# CHAPTER 4.0 GEOTECHNICAL EXPLORATION AND TESTING

## 4.1 INTRODUCTION

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

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

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

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

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

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

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

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

## 4.2 LEVELS OF GEOTECHNICAL EXPLORATION FOR DIFFERENT TYPES OF PAVEMENT PROJECTS

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

- new construction,
- reconstruction, and
- rehabilitation.

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

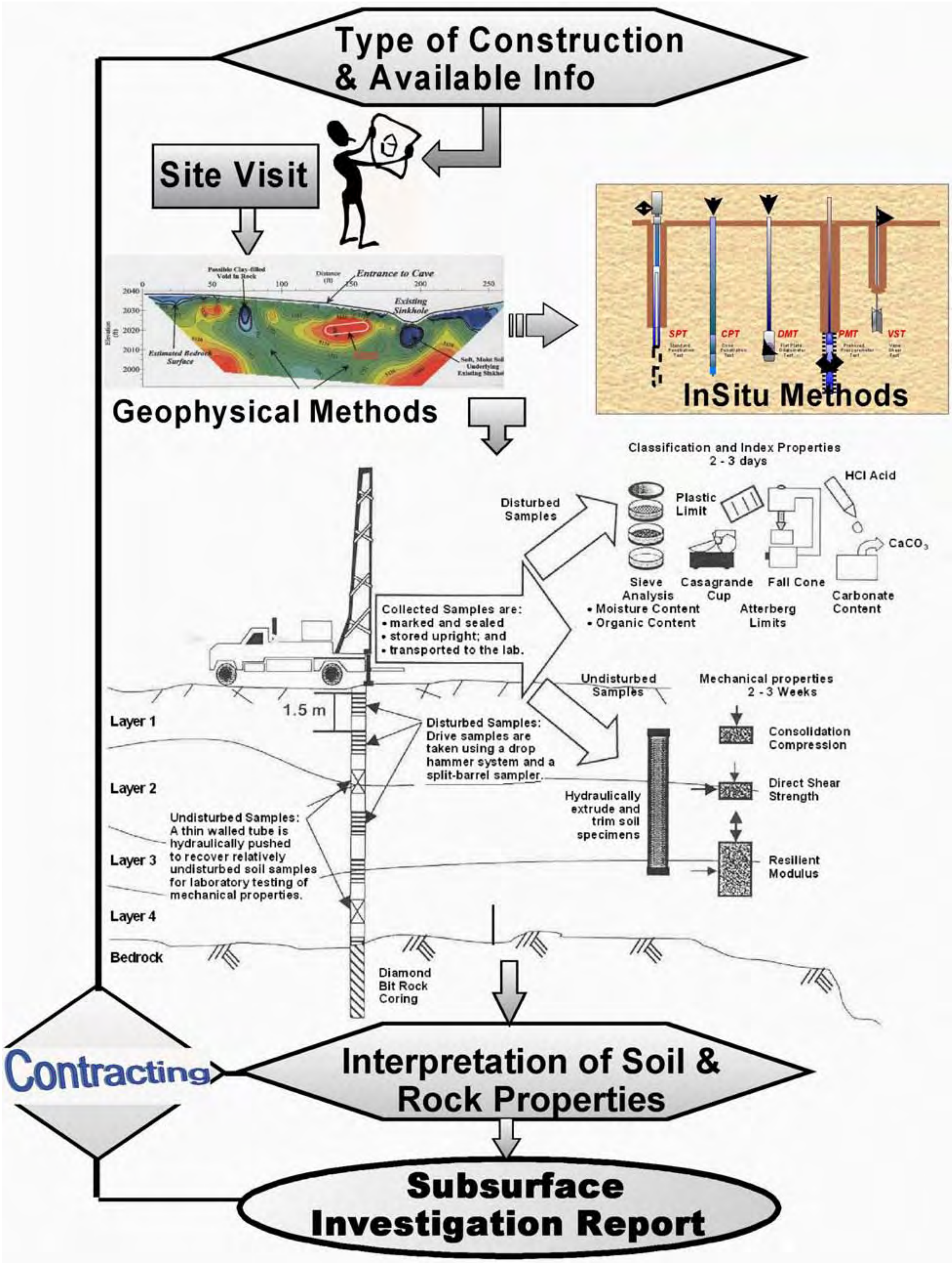



Figure 4-1. Geotechnical exploration and testing for pavement design.

