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Geotechnical Issues in Pavement Design

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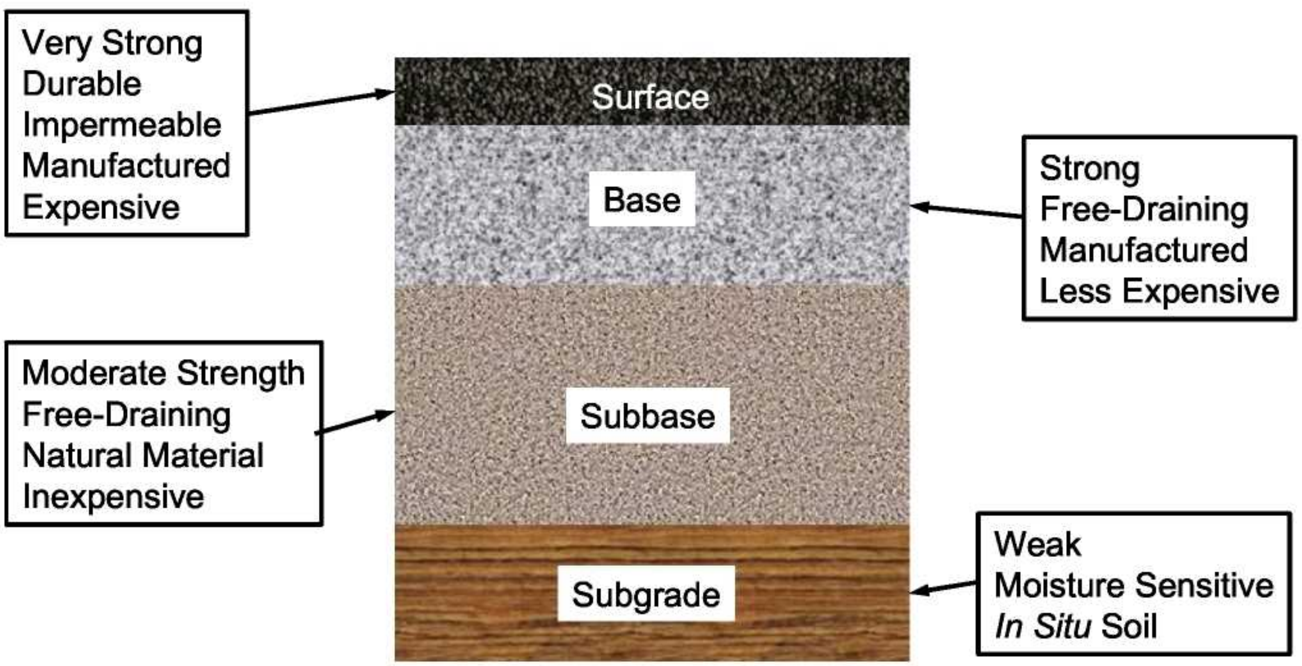
Geotechnical Aspects of Pavements

Reference Manual / Participant Workbook



National Highway Institute

NOTICE

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.

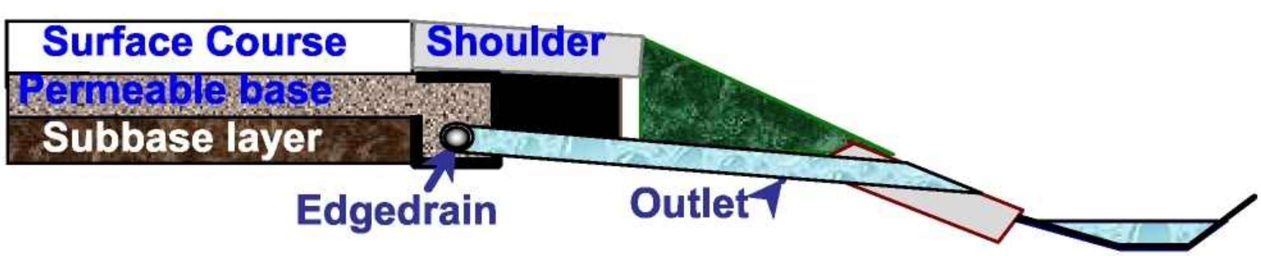


Figure 1-2. Pavement system with representative alternative features.

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.

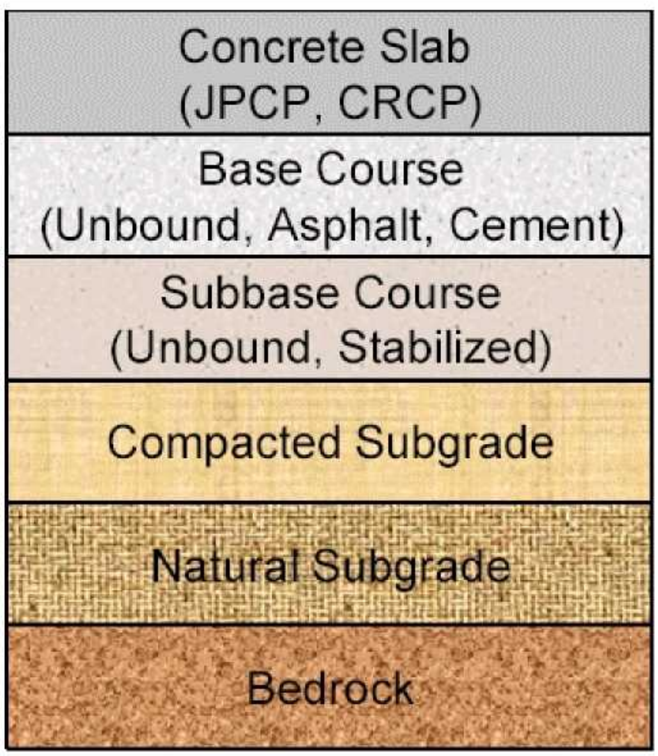


Figure 1-4. Variations for rigid pavement section (NCHRP 1-37A, 2002).

- *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.

