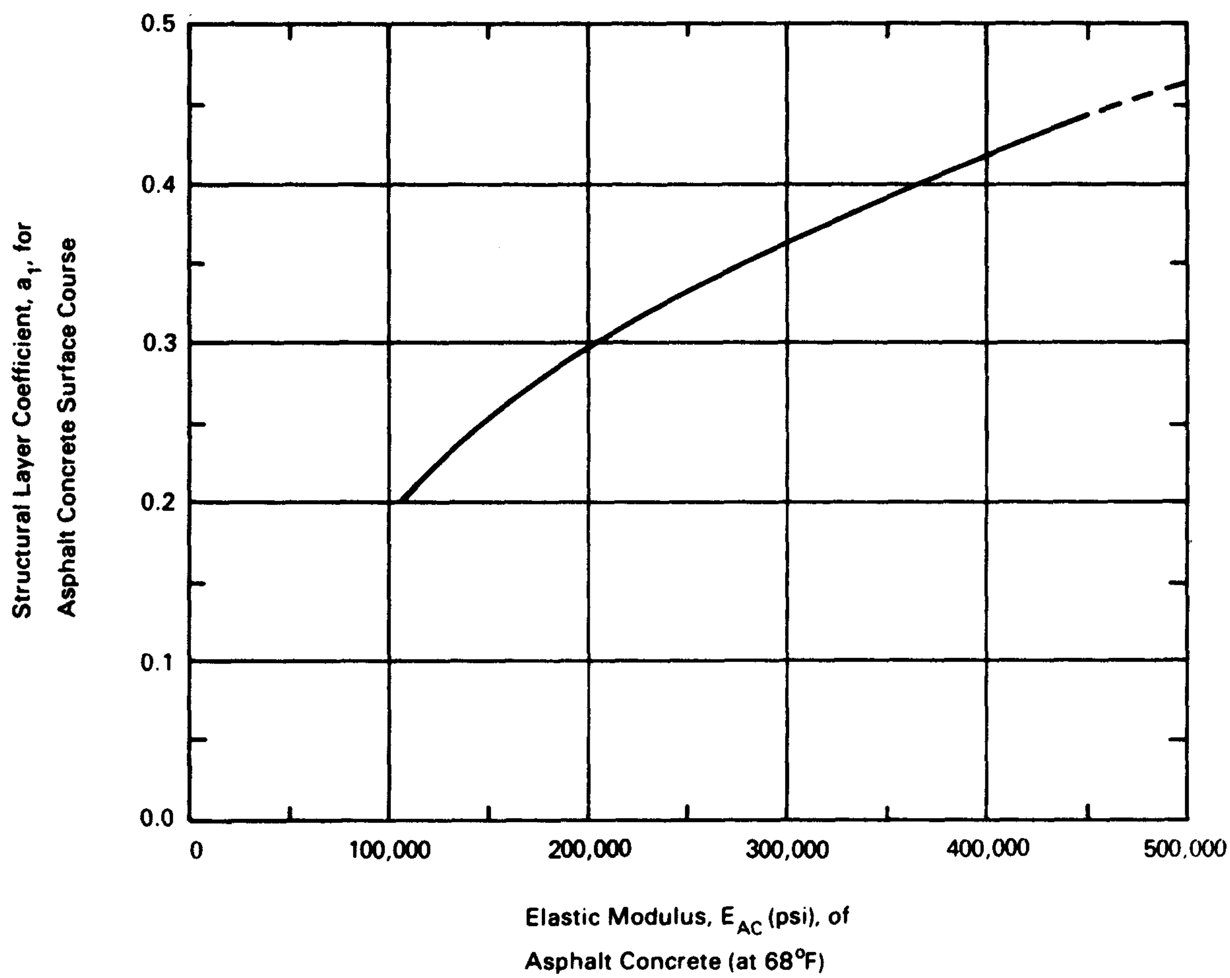

Figure C-1. Chart for estimating structural layer coefficient of dense-graded asphalt concrete based on the elastic (resilient) modulus (AASHTO, 1993).

Procedure

The steps in the 1993 AASHTO flexible pavement design procedure are summarized below in the context of the example baseline scenario presented in Section 6.2.1:

1. Determine the analysis period. For the example design scenario, a 30-year design life is specified.

2. Evaluate the design traffic: W_{18} = 11.6 million ESALs.

3. Determine the design reliability factors: Reliability = 90% (usually set by agency policy), Z_R = -1.282, S_0 = 0.45.

4. Determine the allowable serviceability loss due to traffic: ΔPSI = 1.7 (this may be reduced if frost heave or swelling soils are an issue).

5. Evaluate the seasonally averaged subgrade resilient modulus M_R using the procedures described in Section 5.4.3: M_R = 7,500 psi.

6. Determine the layer properties:
   - Structural layer coefficients a_i for all bound layers (see Section 0 for asphalt concrete, 1993 AASHTO Guide for other stabilized materials) and unbound layers (Section 5.4.5). Recommendations for appropriate a_i values for rehabilitation design are given in Table 5-44 in Section 5.4.5. Values for example design: a_1 = 0.44, a_2 = 0.17.
   - Drainage coefficients m_i for all unbound layers (Section 5.5.1): m_2 = 1.0.

7. Solve Eq. (C.2) for the required overall structural number: SN = 5.07.

8. Determine the design layer thicknesses for the pavement section:
   - Using Eq. (C.2) with M_R set equal to the granular base resilient modulus E_{BS} = 40,000 psi (from the correlation in Eq. 5.16), solve for the required structural number for the asphalt concrete surface layer: SN_1 = 2.62.
   - Convert SN_1 to the required thickness of asphalt: $D_1 = \dfrac{SN_1}{a_1} = 5.95 \rightarrow 6$ inches.[1]

[1]After rounding to the nearest half-inch, per the recommendations in the 1993 AASHTO Design Guide. Unbound layer thicknesses are rounded to the nearest inch.

- Assign the remaining required structural number to the granular base layer: $SN_2 = SN - D_1 a_1 = 2.43$.
- Convert SN_2 to the required thickness of granular base: $D_2 = \dfrac{SN_2}{m_2 a_2} = 14.3 \rightarrow 14$ inches.[1]

C.3 RIGID PAVEMENT STRUCTURAL DESIGN

Design Equation

The empirical expression relating traffic, pavement structure, and pavement performance for rigid pavements is:

$$\log_{10}\left(W_{18}\right) = Z_R S_o + 7.35\log_{10}\left(D+1\right) - 0.06 + \frac{\log_{10}\left[\dfrac{\Delta PSI}{4.5-1.5}\right]}{1+\dfrac{1.64\text{x}10^7}{\left(D+1\right)^{8.46}}} + \left(4.22 - 0.32 p_t\right)\log_{10}\left[\frac{S_c C_d\left(D^{0.75}-1.132\right)}{215.63J\left[D^{0.75}-\dfrac{18.42}{\left(E_c/k\right)^{0.25}}\right]}\right] \tag{C.6}$$

in which:

W_{18}	=	number of 18 kip equivalent single axle loads (ESALs)
Z_R	=	standard normal deviate (function of the design reliability level)
S_0	=	overall standard deviation (function of overall design uncertainty)
ΔPSI	=	allowable serviceability loss at end of design life
p_t	=	terminal serviceability
k	=	modulus of subgrade reaction (pci)
S_c	=	PCC modulus of rupture (psi)
E_c	=	PCC modulus of elasticity (psi)
J	=	an empirical joint load transfer coefficient
C_d	=	an empirical drainage coefficient
D	=	required PCC slab thickness (inches)

The first ten parameters typically are the inputs to the design equation, and D is the output. Equation (C.6) must be solved implicitly for the slab thickness D as a function of the input parameters.

The design of JRCP and CRCP pavements also requires design of the steel reinforcement. Reinforcement design is beyond the scope of this manual; refer to the 1993 AASHTO Guide for details on this.

Design Inputs

Analysis Period
Same as for flexible pavements; see Section 0.

Traffic
Same as for flexible pavements; see Section 0. Note that the truck factor T_f will not in general be the same for rigid and flexible pavements. Refer to the 1993 AASHTO Design Guide or standard pavement engineering textbooks like Huang (2004) for determination of the truck factor.

Reliability
Similar to flexible pavements; see Section 0. For rigid pavements, values for S_0 typically range between 0.3 and 0.45, with a value of 0.35 commonly used for design.

Serviceability
Similar to flexible pavements; see Section 0. A typical value of p_o for rigid pavements is 4.4. As for flexible pavements, a terminal serviceability index of 2.5 or higher is recommended for design of major highways. Thus, a typical allowable serviceability loss due to traffic for rigid pavements can be expressed as:

$$\Delta PSI = p_t - p_o = 4.4 - 2.5 = 1.9 \tag{C.7}$$

Modulus of Subgrade Reaction
The design modulus of subgrade reaction k is a computed quantity that is a function of the following properties:

- Subgrade resilient modulus M_R
- Thickness of granular subbase D_{SB}
- Resilient modulus of granular subbase E_{SB}
- Depth to bedrock D_{SG} (if shallower than 10 feet)

- Loss of Service *LS* (an index of the erodibility of the granular subbase)

See Section 5.4.6 for the procedure for determining the design value for the modulus of subgrade reaction *k*.

Other Layer Properties

Other layer properties include the modulus of rupture S_c and elastic modulus E_c for the Portland cement concrete slabs, an empirical joint load transfer coefficient *J*, and the subbase drainage coefficient C_d. The PCC parameters S_c and E_c are standard material properties; mean values should be used for the pavement design inputs. The joint load transfer coefficient *J* is a function of the shoulder type and the load transfer condition between the pavement slab and shoulders; recommended values are summarized in Table C-4. See Section 5.5.1 for determination of the drainage coefficient C_d.

Table C-4. Recommended load transfer coefficients for various pavement types and design conditions (AASHTO, 1993).

	No Shoulders		Asphalt Shoulders		Tied PCC Shoulders	
	With Load Transfer Devices	Without Load Transfer Devices	With Load Transfer Devices	Without Load Transfer Devices	With Load Transfer Devices	Without Load Transfer Devices
JPCP/ JRCP	3.2	3.8 – 4.4	3.2	3.8 – 4.4	2.5 – 3.1	3.6 – 4.2
CRCP	2.9	N.A.	2.9 - 3.2	N.A.	2.3 – 2.9	N.A.

Procedure

The steps in the 1993 AASHTO rigid pavement design procedure are summarized below in the context of the example baseline scenario presented in Section 6.2.1:

1. Determine the analysis period. For the example design scenario, a 30-year design life is specified.

2. Evaluate the design traffic: W_{18} = 18.9 million ESALs

.

3. Determine the design reliability factors: Reliability = 90% (usually set by agency policy), Z_R = -1.282, S_0 = 0.45.

4. Determine the terminal serviceability and allowable serviceability loss due to traffic: $p_t = 2.5$, $\Delta PSI = 1.9$ (this may be reduced if frost heave or swelling soils are an issue).

5. Evaluate the effective modulus of subgrade reaction k using the procedures described in Section 5.4.6. Specific design inputs to this procedure are the seasonally averaged subgrade resilient modulus $M_R = 7{,}500$ psi, the assumed thickness of the granular subbase D_{SB}, the seasonally averaged subbase resilient modulus $E_{SB} = 40{,}000$ psi, the depth to bedrock D_{SG} (if less than 10 feet—not the case for this example design), and the loss of service coefficient $LS = 2$.

6. Specify the PCC properties: $S_c = 690$ psi, $E_c = 4.4\text{x}10^6$ psi (these would typically be from material specifications; mean values should be used for inputs).