### 4.2.1 New Pavement Construction

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

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

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

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

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

### 4.2.2 Reconstruction

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

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

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

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

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

### 4.2.3 Rehabilitation

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

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

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

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

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

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

### 4.2.4 Subsurface Exploration Program Objectives

As stated in the NCHRP 1-37A Design Guide, the objective of subsurface investigation or field exploration is to obtain sufficient subsurface data to permit the selection of the types, locations, and principal dimensions of foundations for all roadways comprising the proposed project, thus providing adequate information to estimate their costs. More importantly, these explorations should identify the site in sufficient detail for the development of feasible and cost-effective pavement design and construction.

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

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

## 4.3 SEARCH AVAILABLE INFORMATION

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

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

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

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

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

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

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

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

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

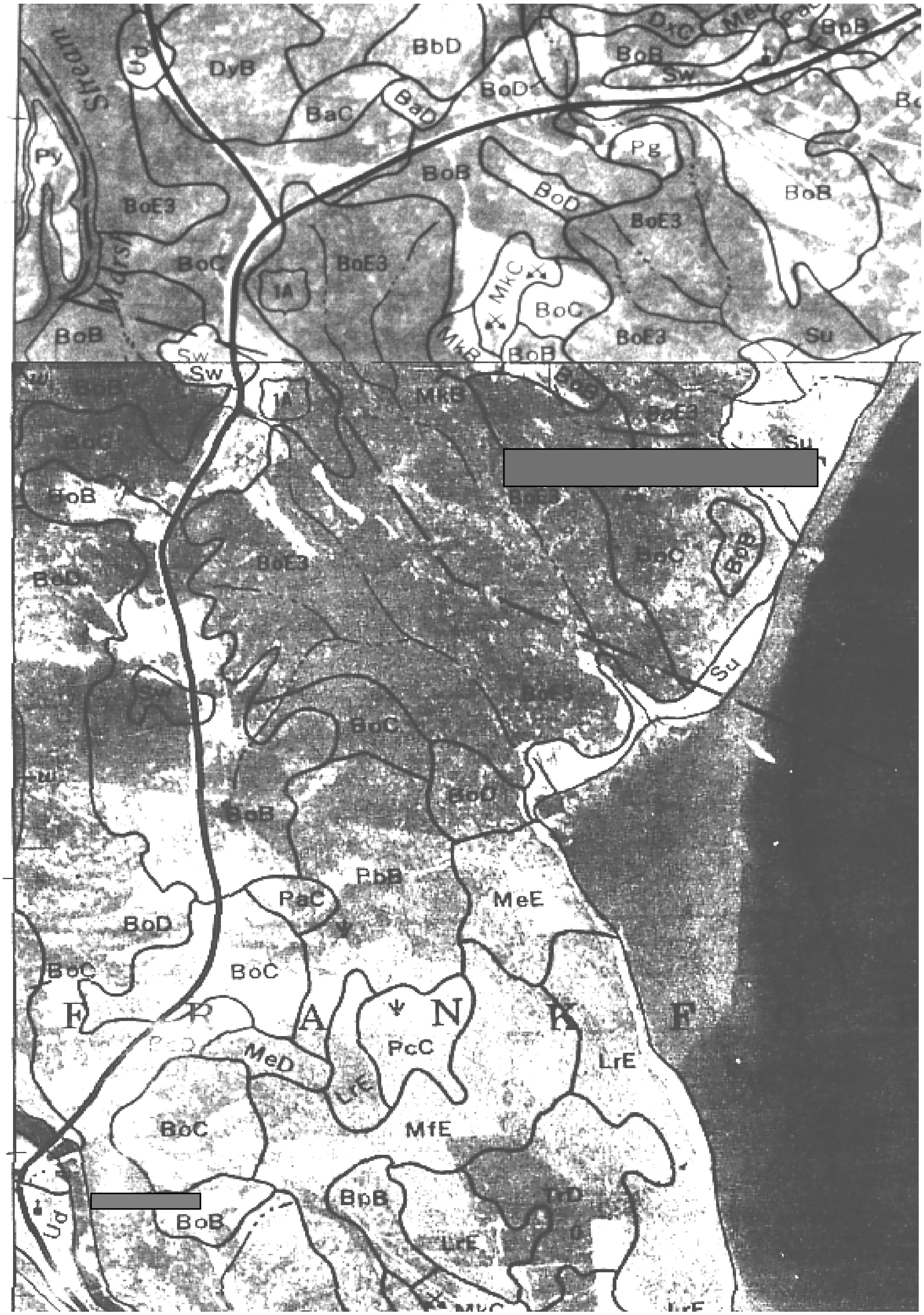



Figure 4-2. Soil Survey information along the Main Highway pavement alignment.