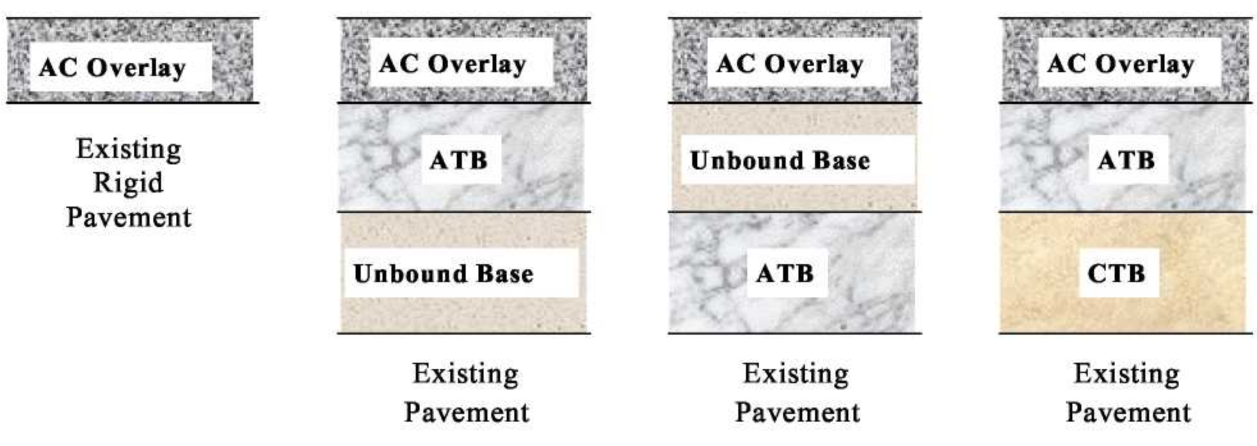


Figure 1-5. Typical composite pavement sections.

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.

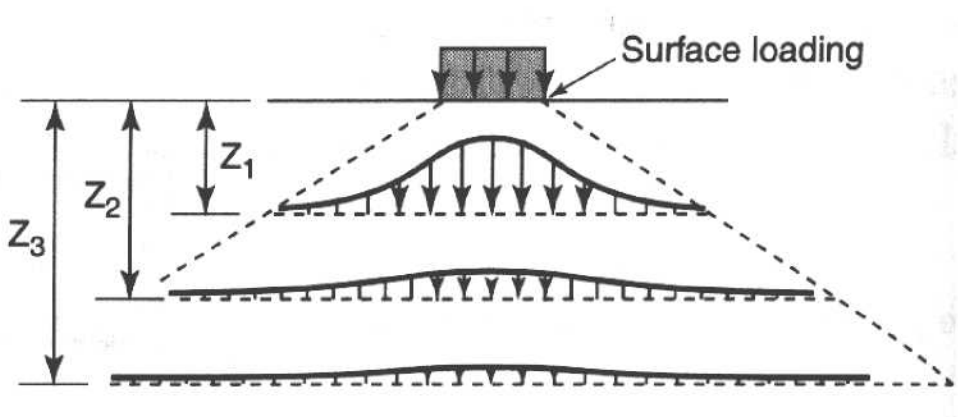


Figure 3-1. Attenuation of load-induced stresses with depth.

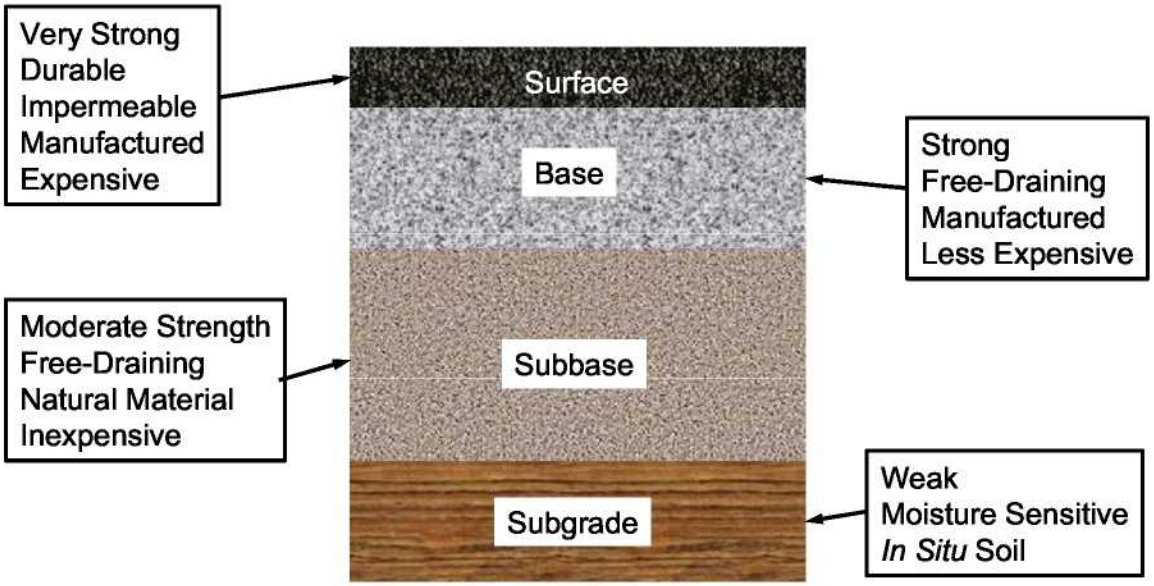


Figure 3-2. Variation of material quality with depth in a pavement system with ideal drainage characteristics.

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

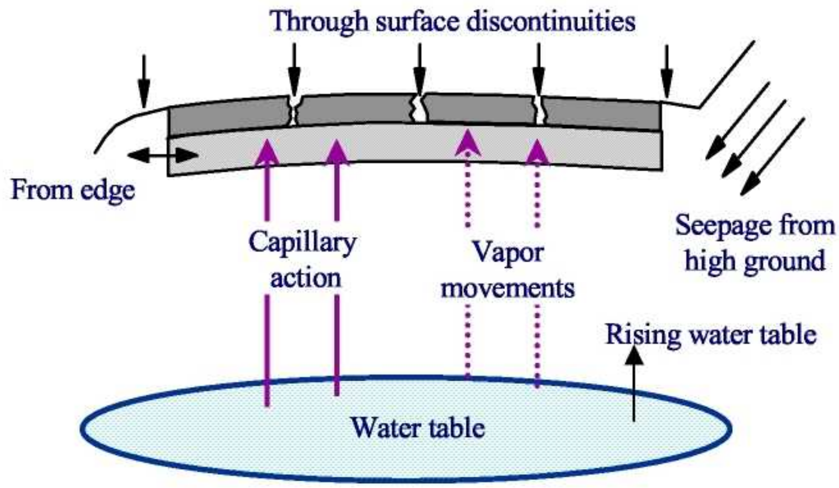


Figure 3-3. Sources of moisture in pavement systems (NHI 13126).