7. Determine the other input parameters: joint load transfer coefficient $J = 3.2$, drainage coefficient $C_d = 1.0$.

8. Solve Eq. (C.6) for the required slab thickness: $D = 10.55 \cong 10.5$ inches.

Note that the thickness assumed for the granular subbase in Step 5 can influence the required slab thickness computed in Step 8. If desired, several design alternatives can be evaluated to arrive at the optimal design.

C.4 SOFTWARE

The empirical design equations for flexible and rigid pavements in Eqs. (C.2) and (C.6) are implicit relationships for the required structural number SN and slab thickness D, respectively. Consequently, an iterative solution algorithm is required. The 1993 AASHTO Design Guide provides nomographs for the graphical evaluation of these equations. They can also be evaluated easily using a spreadsheet, *e.g.,* via the Solver tool in Microsoft Excel. DARWin, a comprehensive software program tied to the 1993 AASHTO Design Guide procedures, is also available through AASHTO. Additional information on DARWin can be found at http://darwin.aashtoware.org/index.htm.

C.5 REFERENCES

AASHTO (1993). *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C.

Huang, Y.H. (2004). *Pavement Analysis and Design (2nd ed.)*, Prentice-Hall, Englewood Cliffs, NJ.

APPENDIX D: NCHRP 1-37A DESIGN METHOD

D.1 INTRODUCTION

The *Guide for the Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures* developed under NCHRP Project 1-37A is the state-of-the-art procedure for the design of flexible and rigid pavement structures. The mechanistic-empirical approach at the heart of the NCHRP 1-37A methodology represents a fundamental paradigm shift for pavement design. In the mechanistic-empirical approach, the response of the pavement – defined in terms of stresses, strains, and other parameters – is analyzed using rigorous theories of mechanics. Critical response quantities – *e.g.,* tensile strains at the bottom of an asphalt or PCC layer – are then related empirically to pavement performance – *e.g.,* fatigue cracking.

Figure D-1 provides a flow chart for the mechanistic-empirical design approach as implemented in the NCHRP 1-37A procedures. The major steps are

1. Define the traffic, environmental, and other general design inputs for the project. In the case of rehabilitation designs, this will also include information on existing pavement conditions (*e.g.,* distress survey, FWD testing).
2. Select a trial pavement section for analysis. For rehabilitation designs, this includes identification of an appropriate rehabilitation strategy.
3. Define the properties for the materials in the various pavement layers.
4. Analyze the pavement response (temperature, moisture, stress, strain) due to traffic loading and environmental influences. The pavement response analysis is performed on a season-by-season basis in order to include variations in traffic loading, environmental conditions, and material behavior over time.
5. Empirically relate critical pavement responses to damage and distress for the pavement distresses of interest. Damage/distress are determined on a season-by-season basis and then accumulated over the design life of the pavement.
6. Adjust the predicted distresses for the specified design reliability.
7. Compare the predicted distresses at the end of design life against design limits. If necessary, adjust the trial pavement section and repeat Steps 3-7 until all predicted distresses are within design limits.

The corresponding major components required to implement this mechanistic-empirical pavement design methodology are

- Inputs—traffic, climate, materials, others.
- Pavement response models—to compute critical responses.

- Performance models or transfer functions—to predict pavement performance over the design life.
- Design reliability and variability—to add a margin of safety for the design.
- Performance criteria—to set objective distress limits against which the pavement performance will be judged.
- Software—to implement the mechanistic-empirical models and calculations in a usable form.

Each of these components will be briefly summarized in the following sections. Readers should refer to the NCHRP 1-37A final reports (NCHRP 1-37A, 2004) for more thorough coverage of each topic. In addition, Chapter 5 provides detailed information on the geotechnical inputs to the NCHRP 1-37A procedure and Chapter 6 gives several example applications.

D.2 INPUTS

D.2.1 Hierarchical Inputs

As described in Chapter 5, the NCHRP 1-37A design methodology incorporates a hierarchical approach for specifying all pavement design inputs. The hierarchical approach is based on the philosophy that the level of engineering effort exerted in determining design inputs should be commensurate with the relative importance, size, and cost of the design project. Three levels are provided for the design inputs in the NCHRP 1-37A procedure:

Level 1 inputs provide the highest level of accuracy and the lowest level of uncertainty. Level 1 design inputs would typically be used for heavily trafficked pavements or whenever there are serious safety or economic consequences of early failure. Level 1 material inputs require field or laboratory evaluation. Subgrade resilient modulus measured from FWD testing in the field or triaxial testing in the laboratory is one example of a Level 1 input.

Level 2 inputs provide an intermediate level of accuracy and are closest to the typical procedures used with the AASHTO Design Guides. This level could be used when resources or testing equipment are not available for Level 1 characterization. Level 2 inputs would typically be derived from a limited testing program or estimated via correlations or experience (possibly from an agency database). Subgrade resilient modulus estimated from correlations with measured CBR values is one example of a Level 2 input.

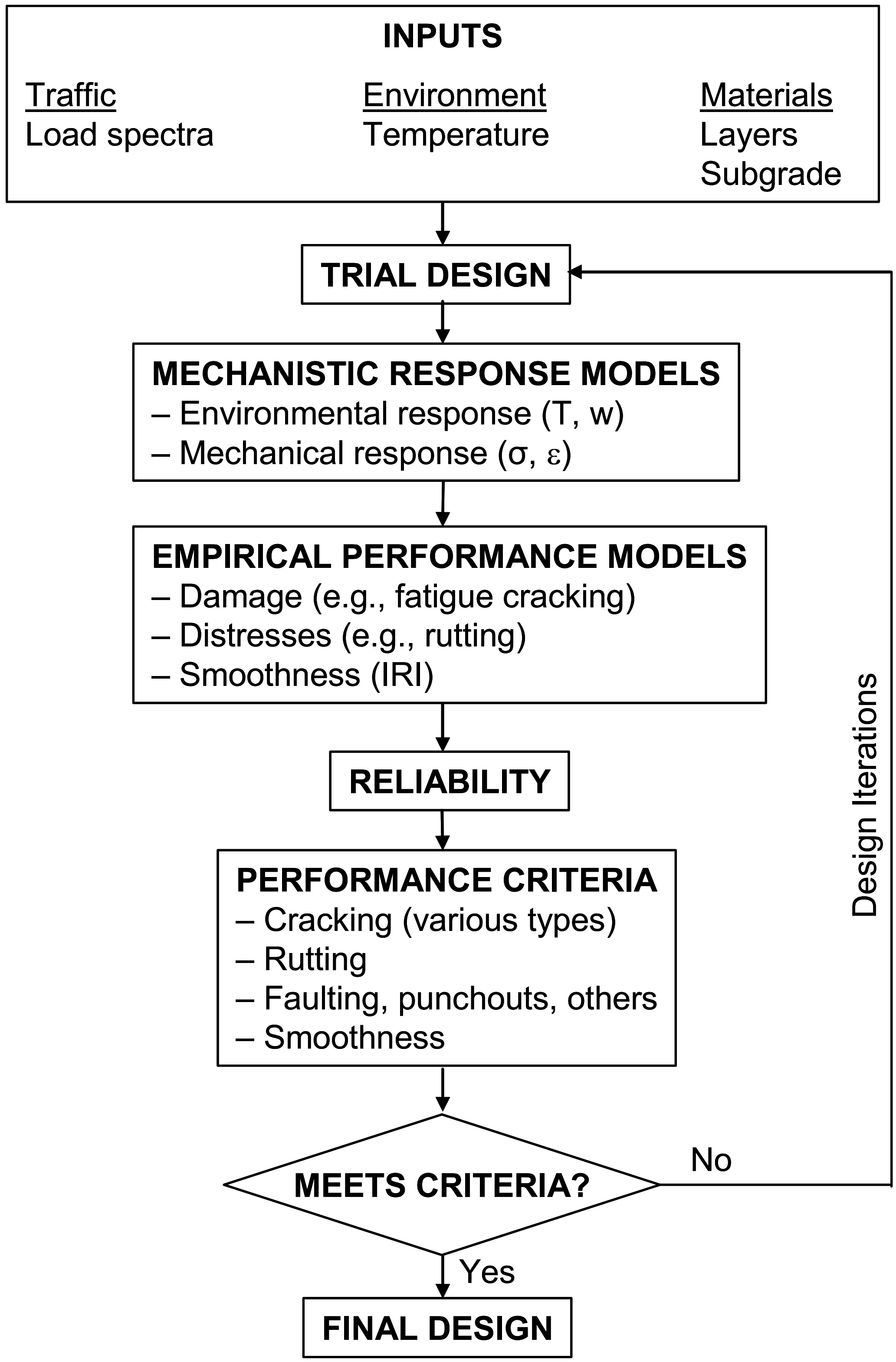

Figure D-1. Flow chart for mechanistic-empirical design methodology.

Level 3 inputs provide the lowest level of accuracy. This level might be used for designs in which there are minimal consequences of early failure (*e.g.,* low volume roads). Level 3 material inputs typically are default values that are based on local agency experience. A default subgrade resilient modulus based on AASHTO soil class is an example of a Level 3 input.

Any given pavement design may incorporate a mix of input data of different levels. For example, measured HMA dynamic modulus values used with default resilient modulus values for the unbound materials in the pavement structure. However, the algorithms used in the design computations are *identical* for all input levels. In other words, the NCHRP 1-37 methodology features levels of input data but not levels of design analysis. The composite input level determines the overall accuracy and reliability of the pavement performance predictions used to judge the acceptability of a trial design.

D.2.2 Traffic

Traffic data are key inputs for the analysis and design of pavement structures. Most existing design procedures, including all of the AASHTO Design Guides, quantify traffic in terms of equivalent single axle loads (ESALs). However, the mechanistic pavement response models in the NCHRP 1-37A methodology require the specification of the magnitudes and frequencies of the actual wheel loads that the pavement is expected to see over its design life. Consequently, traffic must be specified in terms of load spectra rather than ESALs. Load spectra are the frequency distributions of axle load magnitudes by axle configuration (single, tandem, tridem, quad) and season of year (monthly, typically).

State highway agencies typically collect two categories of traffic data. Weigh-in-motion (WIM) data provide information about the number and configuration of axles observed within a set of load groups. Automatic vehicle classification (AVC) data provide information about the number and types of vehicles that use a given roadway as counted over some period of time. **Error! Reference source not found.** summarizes the WIM and AVC data that are required at each of the hierarchical input levels in the NCHRP 1-37A methodology.

The traffic data required in the NCHRP 137A methodology are the same for all pavement types (flexible or rigid) and construction types (new or rehabilitated). Four categories of traffic data are required:

- Traffic volume—base year information
    - Two-way annual average daily truck traffic (AADTT)
    - Number of lanes in the design direction
    - Percent trucks in design direction

- Percent trucks in design lane
- Vehicle (truck) operational speed

- Traffic volume adjustment factors
  - Monthly adjustment
  - Vehicle class distribution (see Table 6-5 for an example)
  - Hourly truck distribution (see Table 6-6 for an example)
  - Traffic growth factors
- Axle load distribution factors by season, vehicle class, and axle type (see Table 6-7 for an example)
- General traffic inputs
- Traffic wander data (mean wheel location and standard deviation of lateral wander; lane width)
- Number axles/trucks (see Table 6-8 for an example)
  - Axle configuration (axle width and spacing; tire spacing and pressure)
  - Wheelbase spacing distribution (rigid pavements only; see Table 6-11 for an example)

The NCHRP 1-37A design software takes all of these traffic inputs and computes the number of applications of each axle load magnitude by axle type (single, tandem, tridem, quad) and month. These axle load spectra are a primary input to the mechanistic pavement structural response models.

Table D-1. Traffic data required for each of the three hierarchical input levels (NHCRP, 2004).