## 4.4 PERFORM SITE RECONNAISSANCE

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

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

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

## 4.5 PLAN AND PERFORM THE SUBSURFACE EXPLORATION PROGRAM

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

The subsurface investigation for any pavement project should be sufficiently detailed to define the depth, thickness, and area of all major soil and rock strata that will affect construction and long-term performance of the pavement structure. The extent of the exploration program depends on the nature of both the project and the site-specific subsurface conditions. To acquire reliable engineering data, each job site must be explored and analyzed according to its subsurface conditions. The engineer in charge of the subsurface exploration must furnish complete data so that an impartial and thorough study of practical pavement designs can be made.

### 4.5.1 Depth of Influence

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

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

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

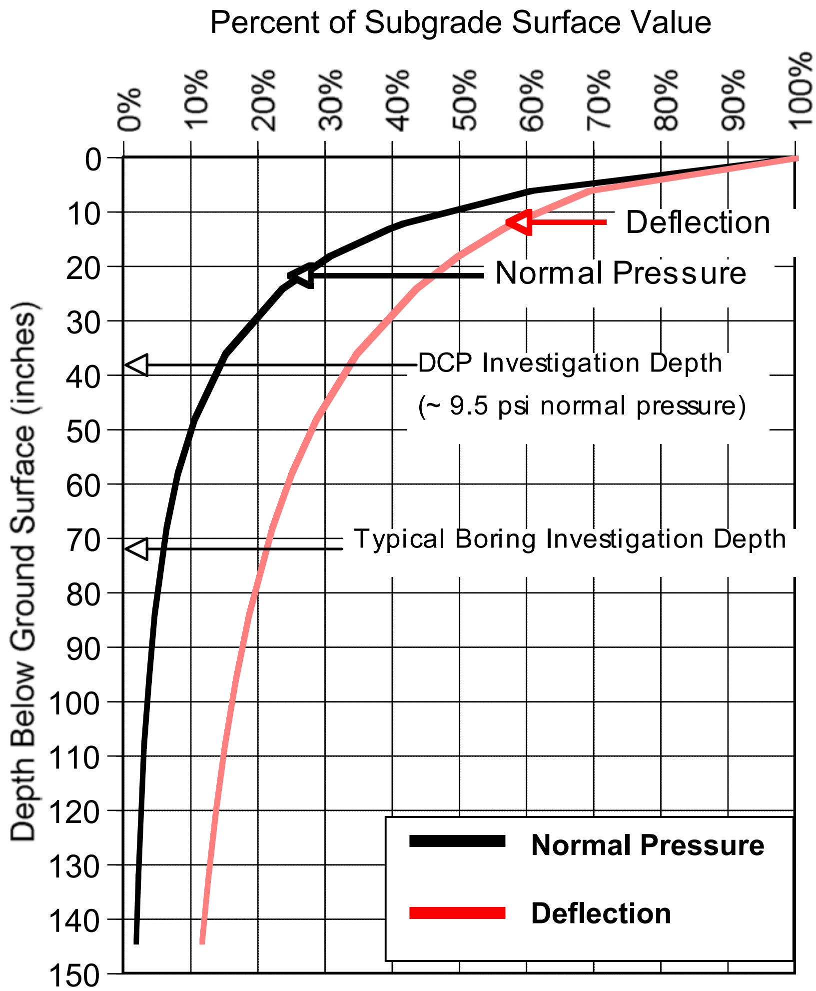



Figure 4-3. Typical zone of influence for an asphalt pavement section (Vandre et al., 1998).