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

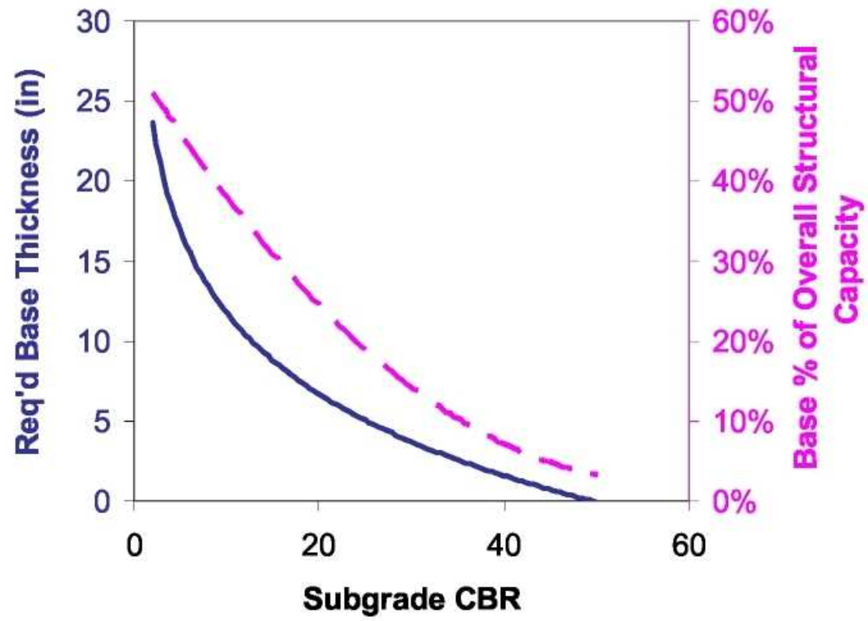


Figure 3-4. Impact of subgrade strength on pavement structural design (AASHTO 93 Design Guide: W_{18}=10M, 85% reliability, S_o=0.4, ΔPSI=1.7, a_1=0.44, a_2=0.14, m_2=1).

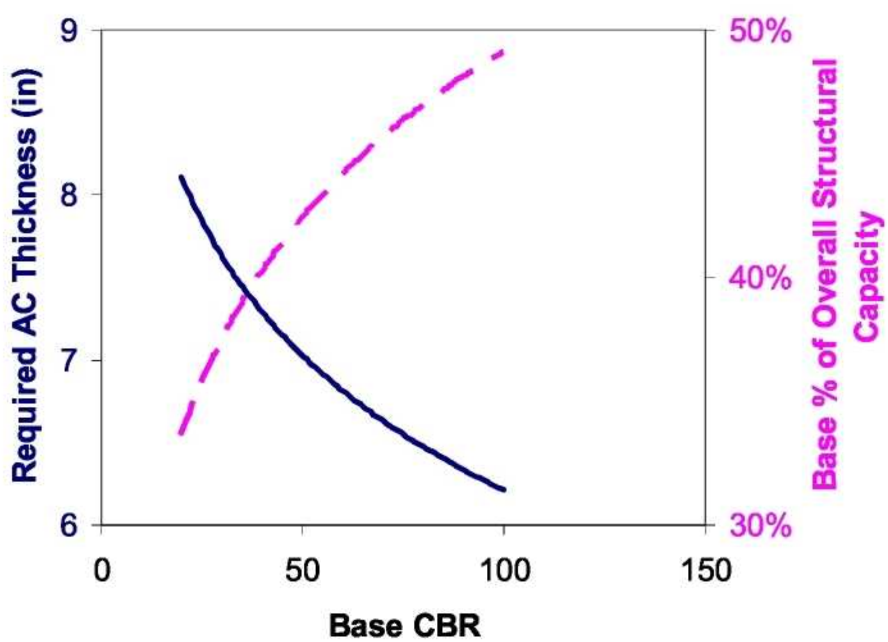
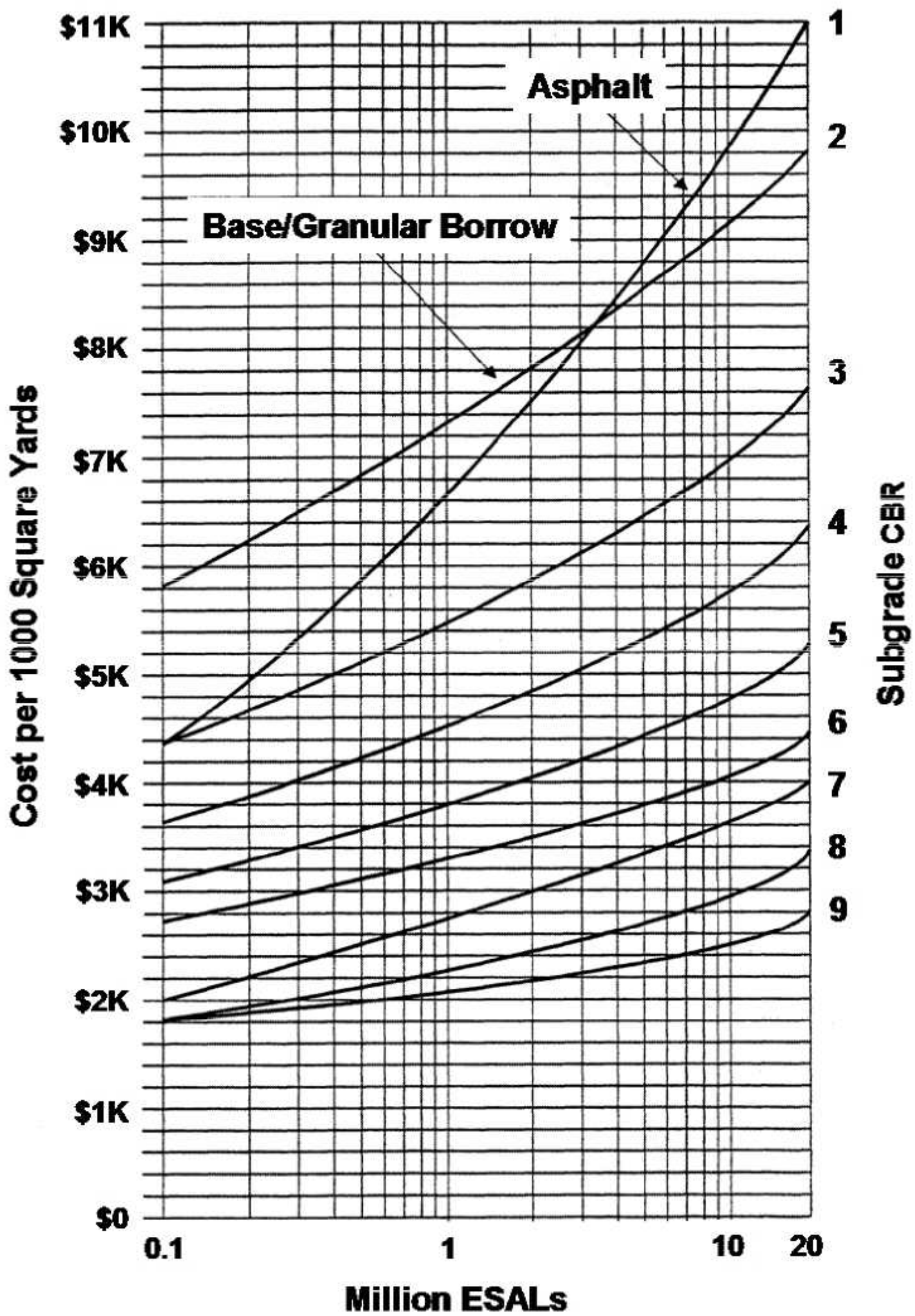