Data Sources		Input Level		
		1	**2**	**3**
Traffic load/volume data	WIM data – site/segment specific	X		
	WIM data – regional default summaries		X	
	WIM data – national default summaries			X
	AVC data – site/segment specific	X		
	AVC data – regional default summaries		X	
	AVC data – national default summaries			X
	Vehicle counts – site/segment specific[1]		X	X
	Traffic forecasting and trip generation models[2]	X	X	X

[1]Level depends on whether regional or national default values are used for the WIM or AVC information.
[2]Level depends on input data and model accuracy/reliability.

D.2.3 Environment

Environmental conditions have a significant effect on the performance of both flexible and rigid pavements. External factors such as precipitation, temperature, freeze-thaw cycles, and depth to water table play key roles in defining the impact of environment on pavement performance. Internal factors such as the susceptibility of the pavement materials to moisture and freeze-thaw damage, drainability of the paving layers, and infiltration potential of the pavement define the extent to which the pavement will react to the external environmental conditions.

Variations in temperature and moisture profiles within the pavement structure and subgrade over the design life of a pavement are simulated in the NCHRP 1-37A design methodology via the Enhanced Integrated Climatic Model (EICM—described more fully in Section D.3.1). The EICM requires a relatively large number of input parameters. As with all other design inputs, EICM input parameters are specified using a hierarchical approach (Levels 1, 2, or 3). Since many of the EICM material property inputs are not commonly measured by most agency and geotechnical laboratories, Level 3 default values will typically be used for most designs. The inputs required by the EICM fall under the following broad categories (see Sections 5.5.2 and 5.6.2 for more detail):

- General information
    - Base/subgrade construction completion date
    - Pavement construction date
    - Traffic opening date
- Weather-related information (Section 5.6.2)
    - Hourly air temperature
    - Hourly precipitation
    - Hourly wind speed
    - Hourly percentage sunshine (used to determine cloud cover)
    - Hourly relative humidity
- Groundwater related information (Section 5.6.2)
    - Groundwater table depth
- Drainage and surface properties (Section 5.5.2)
    - Surface shortwave absorptivity (Section 5.6.2)
    - Infiltration
    - Drainage path length
    - Cross slope
- Pavement materials
    - Asphalt and Portland cement concrete
- Thermal conductivity

- Heat capacity
- Unbound materials (Section 5.5.2)
- Physical properties (specific gravity, maximum dry unit weight, optimum moisture content)
- Soil water characteristic curve
- Hydraulic conductivity (permeability)
- Thermal conductivity
- Heat capacity

The weather-related information required by the EICM can be obtained from weather stations located near the project site. The software accompanying the NCHRP 1-37A Design Guide includes a database from nearly 800 weather stations throughout the United States that can be used to generate the weather-related design inputs.

D.2.4 Material Properties

The material property inputs required for the environmental effects model in the NCHRP 1-37A methodology have already been described in Section D.2.3 (and Sections 5.5.2 and 5.6.2). Additional material property inputs are required for the structural response models used to calculate the stresses and strains in the pavement. As with all other design inputs, the material property inputs can be provided at any of the hierarchical Levels (1, 2, or 3). The material property inputs are most conveniently grouped by material type:

- Asphalt concrete
    - Layer thickness
    - Dynamic modulus (measured value for level 1 or mixture gradation and volumetrics for Level 2 and 3 estimation)
    - Asphalt binder properties (dynamic shear stiffness or viscosity for Levels 1 and 2, binder grade for Level 3)
    - Mixture volumetrics (effective binder content, air voids, unit weight)
    - Poisson's ratio
    - Thermal cracking properties (low temperature tensile strength, creep compliance, thermal expansion coefficient)
- Portland cement concrete
    - Layer thickness
    - Mixture properties (cement and aggregate type, cement content, water/cement ratio, unit weight)
    - Shrinkage characteristics
    - Elastic modulus

    - Poisson's ratio
    - Compressive strength
    - Modulus of rupture
    - Thermal expansion coefficient
- Unbound materials (see Sections 5.3 and 5.4 for more details)
    - Material type
    - Layer thickness
    - Unit weight
    - Coefficient of lateral earth pressure
    - Resilient modulus (see Section 5.4.3 for details on inputs at different hierarchical levels)
    - Poisson's ratio

D.2.5 Other

A variety of other input data are required for the NCHRP 1-37A methodology. Some of these inputs are dependent upon the particular pavement type (flexible vs. rigid) and construction type (new vs. rehabilitation) being considered. A brief summary of these other inputs are as follows:

- General project information
    - Design life
    - Latitude, longitude, and elevation (for accessing weather station database)
- Rigid pavement design features (all rigid pavement types)
    - Permanent curl/warp effective temperature difference
    - Base erodibility index
- JPCP design features
    - Joint spacing, sealant type
    - Dowel bar diameter, spacing
    - Edge support (*e.g.,* tied shoulder, widened slab)
    - PCC-base interface bond condition
- CRCP design features
    - Shoulder type
    - Reinforcement (steel percentage, diameter, depth)
    - Mean crack spacing
- Flexible pavement distress potential (new construction)
    - Block cracking
    - Longitudinal cracks outside wheel paths
- Pre-rehabilitation distresses (overlay over AC surface)
    - Rutting

    - Fatigue cracking within wheel path
    - Longitudinal cracks outside wheel path
    - Patches
    - Potholes
- Pre-rehabilitation distresses (overlay over PCC surface)
    - Percent cracked slabs before, after restoration
    - CRCP punchouts
    - Dynamic modulus of subgrade reaction

Note that no design features are included for jointed reinforced concrete pavements (JRCP). The NCHRP 1-37A methodology does not include a design capability for this pavement type.

D.3 PAVEMENT RESPONSE MODELS

There are two types of pavement response models in the NCHRP 1-37A methodology: (a) an environmental effects model for simulating the time- and depth-dependent temperature and moisture conditions in the pavement structure in response to climatic conditions; and (b) structural response models for determining the stresses and strains at critical locations in the pavement structure in response to traffic loads. The same environmental effects model is used for all pavement types. Different structural response models are employed for rigid vs. flexible pavements because of the fundamental differences in their mechanical behavior.

D.3.1 Environmental Effects

Diurnal and seasonal fluctuations in the moisture and temperature profiles in the pavement structure induced by changes in groundwater table, precipitation/infiltration, freeze-thaw cycles, and other external factors are incorporated in the NCHRP 1-37A design methodology via the Enhanced Integrated Climatic Model (EICM). The EICM is a mechanistic one-dimensional coupled heat and moisture flow analysis that simulates changes in the behavior and characteristics of pavement and subgrade materials induced by environmental factors. The EICM consists of three major components:

- The Climatic-Materials-Structural Model (CMS Model) originally developed at the University of Illinois (Dempsey *et al*., 1985).
- The CRREL Frost Heave and Thaw Settlement Model (CRREL Model) originally developed at the United States Army Cold Regions Research and Engineering Laboratory (CRREL) (Guymon *et al*., 1986).
- The Infiltration and Drainage Model (ID Model) originally developed at Texas A&M University (Lytton *et al.*, 1990).

Each of these components has been enhanced substantially for use in the NCHRP 1-37A design methodology.

For flexible pavements, the EICM evaluates the following environmental effects:

- Seasonal changes in moisture content for all subgrade and unbound materials.
- Changes in resilient modulus, M_R, of all subgrade and unbound materials caused by changes in soil moisture content.
- Changes M_R due to freezing and subsequent thawing and recovery from frozen conditions.
- Temperature distributions in bound asphalt concrete layers (for determining the temperature-dependent asphalt concrete material properties).

For rigid pavements, the following additional environmental effects are simulated by the EICM:

- Temperature profiles in PCC slabs (for thermal curling prediction).
- Mean monthly relative humidity values (for estimating moisture warping PCC slabs).

One of the important outputs from the EICM for both flexible and rigid pavement design is a set of adjustment factors for unbound layer materials that account for the effects of environmental conditions such as moisture content changes, freezing, thawing, and recovery from thawing. This factor, denoted F_{env}, varies with position within the pavement structure and with time throughout the analysis period. The F_{env} factor modifies the resilient modulus at optimum moisture and density conditions M_{Ropt} to obtain the seasonally adjusted resilient modulus M_R as a function of depth and time.

D.3.2 Structural Response

The mechanistic structural response models determine the stresses, strains, and displacements within the pavement system caused by traffic loads and as influenced by environmental conditions. Environmental influences may be direct (*e.g.,* strains due to thermal expansion and/or contraction) or indirect (*e.g.,* changes in material properties due to temperature and/or moisture effects).

Flexible Pavements

Two flexible pavement analysis methods have been implemented in the NCHRP 1-37A computational procedures. For cases in which all materials in the pavement structure can realistically be treated as linearly elastic, multilayer elastic theory (MLET) is used to determine the pavement response. MLET provides an excellent combination of analysis capabilities, theoretical rigor, and computational speed for linear pavement analyses. In cases where the consideration of unbound material nonlinearity is desired (*i.e.,* Level 1 resilient modulus for new construction), a nonlinear finite element (FE) methodology is employed instead for determining the pavement stresses, strains, and displacements.

A major advantage of MLET solutions is very quick computation times. Solutions for multiple wheel loads can be constructed from the fundamental axisymmetric single wheel solutions via superposition automatically by the computer program. The principal disadvantage of MLET is its restriction to linearly elastic material behavior. Real pavement materials, and unbound materials, in particular, often exhibit stress-dependent stiffness. The materials may even reach a failure condition in some locations, such as in tension at the bottom of the unbound base layer in some pavement structures. These nonlinearities vary both vertically through the thickness of the layer and horizontally within the layer. Some attempts have been made in the past to incorporate these material nonlinearity effects into MLET solutions in an approximate way, but the fundamental axisymmetric formulation of MLET makes it impossible to include the spatial variation of stiffness in a realistic manner.

Some of the limitations of MLET solutions are the strengths of FE analysis. In particular, finite element methods can simulate a wide variety of nonlinear material behavior; the underlying finite element formulation is not constrained to linear elasticity, as is the case with MLET. Stress-dependent stiffness and no-tension conditions for unbound materials can all be treated within the finite element framework. However, the FE computational times are substantially longer than for MLET analyses.

The choice of MLET vs. FE structural response model is made automatically by the NCHRP 1-37A software based on the input data from the user (*i.e.,* whether Level 1 new construction inputs are specified for the unbound resilient modulus values). In both cases, the NCHRP 1-37A software automatically pre-processes all of the input data required for the analysis (*e.g.,* automatically generates a finite element mesh), automatically performs the season-by-season analyses over the specified pavement design life, and automatically post-processes all of the analysis output data to compute the season-by-season values of the critical pavement responses for subsequent use in the empirical performance prediction models.

Performance prediction requires identification of the locations in the pavement structure where the critical pavement responses (stress or strain) attain their most extreme values. For multilayer flexible pavement systems, these locations can be difficult to determine. Critical responses are evaluated at several depth locations in the NCHRP 1-37A analyses, depending upon the distress type:

- Fatigue Depth Locations:
  - Surface of the pavement (z = 0),
  - 0.5 inches from the surface (z = 0.5),
  - Bottom of each bound or stabilized layer.
- Rutting Depth Locations:
  - Mid-depth of each layer/sub-layer,
  - Top of the subgrade,
  - Six inches below the top of the subgrade.

The horizontal locations for the extreme values of critical responses are more difficult to determine. The critical location for the simplest case of a single wheel load can usually be determined by inspection – *e.g.,* directly beneath the center of the wheel. The critical location under multiple wheels and/or axles will be a function of the wheel load configuration and the pavement structure. Mixed traffic conditions (single plus multiple wheel/axle vehicle types) further complicate the problem, as the critical location within the pavement structure will not generally be the same for all vehicle types. The NCHRP 1-37A calculations address this problem by evaluating the pavement responses for a set of potential critical locations. Damage/distress magnitudes are calculated from the pavement responses at each location, with the final performance prediction based on the location having the maximum damage/distress at the end of the analysis period.