### 4.5.2 Subsurface Exploration Techniques

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

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

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

### 4.5.3 Remote Sensing

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

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

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

### 4.5.4 Geophysical Investigations

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

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

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

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

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

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

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

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

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

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

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

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

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

$$r \geq 0.7a_e$$

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

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

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

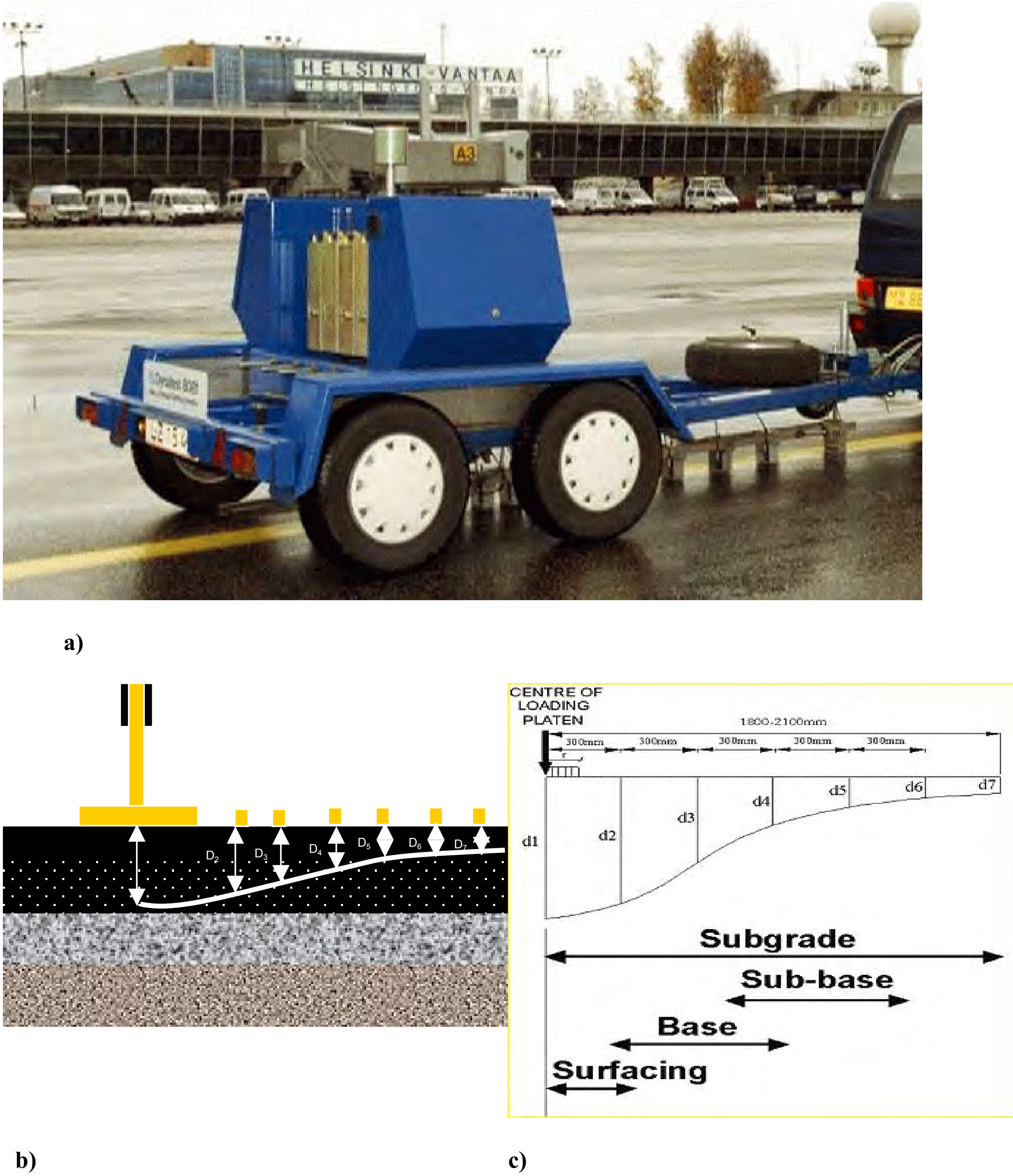



Figure 4.4. FWD showing: a) typical equipment; b) schematic of procedure; and, c) sketch of deflection bowl for interpretation of results.