Figure 3-5. Impact of base strength on pavement structural design design (AASHTO 93 Design Guide: W_{18}=10M, 85% reliability, S_o=0.4, ΔPSI=1.7, a_1=0.44, m_2=1, subgrade CBR=4).



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.

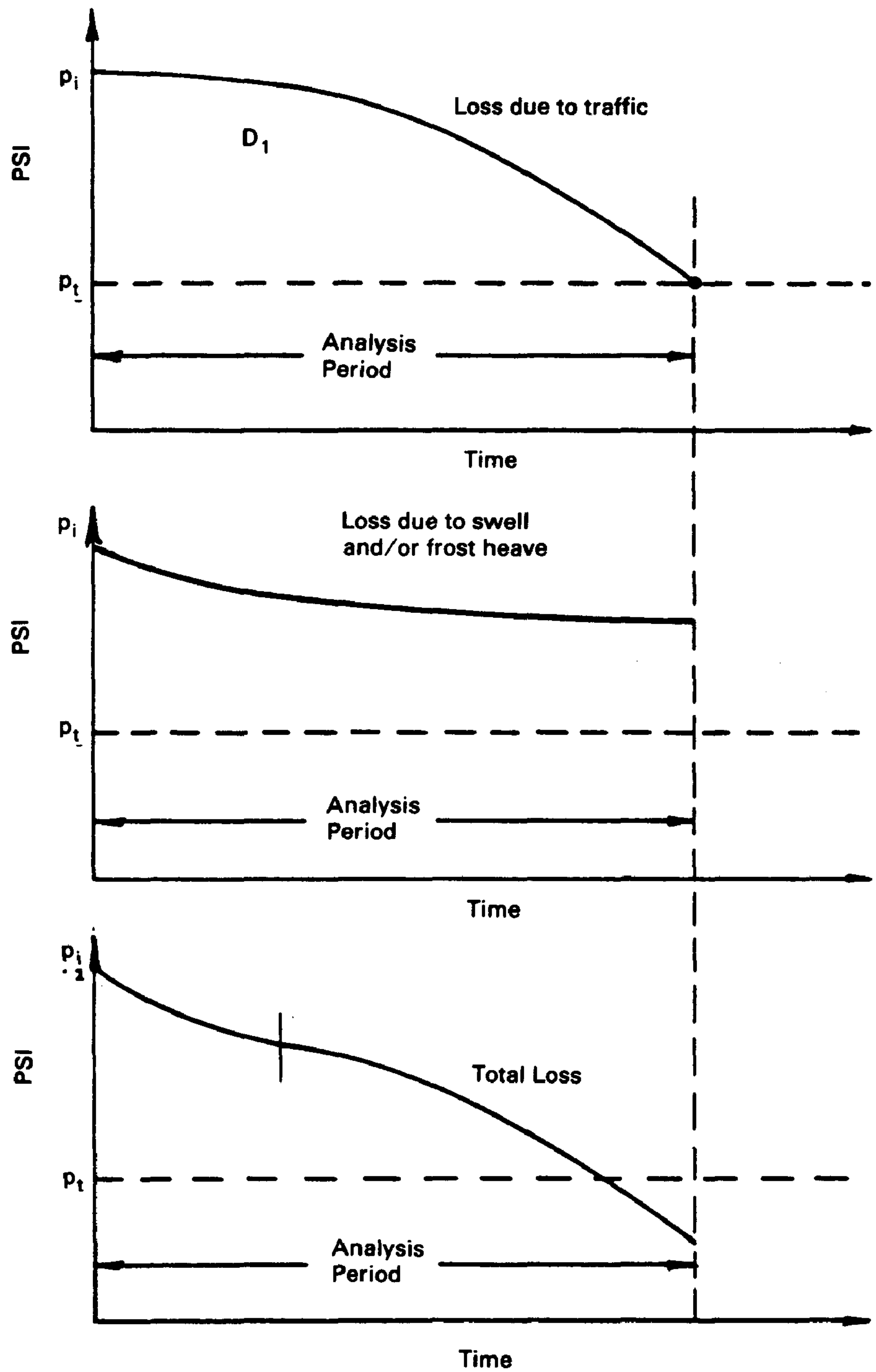


Figure 3-8. Pavement serviceability in the AASHTO Design Guides (AASHTO, 1993).

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

$$SN = a_1D_1 + a_2D_2 + a_3D_3 \tag{3.2}$$

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

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

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.

Flexible Pavements

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:

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

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.

<u>Rigid Pavements</u>

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.

$$\log W_{18} = 7.3\log(D+1) - 0.06 + \frac{\log\left[(4.5-p_t)/(4.5-1.5)\right]}{1+1.624\times 10^7/(D+1)^{8.46}}$$
$$+(4.22-0.32p_t)\left[\log\left(\frac{S_c}{215.63J}\right)\left(\frac{D^{0.75}-1.132}{D^{0.75}-18.42/(E_c/k)^{0.25}}\right)\right] \quad (3.5)$$

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.

<u>Sensitivity to Geotechnical Inputs</u>

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.