Rigid Pavements

Finite element analysis has been proven a reliable tool for computing rigid pavement structural responses. However, the season-by-season distress/damage calculations implemented in the NCHRP 1-37A procedure requires hundreds of thousands of calculations to compute incremental damage over a design period of many years. These computations would take days to complete using existing rigid pavement finite element programs. To reduce computer time to a practical level, neural network models have been developed from a large parametric study performed using the ISLAB2000 finite element program (Khazanovich *et al.*, 2000). The neural network models, which, in effect, are similar to regression models, make it possible to accurately compute critical stresses and deflections virtually instantaneously. This in turn makes it possible to perform detailed month-by-month incremental analysis within a practical timeframe (*i.e.,* a few minutes). Appendix QQ in the NCHRP 1-37A final report (NCHRP, 2004) provides a detailed description of the finite element models, parametric study, and neural networks used for the structural analysis of rigid pavements.

A key feature of the rigid pavement structural response model is its treatment of the pavement foundation. The ISLAB2000 analysis program and the neural network models derived from it employ a modified version of the conventional slab-on-Winkler springs pavement structural model (also called a "dense liquid" foundation model). As shown in Figure D-2, the actual multi-layer pavement structure is replaced by an equivalent 2-layer (slab and base) pavement section resting on a Winkler spring foundation having a stiffness characterized by *k*, the modulus of subgrade reaction (see Section 5.4.6). The effective *k* value in the equivalent 2-layer pavement is determined by matching the computed surface deflections for the actual multi-layer pavement section. The surface deflection profile of the actual section is determined using MLET, modeling all layers in the structure. This computed deflection profile is then used to backcalculate the effective *k* value for the equivalent 2-layer section. Thus, the effective *k* value is an internally computed value, not a direct input to the design procedure. The exception to this is rehabilitation design, where *k* determined from FWD testing may be input directly.

The effective *k* value used in the NCHRP 1-37A methodology is interpreted as a dynamic *k* value (e.g., as determined from FWD testing), which should be distinguished from the traditional static *k* values used in previous AASHTO design procedures.

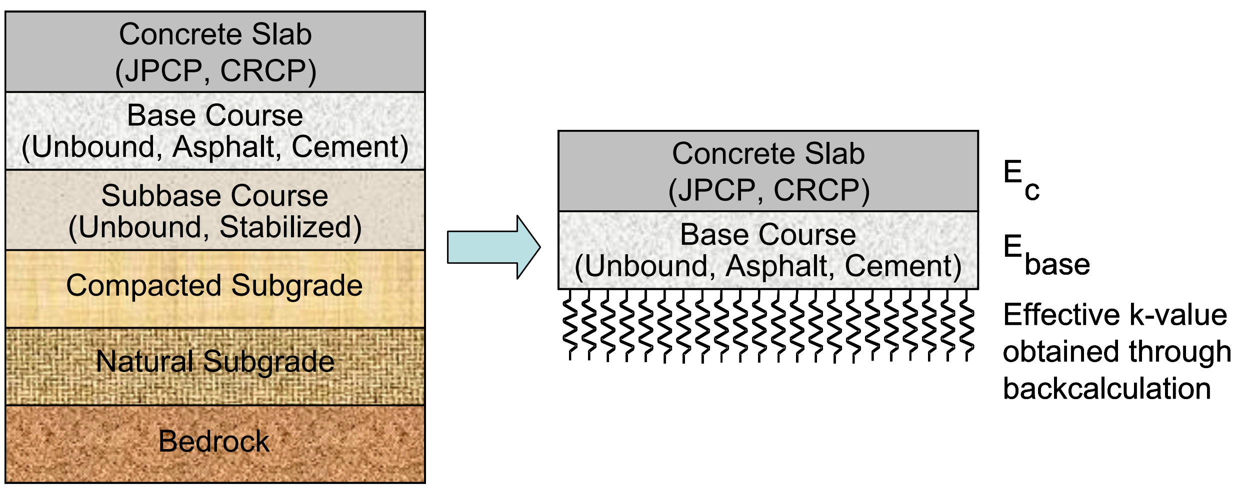

Figure D-2. Structural model for rigid pavement structural response computations.

D.4 PAVEMENT PERFORMANCE MODELS

Pavement performance is evaluated in terms of individual distress modes in the NCHRP 1-37A methodology. A variety of empirical distress models – also sometimes termed "transfer functions" – are incorporated in the NCHRP 1-37A methodology for the major structural distresses in flexible and rigid pavements. Empirical models are also provided for estimating smoothness as a function of the individual structural distresses and other factors.

D.4.1 Damage vs. Distress

Some distresses can be evaluated directly during the season-by-season calculations. For example, the empirical model for rutting in the asphalt layers in flexible pavements is of the form:

$$\frac{\varepsilon_p}{\varepsilon_r} = \beta_{r1} a_1 T^{a_2 \beta_{r2}} N^{a_3 \beta_{r3}} \tag{D.1}$$

in which:

ε_p = accumulated plastic strain after N repetitions of load at the critical location
ε_r = resilient strain at the critical location
N = number of load repetitions
T = temperature
a_i = regression coefficients derived from laboratory repeated load permanent deformation tests
β_{ri} = field calibration coefficients (see Section D.4.4)

Each asphalt layer is divided into sublayers, and Eq. (D.1) is evaluated at the midthickness of each sublayer. The contribution ΔR_{d_i} to total rutting R_d from sublayer *i* having thickness h_i can then be expressed as:

$$\Delta R_{d_i} = \varepsilon_{p_i} . \Delta h_i \tag{D.2}$$

The contributions of all of the sublayers *l* can then be summed to give the total rutting for the asphalt concrete layer:

$$R_d = \sum_{i=1}^{l} \Delta R_{d_i} \tag{D.3}$$

Other distresses cannot be evaluated directly, but must be quantified in terms of computed damage factors. For example, the empirical model for "alligator" fatigue cracking in the asphalt layers in flexible pavements is of the form:

$$N_f = \beta_{f1} k_1 (\varepsilon_t)^{-\beta_{f2} k_2} (E)^{-\beta_{f3} k_3} \tag{D.4}$$

in which:

N_f	=	number of repetitions to fatigue cracking failure
ε_t	=	tensile strain at the critical location
E	=	asphalt concrete stiffness (at appropriate temperature)
k_1, k_2, k_3	=	regression coefficients determined from laboratory fatigue tests
β_{f1}, β_{f2}, β_{f3}	=	field calibration coefficients (see Section D.4.4)

Computation of fatigue damage is based upon Miner's Law:

$$D = \sum_{i=1}^{T} \frac{n_i}{N_{f\,i}} \tag{D.5}$$

in which:

D	=	damage
T	=	total number of seasonal periods
n_i	=	actual traffic for period *i*
N_{fi}	=	traffic repetitions causing fatigue failure under conditions prevailing during period *i*

The damage factor determined using Eq. (D.5) is then related to observed fatigue distress quantities (*e.g.,* area of fatigue cracking within the lane) during the field calibration process (Section D.4.4).

D.4.2 Distress Models

Empirical distress prediction models are provided for the following structural distresses in the NCHRP 1-37A flexible pavement design methodology:

- Permanent deformation (rutting)
  - Within asphalt concrete layers
  - Within unbound base and subbase layers
  - Within the subgrade
- Fatigue cracking
  - Within asphalt concrete layers
- Bottom-up (classical "alligator" cracking)
- Top-down (longitudinal fatigue cracking)
  - Within cement stabilized layers
- Thermal cracking

The empirical structural distress models for rigid pavements include

- Transverse joint faulting (JPCP)
- Transverse fatigue cracking (JPCP)
- Punchouts (CRCP)

Note that reflection cracking for asphalt concrete overlays is not included in the current version of the NCRHP 1-37A methodology. At the time of the NCHRP 1-37A development, it was judged that no suitable empirical reflection cracking models yet existed. It is anticipated that a suitable model will be developed and added to the NCHRP 1-37A procedure in the future.

D.4.3 Smoothness

Pavement smoothness is often used as a composite index of pavement quality. Smoothness (or loss thereof) is influenced by nearly all of the distresses of interest in flexible and rigid pavement systems. Smoothness data is also regularly and routinely collected and stored as part of the pavement management systems at many agencies. Lastly, smoothness is directly related to overall ride quality, the factor of most importance to highway users. Because of these reasons, empirical smoothness prediction models have been incorporated in the NCHRP 1-37A design methodology.

Pavement smoothness in the NCHRP 1-37A models is characterized in terms of the International Roughness Index, or IRI. IRI is predicted as a function of the initial as-constructed IRI, the subsequent development of distresses over time, and other factors such as subgrade type and climatic conditions that may affect smoothness through mechanisms such as shrinkage or swelling of subgrade soils and frost heave. The structural distresses influencing smoothness are predicted directly by the NCHRP 1-37A mechanistic-empirical methodology. However, nonstructural distresses cannot be evaluated using mechanistic-empirical principles, so the NCHRP 1-37A procedure provides the option of specifying the overall potential for these other distresses. Smoothness loss due to soil shrinking/swelling/frost heave and other climatic factors are incorporated into the NCHRP 1-37A IRI models through the use of a "site factor."

The NCHRP 1-37A design method provides IRI prediction models as a function of pavement type (flexible vs. rigid), base type (flexible pavements), and construction type (new vs. rehabilitation). IRI models are provided for the following cases:

- AC (new construction)
  - AC over granular base
  - AC over asphalt-treated base
  - AC over cement-stabilized material
- AC overlay (rehabilitation)
  - AC over flexible pavement
  - AC over rigid pavement
- JPCP (new construction)
- JPCP (rehabilitation)
  - JPCP restoration
  - Bonded PCC over JPCP
  - Unbonded PCC over JPCP
- CRCP (new construction)
- CRCP (rehabilitation)
  - CRCP restoration
  - Bonded PCC over JPCP
  - Unbonded PCC over CRCP (rehabilitation)

Appendix OO in the NCHRP 1-37A final documentation (NCHRP, 2004) provides a detailed description of the development of these models.

D.4.4 Field Calibration

The distress prediction models are key components of the NCHRP 1-37A mechanistic-empirical design and analysis procedure. Calibration of these models against field performance is an essential part of the model development. Calibration refers to the mathematical process by which the models are adjusted to minimize the differences between predicted and observed values of distress.

All performance models in the NCHRP 1-37A design method have been calibrated on a global level to observed field performance at a representative set of pavement test sites around North America. Test sections from the FHWA Long Term Pavement Performance (LTPP) program were used extensively in the calibration process because of the consistency of the monitored data over time and the diversity of test sections throughout North America.

However, there were some serious limitations to the NCHRP 1-37A field calibration. Many of the material property and site feature inputs required for the NCHRP 1-37A analyses were unavailable from the LTPP database. Because of the limited number of pavement test sites with complete input data, the minimal material testing available, the use of calculated properties from correlations (*i.e.,* Level 3 inputs), and the global scope of the calibration effort, the predictions from the calibrated models still have relatively high levels of uncertainty and a limited inference space of application. The recently completed NCHRP Project 9-30 (Von Quintus *et al*., 2003) has formulated a plan for developing an enhanced database for future recalibration of the NCHRP 1-37A and other similar pavement models.

The NCHRP 1-37A software also includes a provision for entering local or regional field calibration factors instead of the national values derived from the LTPP database. This feature permits local agencies to adjust the mechanistic-empirical performance predictions to better reflect their local conditions.

D.5 DESIGN RELIABILITY

A large amount of uncertainty and variability exists in pavement design and construction, as well as in the traffic loads and climatic factors acting over the design life. In the NCHRP 1-37A mechanistic-empirical design, the key outputs of interest are the individual distress quantities. Therefore, variability of the predicted distresses is the focus of design reliability.