Commentary

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

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

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

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

**Table 4-3. Surface resistivity (SR).**

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

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

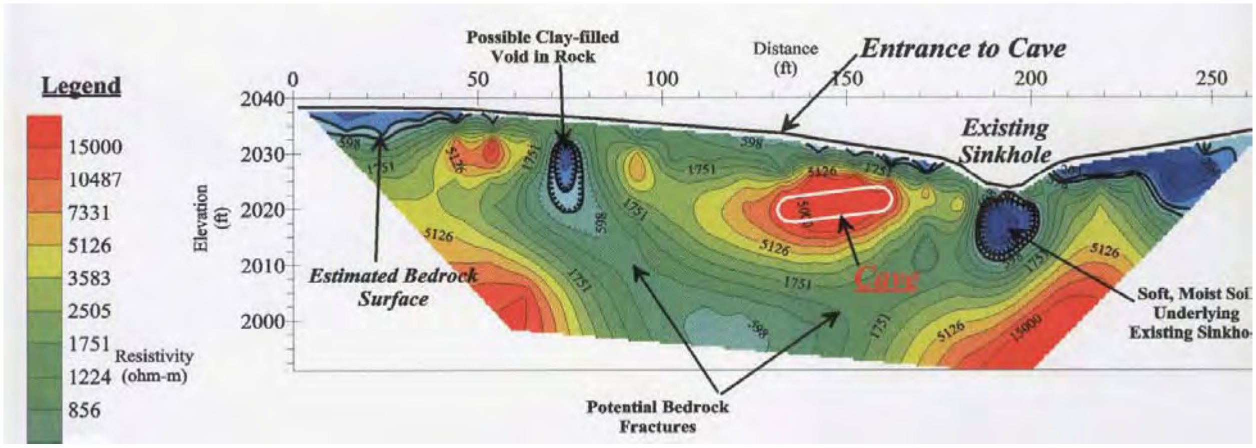



Figure 4-5. Two-dimensional cross-section resistivity profile for detection of sinkholes and caves in limestone (from Schnabel Engineering Associates).

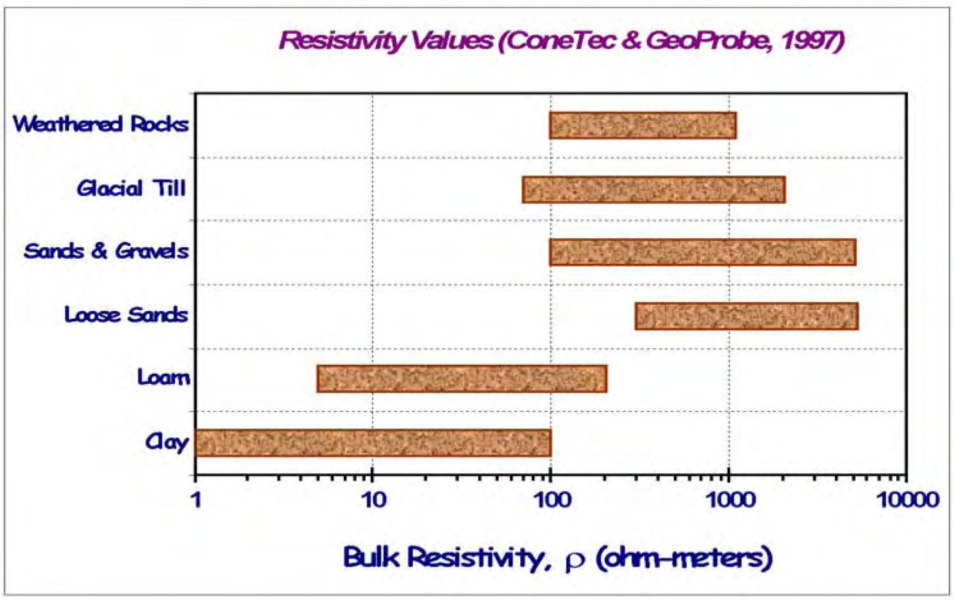



Figure 4-6. Representative values of resistivity for different soils (Mayne et al., 2002).