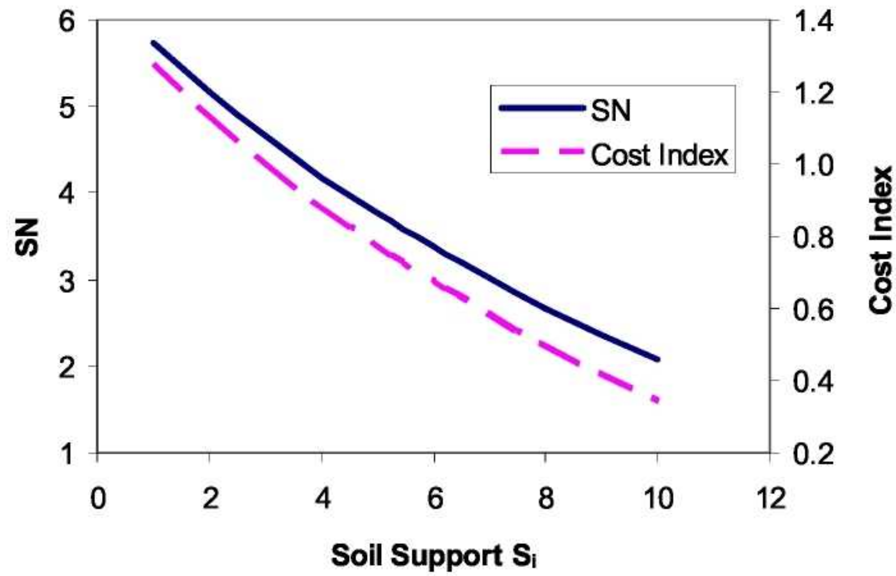


Figure 3-9. Sensitivity of 1972 AASHTO flexible pavement design to foundation support quality.

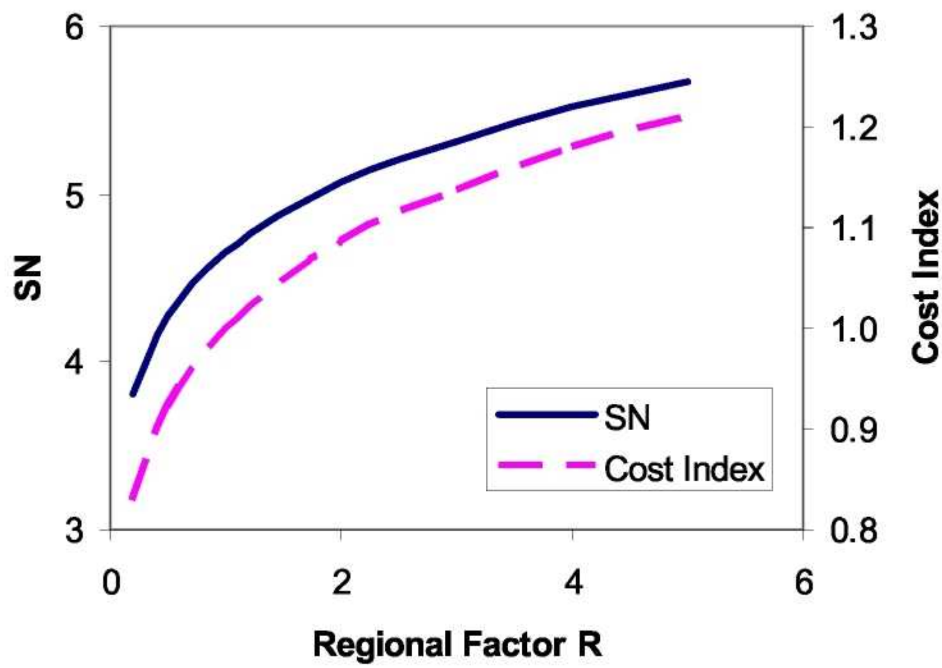


Figure 3-10. Sensitivity of 1972 AASHTO flexible pavement design to environmental conditions.

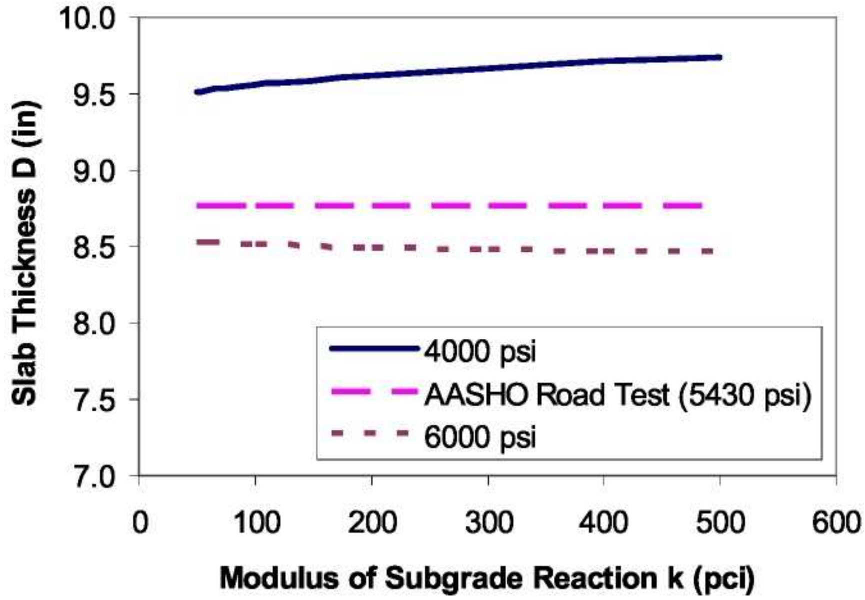


Figure 3-11. Sensitivity of 1972 AASHTO rigid pavement design to foundation stiffness (1 in = 25 mm; 1 pci = 284 MN/m^3).

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:

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3 \tag{3.7}$$

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:

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

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:

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

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

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

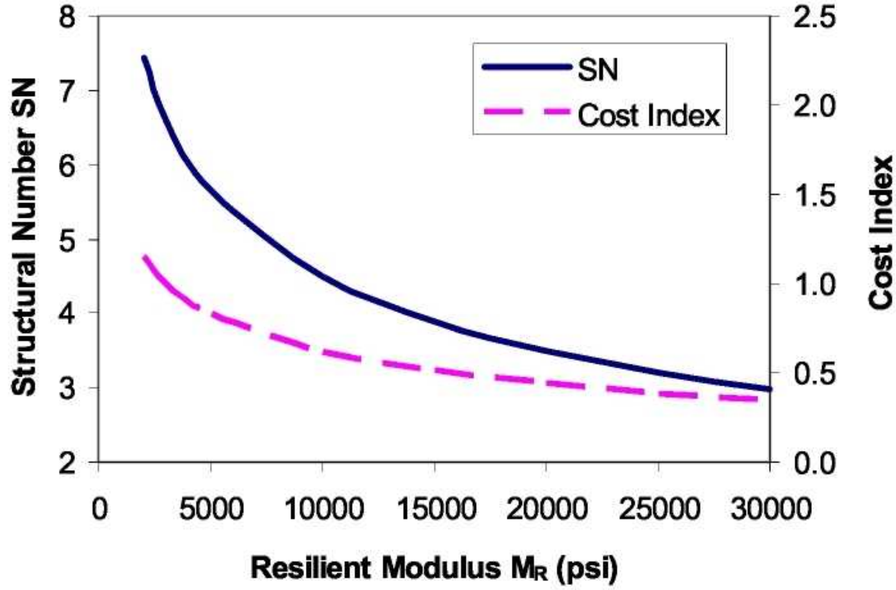


Figure 3-12. Sensitivity of 1986 AASHTO flexible pavement design to subgrade stiffness (1 psi = 6.9 kPa).