The incorporation of reliability in the NCHRP 1-37A procedure is similar in some respects to the way it is treated in the 1993 AASHTO Guide. In the 1993 AASHTO Guide, an overall standard deviation or "uncertainty" is specified for the design inputs (the S_0 value—see Appendix C), a desired reliability level is selected based on agency policy, and the combination of the standard deviation and reliability are then used in essence to add a "margin of safety" to the design traffic W_{18}. The NCHRP 1-37A methodology differs from the 1993 AASHTO procedure in that the standard deviations and reliability levels are set for each individual distress mode predicted in the mechanistic-empirical computations. The default value for the standard deviation of each predicted distress quantity is based on a careful analysis of the differences between the predicted versus actual distresses during the field calibration of the empirical performance models (Section D.4.4). These estimates of error represent the combined effects of input variability, variability in the construction process, and model error.

The desired level of reliability is specified along with the acceptable level of distress at the end of design life (Section D.6) to define the performance requirements for a pavement design in the NCHRP 1-37A procedure. For example, one criterion might be to limit the percent of cracked PCC slabs to 8% at a design reliability of 90%. Then, on average for 100 projects, 90 would be expected to exhibit fewer than 8% slabs cracked at the end of the design life. Different reliability levels may be specified for different distresses in the same design. For example, the designer may choose to specify 95% reliability for slab cracking, but 90% reliability for faulting and IRI. Of course, increasing design reliability will lead to more substantial pavement sections and higher initial costs. The beneficial trade-off is that future maintenance costs should be lower for the higher-reliability design.

D.6 PERFORMANCE CRITERIA

Performance criteria are definitions of the maximum amounts of individual distress or smoothness acceptable to an agency at a given reliability level. Performance criteria are a user input in the NCHRP 1-37A methodology and depend on local design and rehabilitation policies. Default performance criteria built into the current version of the NCHRP 1-37A software are summarized in Table D-2. The designer can select all or some subset of the performance criteria to be evaluated during the design.

D.7 SOFTWARE

The mechanistic-empirical calculations in the NCHRP 1-37A design methodology cannot be performed by hand or simple spreadsheets. A Windows-based program has been developed to implement the NCHRP 1-37A methodology by providing: (1) an interface to input all design variables, (2) computational engines for analysis and performance prediction, and (3) results and outputs from the analyses in formats suitable for use in electronic documents or for making hard copies.

Table D-2. Default performance criteria in NCHRP 1-37A software.

Distress	Unit	Limit[1]
Flexible Pavements		
Top-down (longitudinal) fatigue cracking	feet/mile	1000
Bottom-up (alligator) fatigue cracking	% of wheel path area	25
Thermal fracture	feet/mile	1000
Chemically stabilized layer fatigue cracking	% of wheel path area	25
Total permanent deformation (rutting)	inch	0.75
Permanent deformation (rutting) in asphalt layer	inch	0.25
Terminal IRI[2]	inches/mile	172
Rigid Pavements		
Transverse fatigue cracking (JPCP)	% slabs cracked	15
Mean joint faulting (JPCP)	inch	0.12
Punchouts (CRCP)	number per mile	10
Terminal IRI[2]	inches/mile	172

[1]Default value from software version 0.700 (4/7/2004).
[2]Default initial IRI = 63 inches/mile.

The software presents a series of information and input screens coordinated through a *main program layout* screen, as illustrated in Figure D-3. On this screen, all access points to the information and data input screens are color-coded to guide the designer in providing all data needed to run a design analysis. Green tags indicate screens on which the designer has already entered/reviewed data, yellow tags indicate screens containing default data that have not yet been reviewed/approved by the designer, and red tags indicate screens that have missing required data that must still be entered by the designer before the calculations can be performed. Clicking on any tag brings up the corresponding data input screen; for example, Figure D-4 shows an example data entry screen for subgrade material properties.

The main program layout screen provides access to the following five groupings of information and input screens (*screens* are denoted by the symbol "•", subordinate screen *tabs* by the symbol "♦"):

1. *Project Information*
- General Information
- Site/Project Identification
- Analysis Parameters

2. *Traffic Inputs*

- Traffic Volume Adjustment Factors
- ♦ Monthly Adjustment
- ♦ Vehicle Class Distribution
- ♦ Hourly Distribution
- ♦ Traffic Growth Factors
- Axle Load Distribution Factors
- General Traffic Inputs
- ♦ Number of Axles/Truck
- ♦ Axle Configuration
- ♦ Wheelbase

3. *Climate Inputs*

- Climate

4. *Structure Inputs*

- Structure
- ♦ Drainage and Surface Properties
- ♦ Layers
- ▪ Layer Material Properties
- ♦ Thermal Cracking

5. *Distress Potential*

Note that the *Structure Inputs* listing above is for the case of a new flexible pavement design. The screens will be slightly different for other pavement and construction types, but they all conform to the general organization listed above.

Once all necessary information and input data have been entered into the program, the user clicks the *Run Analysis* button to carry out all the required computations. Separate areas of the main program layout screen provide (1) the status (% complete) of the analyses in progress and (2) links to summary screens for the inputs to the analyses and their results in both tabular and graphical formats. For example, the design analysis of a conventional flexible pavement design might provide output plots of HMA modulus, alligator cracking, thermal cracking, rutting, and IRI versus pavement age. Figure D-5 is an example of the type of output generated by the software. Output can be generated as either Microsoft Excel spreadsheets or as HTML documents for easy import into other engineering applications.

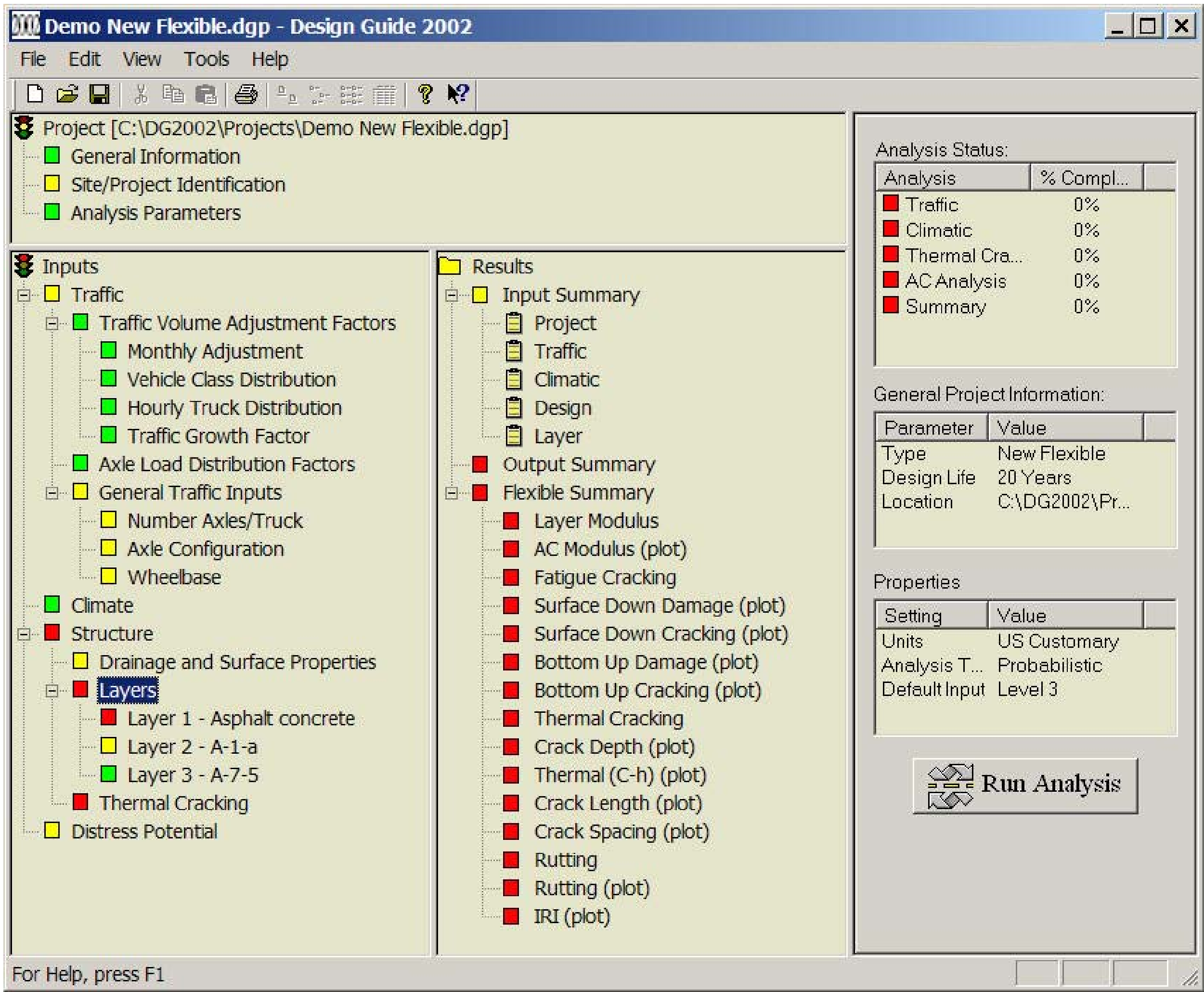

Figure D-3. Main input screen for NCHRP 1-37A software.

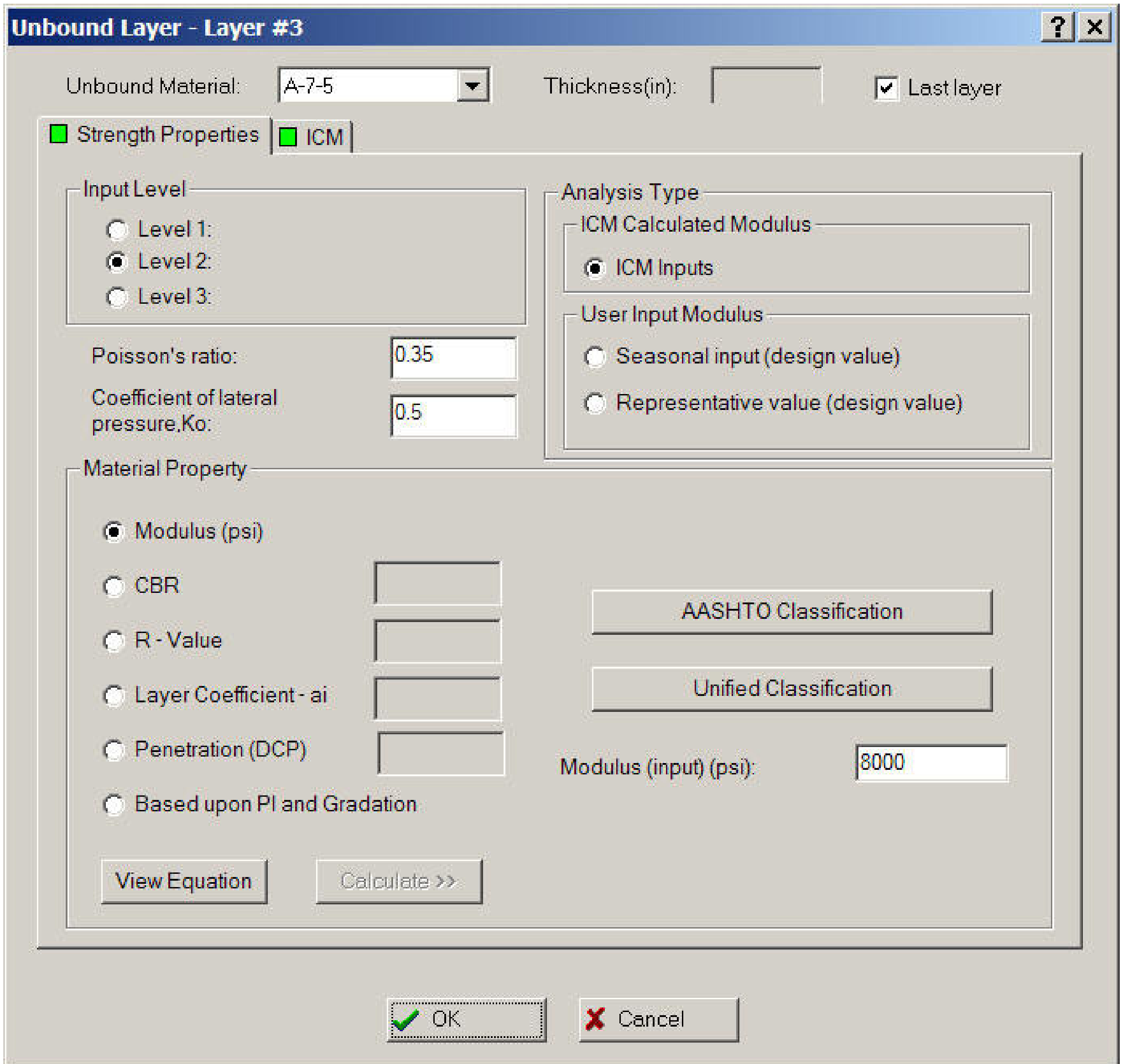

Figure D-4. Typical data entry screen for NCHRP 1-37A software.