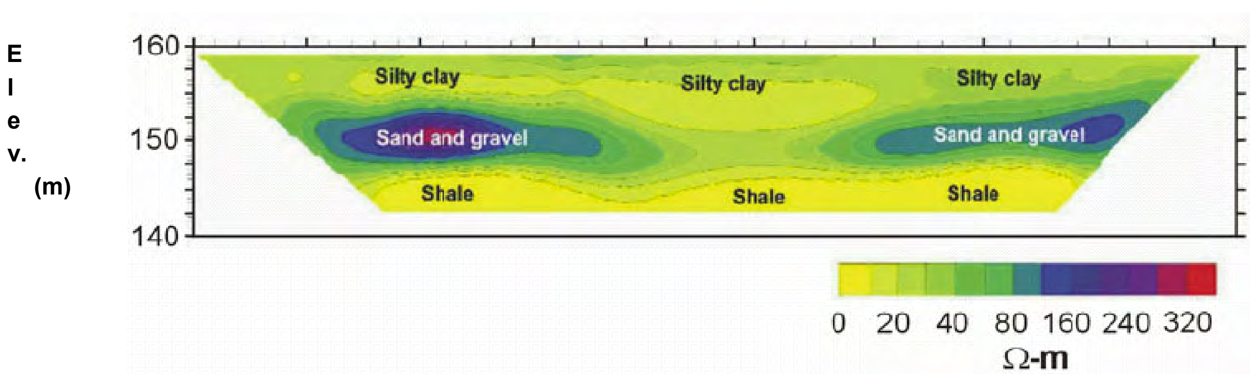



Figure 4-7. Resistivity data showing stratigraphic changes (Advanced Geosciences, Inc.).

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

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

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

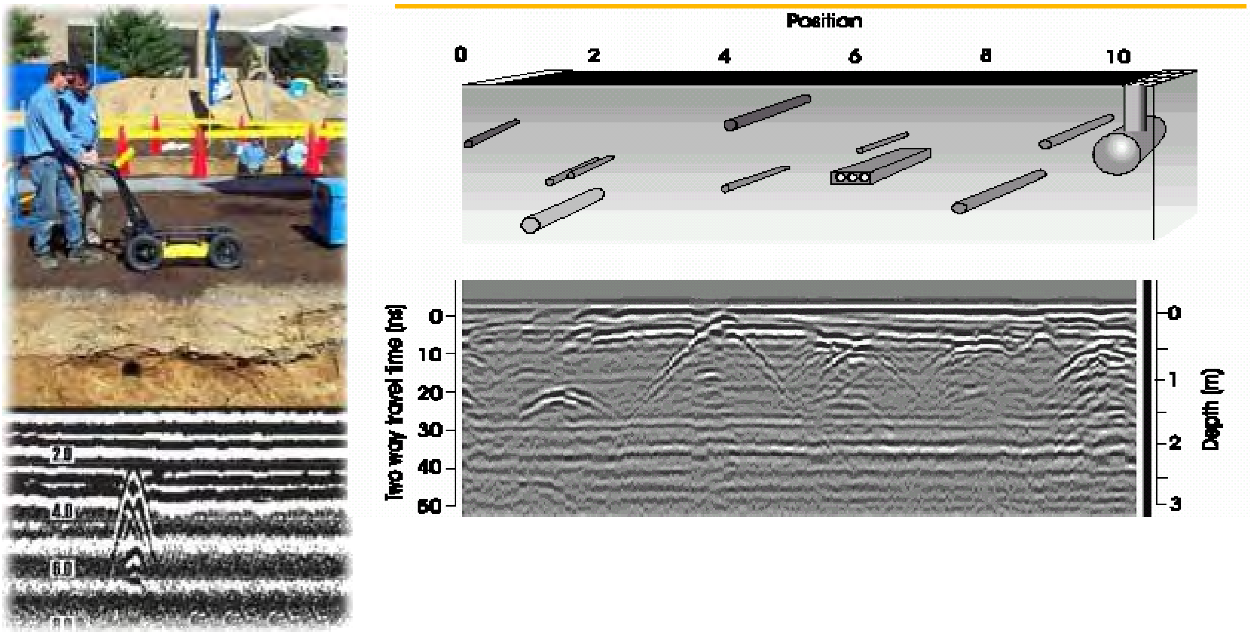



Figure 4-8. Representative ground-coupled GPR results showing buried utilities and soil profile (from EKKO Sensors & Software: www.sensoft.on.ca).