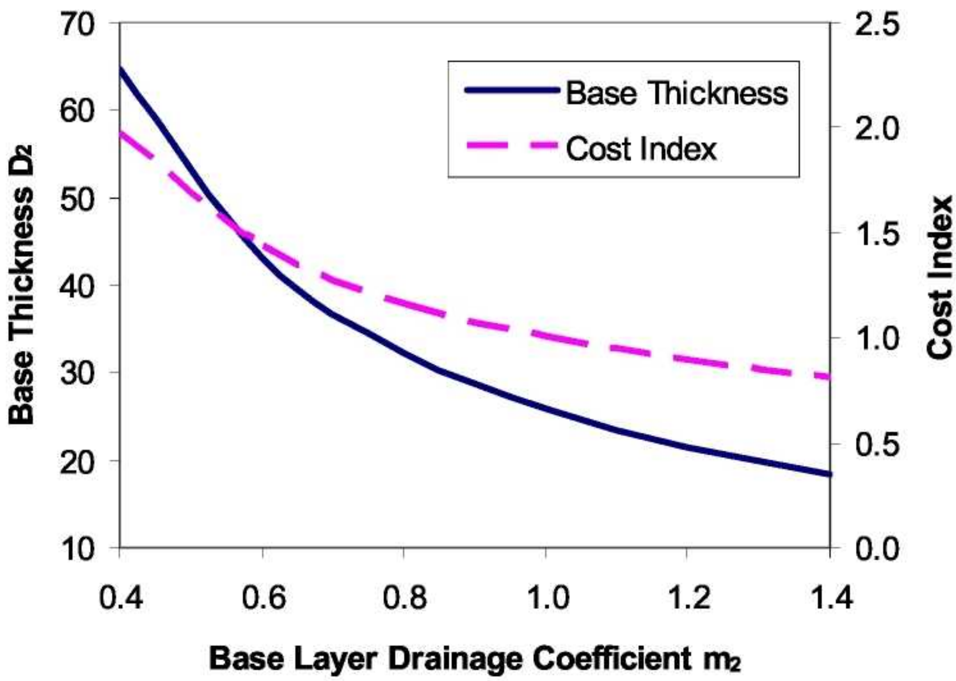


Figure 3-13. Sensitivity of 1986 AASHTO flexible pavement design to drainage conditions (1 inch = 25 mm).

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})	10×10^6 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)	4.2×10^6 psi (29 GPa)
Joint load transfer coefficient (J)	4.1

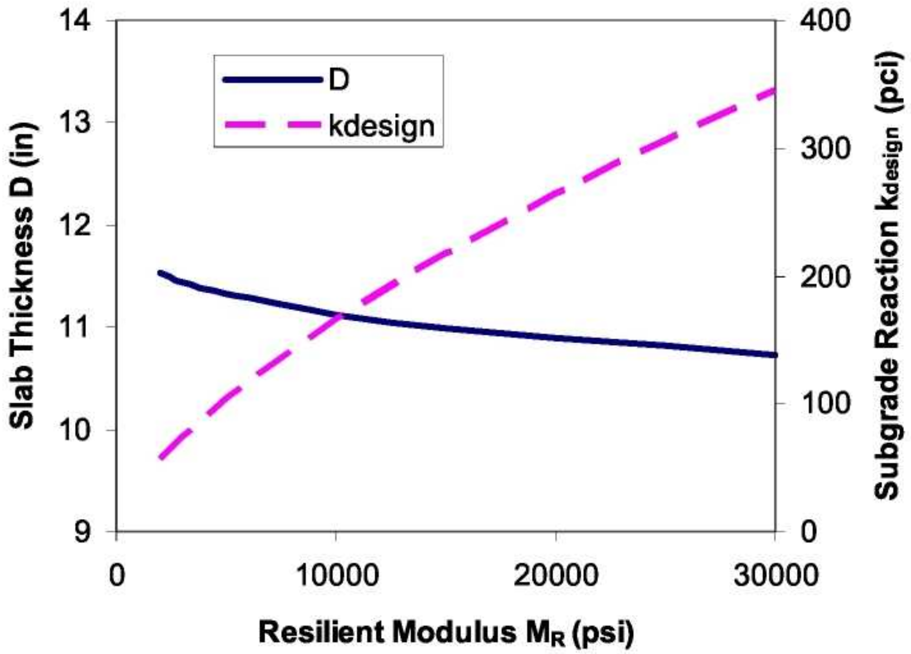


Figure 3-14. Sensitivity of 1986 AASHTO rigid pavement design to subgrade stiffness (1 inch = 25 mm; 1 psi = 6.9 kPa; 1 pci = 284 MN/m^3).

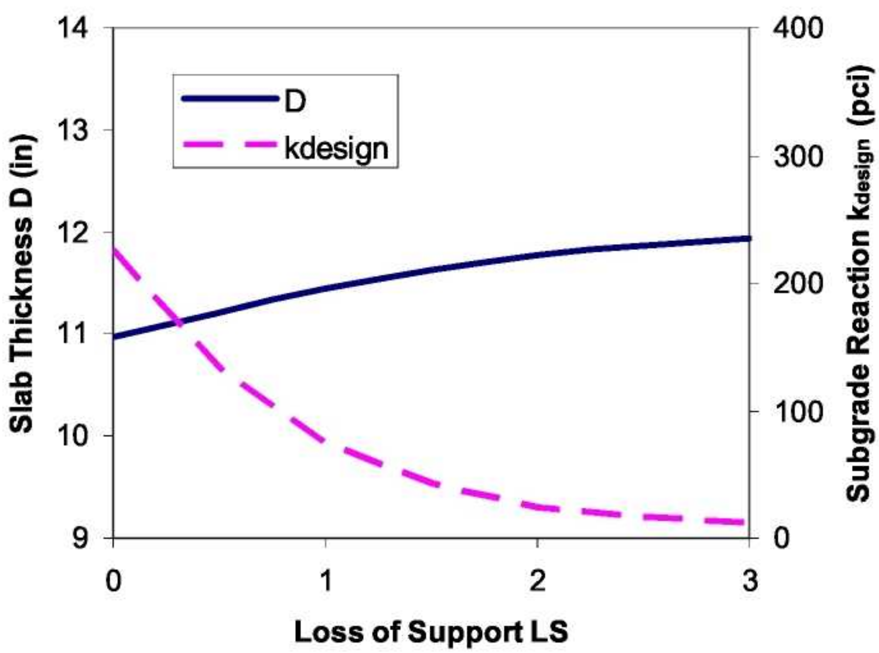


Figure 3-15. Sensitivity of 1986 AASHTO rigid pavement design to subbase erodibility (1 inch = 25 mm; 1 pci = 284 MN/m^3).