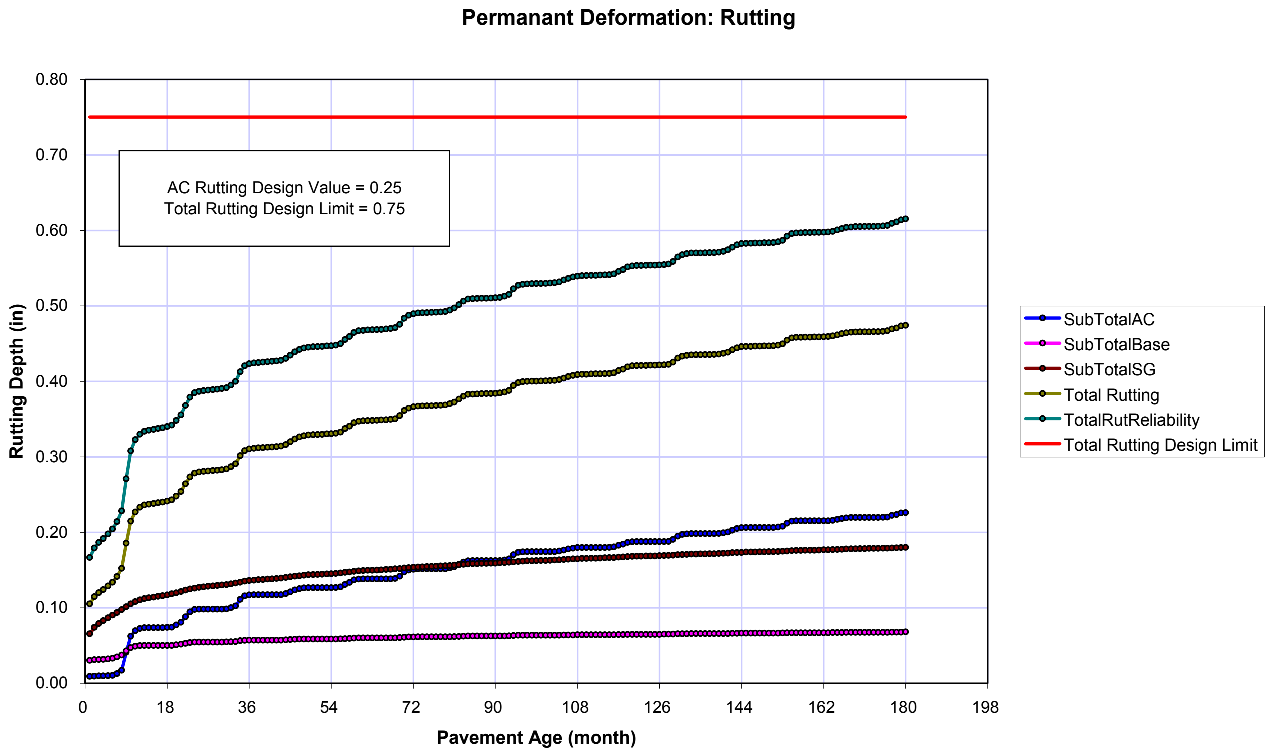

Figure D-5. Typical graphical output from NCHRP 1-37A software.

D.8 REFERENCES

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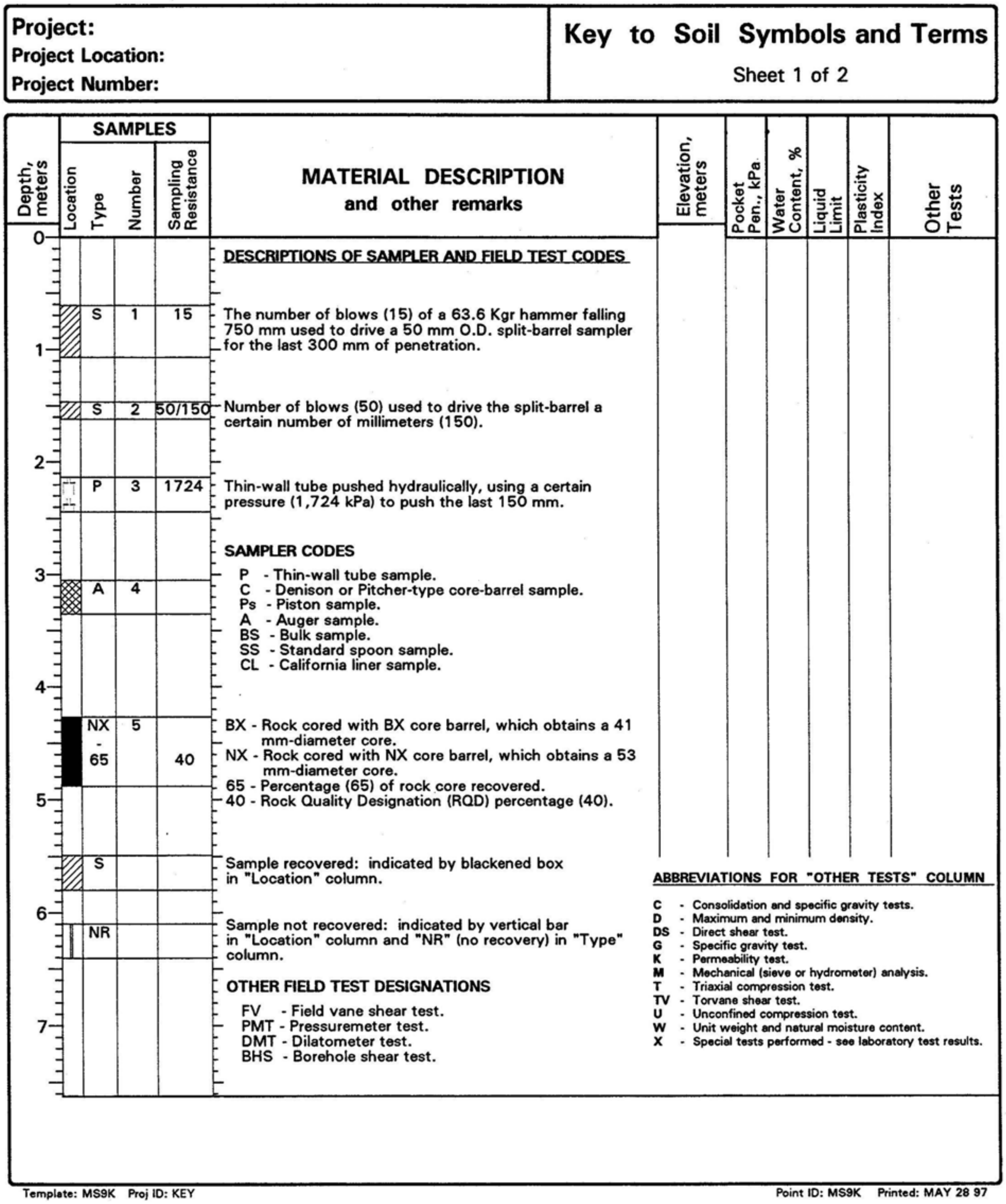
Von Quintus, H.L., Schwartz, C.W., McCuen, R.H. and Andrei D. (2003). "Experimental Plan for Calibration and Validation of Hot Mix Asphalt Performance Models for Mix and Structural Design," *Final Report, NCHRP Project 9-30*, Transportation Research Board, National Research Council, Washington, D.C.

APPENDIX E:
TYPICAL KEY FOR BORING LOG PREPARATION



Project:
Project Location:
Project Number:

Key to Soil Symbols and Terms

Sheet 1 of 2

Depth, meters	SAMPLES: Location	Type	Number	Sampling Resistance	MATERIAL DESCRIPTION and other remarks
0					DESCRIPTIONS OF SAMPLER AND FIELD TEST CODES
1		S	1	15	The number of blows (15) of a 63.6 Kgr hammer falling 750 mm used to drive a 50 mm O.D. split-barrel sampler for the last 300 mm of penetration.
		S	2	50/150	Number of blows (50) used to drive the split-barrel a certain number of millimeters (150).
2		P	3	1724	Thin-wall tube pushed hydraulically, using a certain pressure (1,724 kPa) to push the last 150 mm.
3		A	4		SAMPLER CODES P - Thin-wall tube sample. C - Denison or Pitcher-type core-barrel sample. Ps - Piston sample. A - Auger sample. BS - Bulk sample. SS - Standard spoon sample. CL - California liner sample.
4		NX - 65	5	40	BX - Rock cored with BX core barrel, which obtains a 41 mm-diameter core. NX - Rock cored with NX core barrel, which obtains a 53 mm-diameter core. 65 - Percentage (65) of rock core recovered. 40 - Rock Quality Designation (RQD) percentage (40).
5		S			Sample recovered: indicated by blackened box in "Location" column.
6		NR			Sample not recovered: indicated by vertical bar in "Location" column and "NR" (no recovery) in "Type" column.
7					OTHER FIELD TEST DESIGNATIONS FV - Field vane shear test. PMT - Pressuremeter test. DMT - Dilatometer test. BHS - Borehole shear test.

Elevation, meters | Pocket Pen., kPa | Water Content, % | Liquid Limit | Plasticity Index | Other Tests

ABBREVIATIONS FOR "OTHER TESTS" COLUMN

- C - Consolidation and specific gravity tests.
- D - Maximum and minimum density.
- DS - Direct shear test.
- G - Specific gravity test.
- K - Permeability test.
- M - Mechanical (sieve or hydrometer) analysis.
- T - Triaxial compression test.
- TV - Torvane shear test.
- U - Unconfined compression test.
- W - Unit weight and natural moisture content.
- X - Special tests performed - see laboratory test results.

Template: MS9K Proj ID: KEY Point ID: MS9K Printed: MAY 28 97

Figure E-1. Example key to boring log.

TERMS DESCRIBING CONSISTENCY OR CONDITION

COARSE-GRAINED SOILS (major portion retained on No. 200 sieve): includes (1) clean gravels and sands and (2) silty or clayey gravels and sands. Condition is rated according to relative density as determined by laboratory tests or standard penetration resistance tests.

Descriptive Term	Relative Density	SPT Blow Count
Very loose	0 to 15%	< 4
Loose	15 to 35%	4 to 10
Medium dense	35 to 65%	10 to 30
Dense	65 to 85%	30 to 50
Very dense	85 to 100%	> 50

FINE-GRAINED SOILS (major portion passing on No. 200 sieve): includes (1) inorganic and organic silts and clays, (2) gravelly, sandy, or silty clays, and (3) clayey silts. Consistency is rated according to shearing strength, as indicated by penetrometer readings, SPT blow count, or unconfined compression tests.

Descriptive Term	Unconfined Compressive Strength, kPa	SPT Blow Count
Very soft	< 25	< 2
Soft	25 to 50	2 to 4
Medium stiff	50 to 100	4 to 8
Stiff	100 to 200	8 to 15
Very stiff	200 to 400	15 to 30
Hard	> 400	> 30

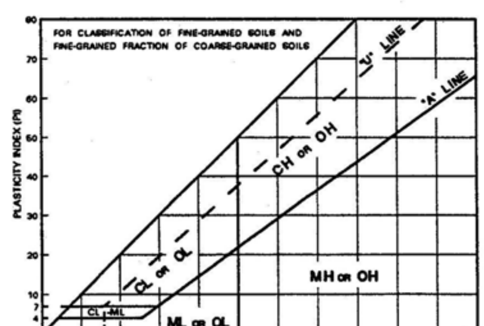
GENERAL NOTES

1. Classifications are based on the Unified Soil Classification System and include consistency, moisture, and color. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate.

2. Surface elevations are based on topographic maps and estimated locations.

3. Descriptions on these boring logs apply only at the specific boring locations and at the time the borings were made. They are not warranted to be representative of subsurface conditions at other locations or times.

Major Divisions			Group Symbols	Typical Names	Laboratory Classification Criteria		
Coarse-Grained Soils (More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve) coarse-grained soils are classified as follows: Less than 5 percent ...... GW, GP, SW, SP; More than 12 percent ...... GM, GC, SM, SC; 5 to 12 percent .... Borderline cases requiring dual symbols **	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM* d / u	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-silt mixtures		Atterberg limits above "A" line or P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly-graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM* d / u	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
Fine-Grained Soils (More than half of material is smaller than No. 200 sieve size)	Silts and Clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Plasticity Chart		
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL	Organic silts and organic silty clays of low plasticity			
	Silts and Clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity, organic silts			
	Highly Organic Soils		Pt	Peat and other highly organic soils			



Plasticity Chart

Particle Size: Material		Sieve size	mm
Silt or clay		< #200	< 0.074
Sand	Fine	#200 to #40	0.074 to 0.42
	Medium	#40 to #10	0.42 to 2.00
	Coarse	#10 to #4	2.00 to 4.76

Particle Size: Material		Sieve	mm
Gravel	Fine	#4 to 3/4 in.	4.76 to 19.1
	Coarse	3/4 in. to 3 in.	19.1 to 76.2
Cobbles		3 in. to 12 in.	76.2 to 304.8
Boulders		12 in. to 36 in.	304.8 to 914.4

* Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg Limits: suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u used when L.L. is greater than 28.

** Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

Figure E-1. Example key for final boring log (continued).

Project: **Project Location:** **Project Number:**	**Key to Rock Core Log** Sheet 1 of 2

Depth, meters	Elevation, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Frac. Freq.	R Q D, %	Fracture Drawing/	Number	Lithology	MATERIAL DESCRIPTION	Packer Tests	Laboratory Tests	Drill Rate, meters/hour	FIELD NOTES
0 1	2	3	4	5	6	7	8	9	10	11 META-ARKOSE, light gray, moderately weathered, moderately strong.	13	14	15	16
2 4		1	1	100	1 0	80		1 M		12 a b c d e f g h 1: 75, J, VN, Fe, Su, Pl, S, VC M: Mechanical Breakage				Slow drilling

1	**Depth:**	Distance (in meters) from the collar of the borehole.
2	**Elevation:**	Elevation (in meters) from the collar of the borehole.
3	**Run No.:**	Number of the individual coring interval, starting at the top of bedrock.
4	**Box No.:**	Number of the core box which contains core from the corresponding run.
5	**Recovery:**	Amount (in percent) of core recovered from the coring interval; calculated as the length of core recovered divided by the length of the run.
6	**Frac. Freq.:**	(Fracture Frequency) The number of naturally occurring fractures in each foot of core; does not include mechanical breaks, which are considered to be induced by drilling.
7	**R Q D:**	(Rock Quality Designation) Amount (in percent) of intact core (pieces of sound core greater than 100 mm in length) in each coring interval; calculated as the sum of the lengths of intact core divided by the length of the core run.
8	**Fracture Drawing:**	Sketch of the naturally occurring fractures and mechanical breaks, showing the angle of the fractures relative to the cross-sectional axis of the core. "NR" indicates no recovery.
9	**Fracture Number:**	Location of each naturally occurring fracture (numbered) and mechanical break (labeled "M"). Naturally occurring fractures are described in Column 11 (keyed by number) using descriptive terms defined on the following page (Items a - h).
10	**Lithology:**	A graphic log presentation using symbols to represent differing rock types.
11	**Description:**	Lithologic description in this order: rock type, color, texture, grain size, foliation, weathering, strength, and other features; descriptive terms are defined on the following page. A detailed descriptive log of overburden materials is not necessarily provided.
12	**Discontinuity Description:**	Abbreviated description of fracture corresponding to number of naturally occurring fracture in Column 9 using terms defined on the following page (Items a - h).
13	**Packer Tests:**	A vertical line depicts the interval over which a packer test is performed.
14	**Laboratory Tests:**	A vertical line depicts the interval over which core has been removed for laboratory testing. Laboratory tests performed are indicated in Column 16.
15	**Drill Rate:**	Rate (in meters per hour) of penetration of drilling. "N/O" indicates rate not observed.
16	**Field Notes:**	Comments on drilling, including water loss, reasons for core loss, and use of drilling mud; also, laboratory tests performed on core.

Template: MASK Proj ID: KEY Print ID: COREKEY Printed: MAY 28 97

Figure E-2. Example key to core boring log.

Project: Project Location: Project Number:	**Key to Rock Core Log** Sheet 2 of 2

Depth, meters	Elevation, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Frac. Freq.	R Q D, %	Fracture Drawing/ Number	Lithology	MATERIAL DESCRIPTION	Packer Tests	Laboratory Tests	Drill Rate, meters/hour	FIELD NOTES

KEY TO DESCRIPTIVE TERMS USED ON CORE LOGS

DISCONTINUITY DESCRIPTORS

a Dip of fracture surface measured relative to horizontal

b Discontinuity Type:
- F - Fault
- J - Joint
- Sh - Shear
- Fo - Foliation
- V - Vein
- B - Bedding

c Discontinuity Width (millimeters):
- W - Wide (12.5-50)
- MW - Moderately Wide (2.5-12.5)
- N - Narrow (1.25-2.5)
- VN - Very Narrow (<1.25)
- T - Tight (0)

d Type of Infilling:
- Cl - Clay
- Ca - Calcite
- Ch - Chlorite
- Fe - Iron Oxide
- Gy - Gypsum/Talc
- H - Healed
- No - None
- Py - Pyrite
- Qz - Quartz
- Sd - Sand

e Amount of Infilling:
- Su - Surface Stain
- Sp - Spotty
- Pa - Partially Filled
- Fi - Filled
- No - None

f Surface Shape of Joint:
- Wa - Wavy
- Pl - Planar
- St - Stepped
- Ir - Irregular

g Roughness of Surface:
- Slk - Slickensided [surface has smooth, glassy finish with visual evidence of striations]
- S - Smooth [surface appears smooth and feels so to the touch]
- SR - Slightly Rough [asperities on the discontinuity surfaces are distinguishable and can be felt]
- R - Rough [some ridges and side-angle steps are evident; asperities are clearly visible, and discontinuity surface feels very abrasive]
- VR - Very Rough [near-vertical steps and ridges occur on the discontinuity surface]

h Discontinuity Spacing (meters):
- EW - Extremely Wide (>20)
- W - Wide (7-20)
- M - Moderate (2.5-7)
- C - Close (0.7-2.5)
- VC - Very Close (<0.7)

ROCK WEATHERING / ALTERATION

Description	Recognition
Residual Soil	Original minerals of rock have been entirely decomposed to secondary minerals, and original rock fabric is not apparent; material can be easily broken by hand
Completely Weathered/Altered	Original minerals of rock have been almost entirely decomposed to secondary minerals, minerals, although original fabric may be intact; material can be granulated by hand
Highly Weathered/Altered	More than half of the rock is decomposed; rock is weakened so that a minimum 50-mm-diameter sample can be broken readily by hand across rock fabric
Moderately Weathered/Altered	Rock is discolored and noticeably weakened, but less than half is decomposed; a minimum 50-mm-diameter sample cannot be broken readily by hand across rock fabric
Slightly Weathered/Altered	Rock is slightly discolored, but not noticeably lower in strength than fresh rock
Fresh	Rock shows no discoloration, loss of strength, or other effect of weathering/alteration

ROCK STRENGTH

Description	Recognition	Approximate Uniaxial Compressive Strength (kPa)
Extremely Weak Rock	Can be indented by thumbnail	250 - 1,000
Very Weak Rock	Can be peeled by pocket knife	1,000 - 5,000
Weak Rock	Can be peeled with difficulty by pocket knife	5,000 - 25,000
Medium Strong Rock	Can be indented 5 mm with sharp end of pick	25,000 - 50,000
Strong Rock	Requires one hammer blow to fracture	50,000 - 100,000
Very Strong Rock	Requires many hammer blows to fracture	100,000 - 250,000
Extremely Strong Rock	Can only be chipped with hammer blows	> 250,000

Template: M4SK Proj ID: KEY — Point ID: COREKEY Printed: MAY 28 97

Figure E-2. Example key to core boring log (continued).

APPENDIX F:

DETERMINATION OF ADMIXTURE CONTENT FOR SUBGRADE STABILIZATION

(Adopted from Joint Departments of the Army and Air Force, USA,
TM 5-822-14/AFMAN 32-8010, *Soil Stabilization for Pavements*,
25 October 1994.)

Lime Content for Lime-Stabilized Soils

To determine the design lime content for a subgrade soil, the following steps are suggested:

1. Determine whether the soil has at least 25% passing the 75-µm sieve and has a plasticity index (PI) of at least 10. The soil screening criteria also limit soluble sulfates to less than 0.3 % by weight in a 10:1 water-to-soil solution.
2. Determine the initial design lime content by mixing varying amounts of lime with the soil in water and measuring the pH levels in 1-hour intervals. Select the lowest lime mixture level for which a pH of 12.4 occurs as the initial design lime content.
3. Using the initial design lime content conduct moisture-density tests to determine the maximum dry density and optimum water content of the soil lime mixture defined by the user agency, *e.g.,* AASHTO T-99, AASHTO T-180, ASTM D 698, or ASTM D 1557. The procedures in ASTM D 3551 will be used to prepare the soil-lime mixture.
4. Prepare specimens at optimum moisture content and specified density requirement (*e.g.,* 90% of AASHTO T-180) using the initial design lime content and at about 2% and 4% lime above that lime content from Step 1. Cure the test specimens in sealed plastic bags for 28 days at 21°C (73°F). (Alternative – cure for 7 days at 40° C (104°F)).
5. Determine the unconfined compressive strength for all cured test specimens (*e.g.,* ASTM 5102). Select as the construction design lime content the minimum percent required to achieve the required compressive strength (*e.g.,* 150 psi). Either prepare a sample at the design lime content and perform resilient modulus test (*e.g.,* AASHTO T 294-94) or estimate from Unconfined compression strength Q_u. A conservative estimate for lime-stabilized soils has been reported to be obtained from (Thompson, 1970):

$$M_R = 0.124\ q_u + 9.98$$

where,

M_R = resilient modulus, ksi,

q_u = unconfined compressive strength, psi, as tested in accordance with

ASTM D 5102, “Standard Test Method for Unconfined Compressive Strength of Compacted Soil-Lime Mixtures”

6. Add 0.5 – 1% additional lime in the lower percentage ranges to compensate for problems associated with non-uniform mixing during construction.