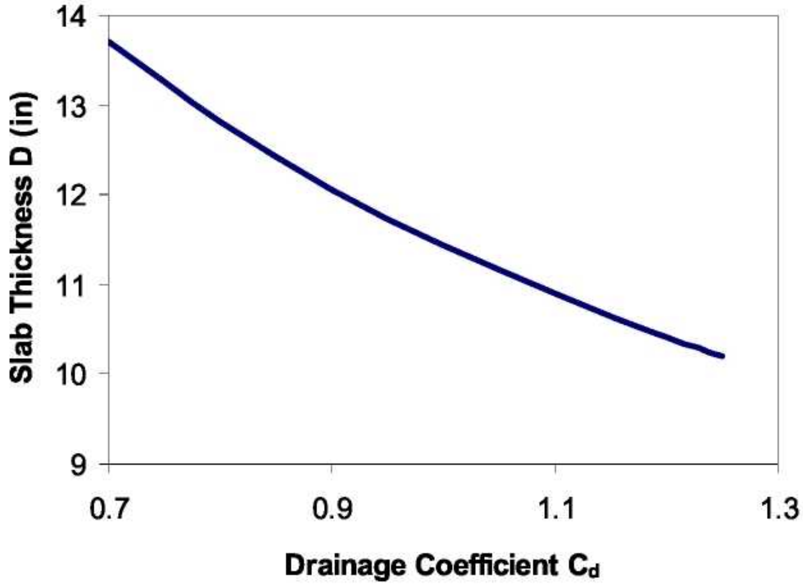


Figure 3-16. Sensitivity of 1986 AASHTO rigid pavement design to drainage conditions (1 inch = 25 mm).

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.

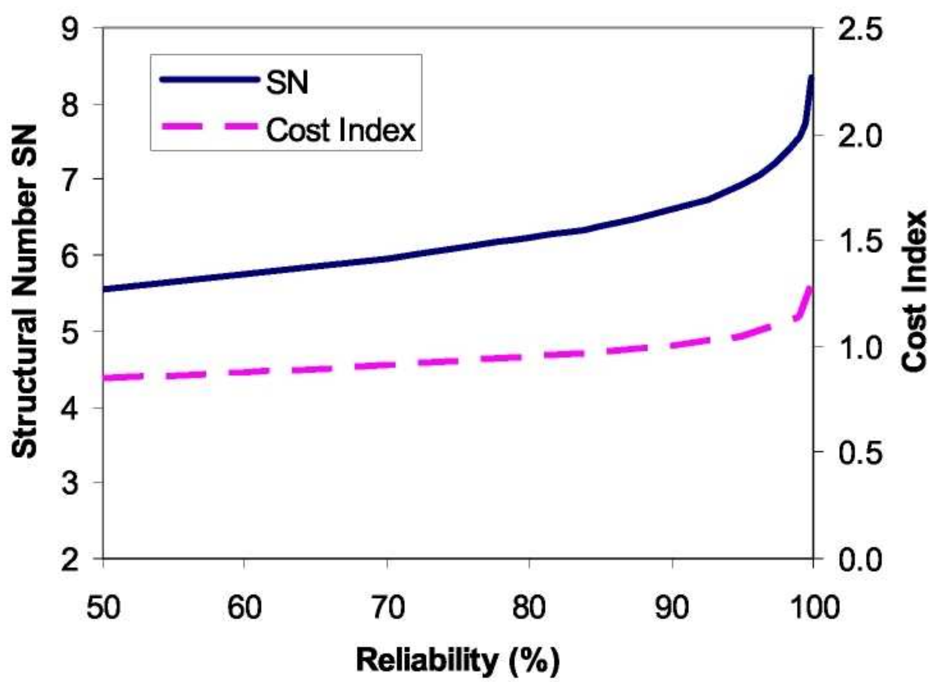


Figure 3-17. Sensitivity of 1986 AASHTO flexible pavement design to reliability level.

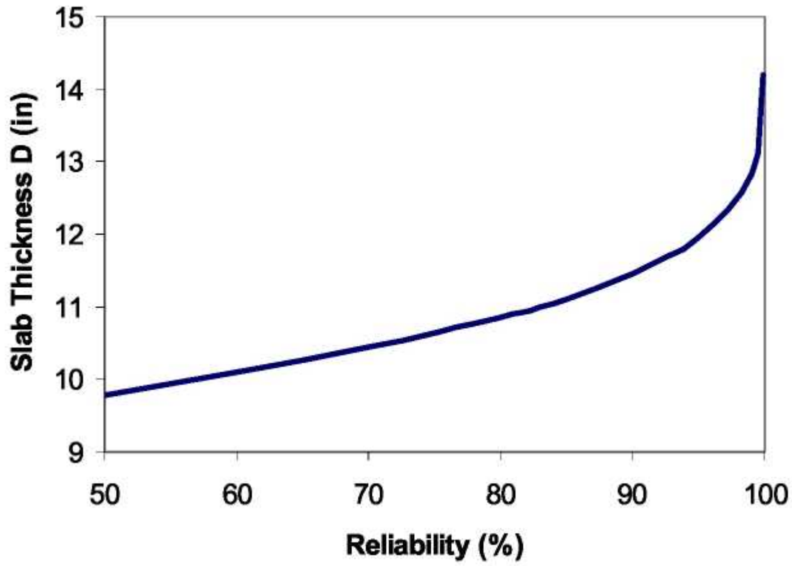


Figure 3-18. Sensitivity of 1986 AASHTO rigid pavement design to reliability level (1 inch = 25 mm).

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.

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[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.