Laboratory testing should always be performed to check whether the stabilization has the desired effect on other engineering properties like plasticity and strength.

Cement Content for Cement-Modified Soils

(1) *Improve plasticity.* The amount of cement required to improve the quality of the soil through modification is determined by the trial-and-error approach. If it is desired to reduce the PI of the soil, successive samples of soil-cement mixtures must be prepared at different treatment levels and the PI of each mixture determined. The Referee Test of ASTM D 423 and ASTM D 424 procedures will be used to determine the PI of the soil-cement mixture. The minimum cement content that yields the desired PI is selected, but since it was determined based upon the minus 40 fraction of the material, this value must be adjusted to find the design cement content based upon total sample weight expressed as:

$$\mathrm{A} = 100BC$$

where,

A = design cement content, percent total weight of soil
B = percent passing No. 40 sieve size, expressed as a decimal
C = percent cement required to obtain the desired PI of minus 40 material, expressed as a decimal

(2) *Improve gradation.* If the objective of modification is to improve the gradation of a granular soil through the addition of fines, then particle-size analysis (ASTM D 422) should be conducted on samples at various treatment levels to determine the minimum acceptable cement content.

(3) *Reduce swell potential.* Small amounts of Portland cements may reduce swell potential of some swelling soils. However, Portland cement generally is not as effective as lime, and may be considered too expensive for this application. The determination of cement content to reduce the swell potential of fine-grained plastic soils can be accomplished by molding several samples at various cement contents and soaking the specimens along with untreated specimens for 4 days. The lowest cement content that eliminates the swell

potential or reduces the swell characteristics to the minimum is the design cement content. Procedures for measuring swell characteristics of soils are found in ASTM D 4546 and MIL-STD-621A, Method 101. The cement content determined to accomplish soil modification should be checked to see whether it provides an unconfined compressive strength great enough to qualify for a reduced thickness design in accordance with criteria established for soil stabilization.

(4) *Condition frost areas.* Cement-modified soil may also be used in frost areas, but in addition to the procedures for mixture design described in (1) and (2) above, cured specimens should be subjected to the 12 freeze-thaw cycles prescribed by ASTM D 560 (but omitting wire-brushing) or other applicable freeze-thaw procedures. This should be followed by determination of frost design soil classification by means of standard laboratory freezing tests. If cement-modified soil is used as subgrade, its frost susceptibility, determined after freeze-thaw cycling, should be used as the basis of the pavement thickness design if the reduced subgrade design method is applied.

Cement Content for Cement-Stabilized Soil

The following procedure is recommended for determining the design cement content for cement-stabilized soils.

Step 1. Determine the classification and gradation of the untreated soil following procedures in ASTM D 422 and D 2487, respectively.

Step 2. Using the soil classification, select an estimated cement content for moisture-density tests from Table F-1.

Table F-1. Cement requirements for various soil types.

Soil Type	Initial Estimated Cement Content percent dry weight
GW, SW	5
GP, GW-GC, GW-GM, SW-SC, SW-SM	6
GC, GM, GP-GC, GP-GM, GM-GC, SC, SM, SP-SC, SP-SM, SM-SC, SP	7
CL. ML, MH	9
CH	11

Step 3. Using the estimated cement content, conduct moisture-density tests to determine the maximum dry density and optimum water content of the soil-cement mixture. The procedure contained in ASTM D 558 will be used to prepare the soil-cement mixture and to make the necessary calculations; however, the procedures outlined in AASHTO T180 or ASTM D 1557 will be used to conduct the moisture density test.

Step 4. Prepare triplicate samples of the soil-cement mixture for unconfined compression and durability tests at the cement content selected in Step 2 and at cement contents 2% above and 2% below that determined in Step 2. The samples should be prepared at the density and water content to be expected in field construction. For example, if the design density is 95% of the laboratory maximum density, the samples should also be prepared at 95%. The samples should be prepared in accordance with ASTM D 1632, except that when more than 35% of the material is retained on the 4.75 mm (# 4) sieve, a 100-mm (4-in.) diameter by 200-mm-high (8-in.) mold should be used to prepare the specimens. Cure the specimens for 7 days in a humid room before testing. Test three specimens using the unconfined compression test in accordance with ASTM D 1633, and subject three specimens to durability tests, either wet-dry (ASTM D 559) or freeze-thaw (ASTM D 560) tests, as appropriate. The frost susceptibility of the treated material should also be determined, as indicated in appropriate pavement design manuals.

Step 5. Compare the results of the unconfined compressive strength and durability tests with the requirements. The lowest cement content that meets the required unconfined compressive strength requirement and demonstrates the required durability is the design cement content. If the mixture should meet the durability requirements, but not the strength requirements, the mixture is considered to be a modified soil. If the results of the specimens tested do not meet both the strength and durability requirements, then a higher cement content may be selected and Steps 1 through 4 above repeated.

Selection of Lime-Flyash Content for LF and the Determination of the Ratio of Lime to Fly LCF Mixtures.

(1) *Step 1.* The first step is to determine the optimum fines content that will give the maximum density. This is done by conducting a series of moisture-density tests using different percentages of flyash and determining the mix level that yields maximum density. The initial flyash content should be about 10%, based on dry weight of the mix. It is recommended that material larger than 19 mm (¾ in.) be removed and the test conducted on the minus 19 mm (¾ in.) fraction. Tests are run at increasing increments of flyash, *e.g.,* 2%,

up to a total of about 20%. Moisture density tests should be conducted following procedures indicated in AASHTO T99, AASHT T180, and ASTM D 1557. The design flyash content is then selected at 2% above that yielding maximum density. An alternate method is to estimate optimum water content and conduct single point compaction tests at flyash contents of 10 – 20%, make a plot of dry density versus flyash content, and determine the flyash content that yields maximum density. The design flyash content is 2% above this value. A moisture density test is then conducted to determine the optimum water content and maximum dry density.

(2) *Step 2.* Determine the ratio of lime to flyash that will yield highest strength and durability. Using the design flyash content and the optimum water content determined in Step 1, prepare triplicate specimens at three different lime-flyash ratios, following the selected density procedure. Use LF ratios of 1:3, 1:4, and 1:5. If desired, about 1% of Portland cement may be added at this time.

(3) *Step 3.* Test three specimens using the unconfined compression test. If frost design is a consideration, subject three specimens to 12 cycles of freeze-thaw durability tests (ASTM D 560), except wire brushing is omitted. The frost susceptibility of the treated material shall also be determined as indicated in the appropriate design manual.

(4) *Step 4.* Compare the results of the unconfined compressive strength and durability tests with the requirements. The lowest LF ratio content, *i.e.,* ratio with the lowest lime content that meets the required unconfined compressive strength requirement and demonstrates the required durability, is the design LF content. The treated material must also meet frost susceptibility requirements, as indicated in the appropriate pavement design manuals. If the mixture should meet the durability requirements, but not the strength requirements, it is considered to be a modified soil. If the results of the specimens tested do not meet both the strength and durability requirements, a different LF content may be selected, or additional Portland cement used and Steps 2 through 4 repeated.

Selection of Cement Content for LCF Mixtures.

Portland cement may also be used in combination with LF for improved strength and durability. If it is desired to incorporate cement into the mixture, the same procedures indicated for LF design should be followed except that, beginning at Step 2, the cement shall be included. Generally, about 1 – 2% cement is used. Cement may be used in place of or in addition to lime; however, the total tines content should be maintained. Strength and durability tests must be conducted on samples at various LCF ratios to determine the combination that gives best results.

Selection of Asphalt Content for Bituminous-Stabilized Soil

Guidance for the design of bituminous-stabilized base and subbase courses is contained in U.S. Army TM 5-822-8/AFM 88-6, Chap. 9. For subgrade stabilization, the following equation may be used for estimating the preliminary quantity of cutback asphalt to be selected:

$$p = \frac{0.02(a) + 0.07(b) + 0.15(c)\ 0.20(d)}{(100 - s)}\ x\ 100$$

where

p = percent cutback asphalt by weight of dry aggregate
a = percent of mineral aggregate retained on No. 50 sieve
b = percent of mineral aggregate passing No. 50 sieve and retained on No. 100 sieve
c = percent of mineral aggregate passing No. 100 sieve and retained on No. 200 sieve
d = percent of mineral aggregate passing No. 200 sieve
s = percent solvent

The preliminary quantity of emulsified asphalt to be used in stabilizing subgrades can be determined from Table F-2. The final design content of cutback or emulsified asphalt should be selected based upon the results of the Marshal Stability test procedure (AASHTO T 245, ASTM D 5581, MIL-STD 620A). The minimum Marshall Stability recommended for subgrades is 2.2 kN (500 lb). If a soil does not show increased stability when reasonable amounts of bituminous materials are added, the gradation of the soil should be modified, or another type of bituminous material should be used. Poorly graded materials may be improved by the addition of suitable tines containing considerable material passing the 75 μm (No. 200) sieve. The amount of bitumen required for a given soil increases with an increase in percentage of the liner sizes.

Table F-2. Emulsified asphalt requirements.

Percent Passing 75-µm (No. 200) Sieve	Pounds of Emulsified Asphalt per 100 pounds of Dry Aggregate at Percent Passing No. 10 Sieve					
	<50	60	70	80	90	100
0	6.0	6.3	6.5	6.7	7.0	7.2
2	6.3	6.5	6.7	7.0	7.2	7.5
4	6.5	6.7	7.0	7.2	7.5	7.7
6	6.7	7.0	7.2	7.5	7.7	7.9
8	7.0	7.2	7.5	7.7	7.9	8.2
10	7.2	7.5	7.7	7.9	8.2	8.4
12	7.5	7.7	7.9	8.2	8.4	8.6
14	7.2	7.5	7.7	7.9	8.2	8.4
16	7.0	7.2	7.5	7.7	7.9	8.2
18	6.7	7.0	7.2	7.5	7.7	7.9
20	6.5	6.7	7.0	7.2	7.5	7.7
22	6.3	6.5	6.7	7.0	7.2	7.5
24	6.0	6.3	6.5	6.7	7.0	7.2
25	6.2	6.4	6.6	6.9	7.1	7.3

1 lb = 0.454 kg

Table F-3. Common guidelines for stabilized drainable base mixes
(after FHWA Demonstration Project 87: Drainable Pavement Systems, FHWA-SA-92-008).

Stabilization Method	Item	Requirement
Asphalt-Stabilized	Gradation of material	AASHTO No. 67 stone, preheat at 135^{o} – 160^{o} C (275^{o} – 320^{o} F).
	Amount of asphalt	2 – 2.5% by weight, using a harder asphalt like AC 40 or AR 8000.
	Temperature of mix	Lay at 90^{o} – 120^{o} C (195^{o} – 250^{o} F) and seal with one pass of a 7.2 – 10.9 metric ton (8 – 12 ton) smooth wheel roller. Start compaction rolling after the temperature reaches 65^{o} C (150^{o} F), but before it drops to 38^{o} C (100^{o} F).
Cement-Stabilized	Gradation of material	AASHTO No. 67 stone.
	Amount of cement	Use 110 – 150 kg of cement per cubic meter (185 – 250 lbs/yd^3). (135 – 150 kg/m^3 (230 – 250 lbs/yd^3) for high traffic loads). (A minimum compressive strength of 4.1 MPa (600 psi) is typically suggested in cold regions to resist frost deterioration.)
	Curing requirements	Not clearly understood, and may require local testing (consider a 150 m-long (500 ft) test strip). It is suggested that the mix be covered with plastic for five days after laydown, or that light misting be done, starting the second day after laydown.