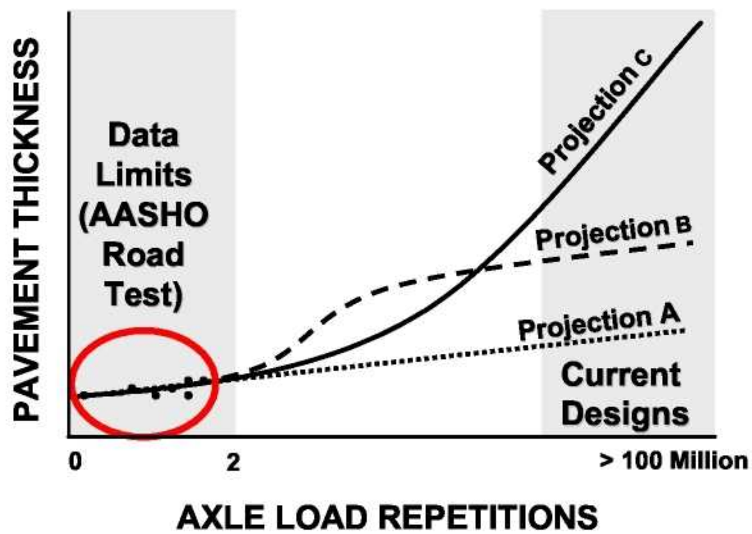

Figure 3-19. Extrapolation of traffic levels in current AASHTO pavement design procedures (NHI Course 131064).

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:

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

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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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:

$$\log_{10}\left(W_{18}\right) = Z_R S_0 + 9.36\log_{10}\left(SN+1\right) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{\left(SN+1\right)^{5.19}}} + 2.32\log_{10}\left(M_R\right) - 8.07 \qquad \text{(C.2)}$$

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:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \qquad \text{(C.3)}$$

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:

$$\Delta PSI = p_t - p_o = 4.2 - 2.5 = 1.7 \tag{C.5}$$

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.

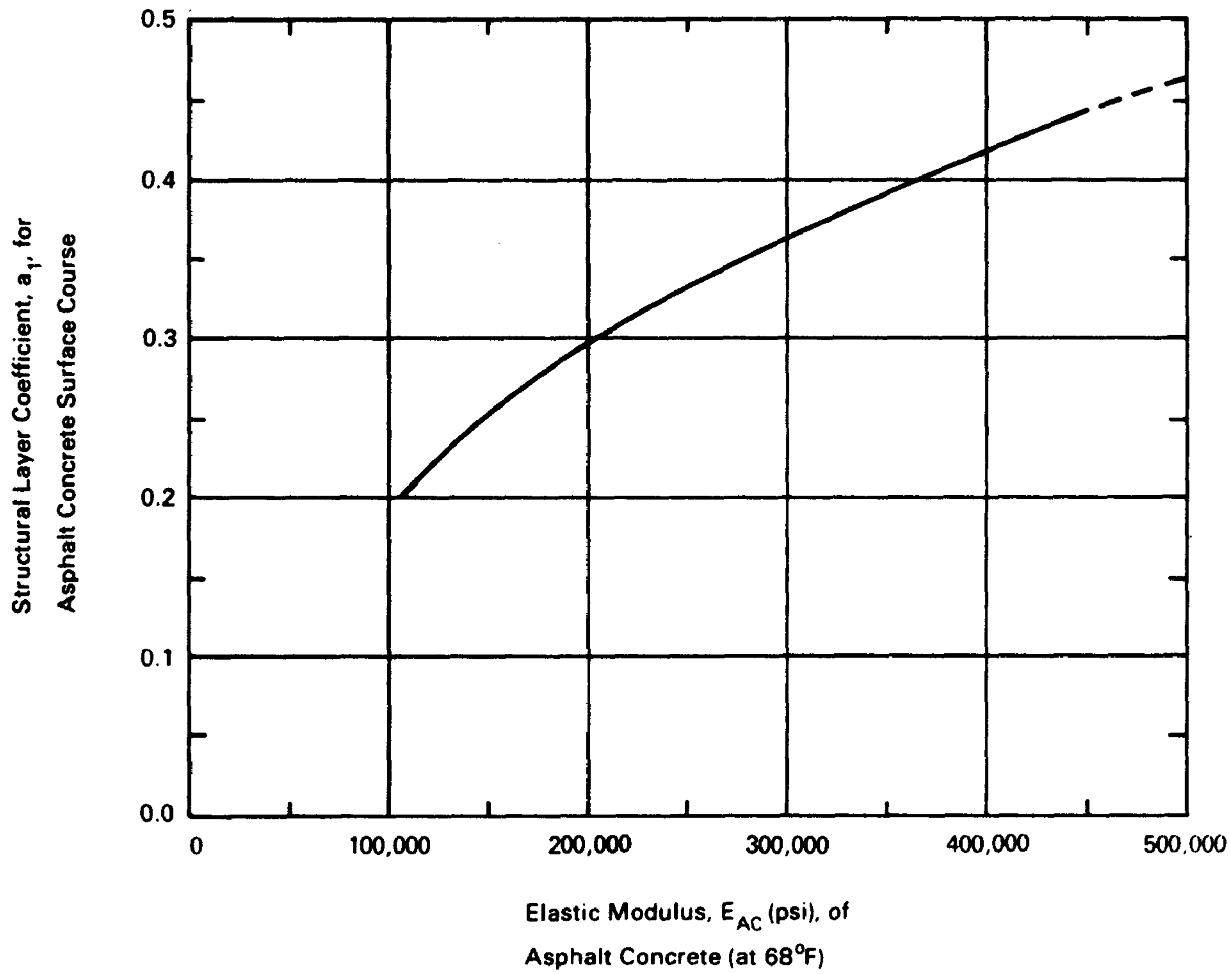

Figure C-1. Chart for estimating structural layer coefficient of dense-graded asphalt concrete based on the elastic (resilient) modulus (AASHTO, 1993).

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: $\Delta PSI = 1.7$ (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: $D_2 = \frac{SN_2}{m_2 a_2} = 14.3 \rightarrow 14$ inches.[1]

C.3 RIGID PAVEMENT STRUCTURAL DESIGN

Design Equation

The empirical expression relating traffic, pavement structure, and pavement performance for rigid pavements is:

$$\log_{10}(W_{18}) = Z_R S_o + 7.35\log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.5-1.5}\right]}{1+\frac{1.64\text{x}10^7}{(D+1)^{8.46}}} + (4.22-0.32p_t)\log_{10}\left[\frac{S_c C_d\left(D^{0.75}-1.132\right)}{215.63J\left[D^{0.75}-\frac{18.42}{(E_c/k)^{0.25}}\right]}\right] \tag{C.6}$$

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:

$$\Delta PSI = p_t - p_o = 4.4 - 2.5 = 1.9 \qquad \text{(C.7)}$$

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

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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.