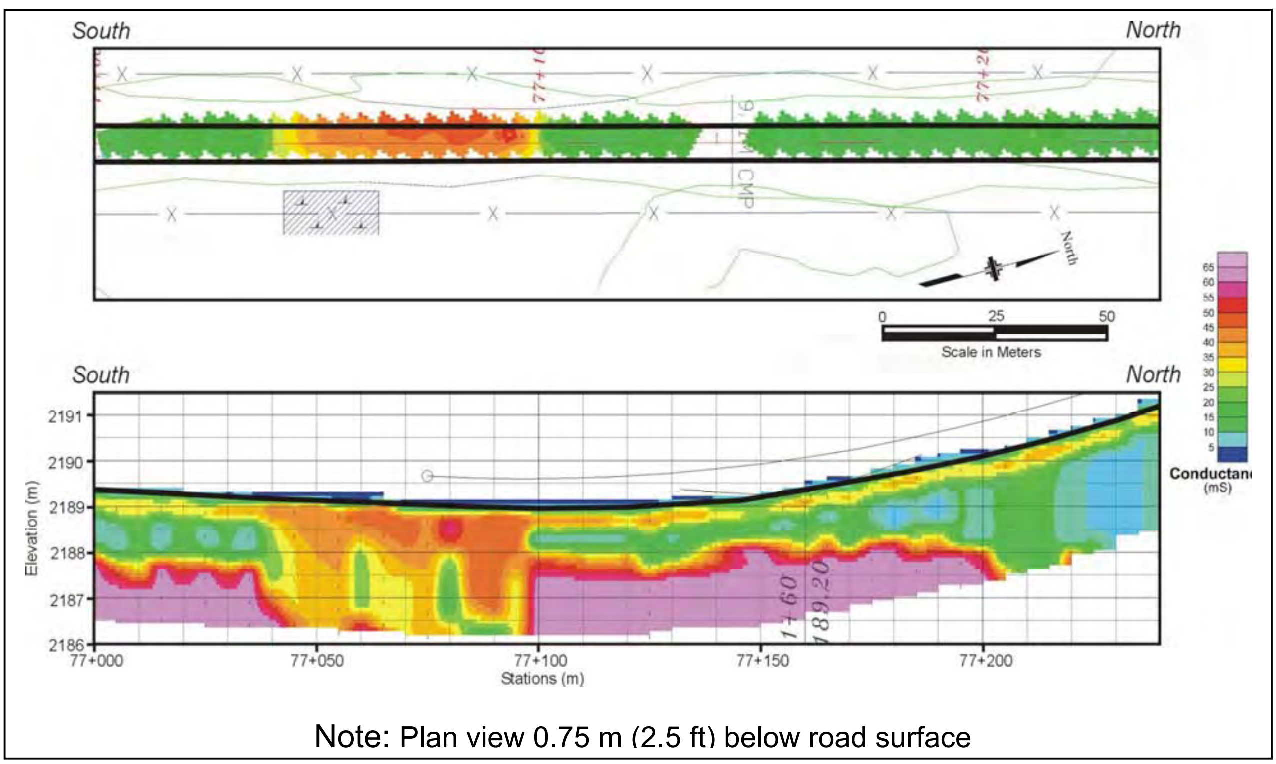



Figure 4-9. Conductivity results along a road in New Mexico (Blackhawk GeoServices, Inc).

**Table 4-5. Electromagnetic conductivity (EM).**

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

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

**Table 4-6. Mechanical wave using seismic refraction.**

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

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

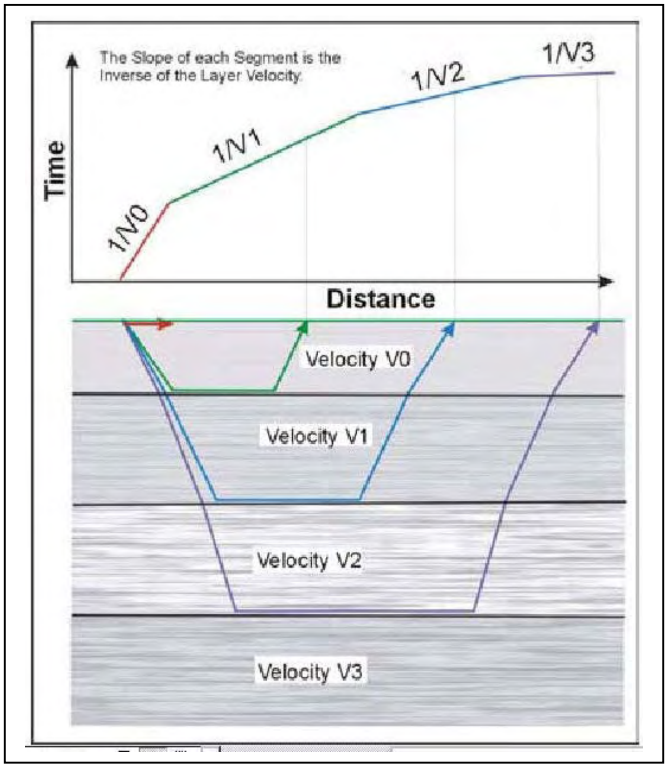



Figure 4-10. Seismic refraction survey (Blackhawk GeoServices, Inc.).

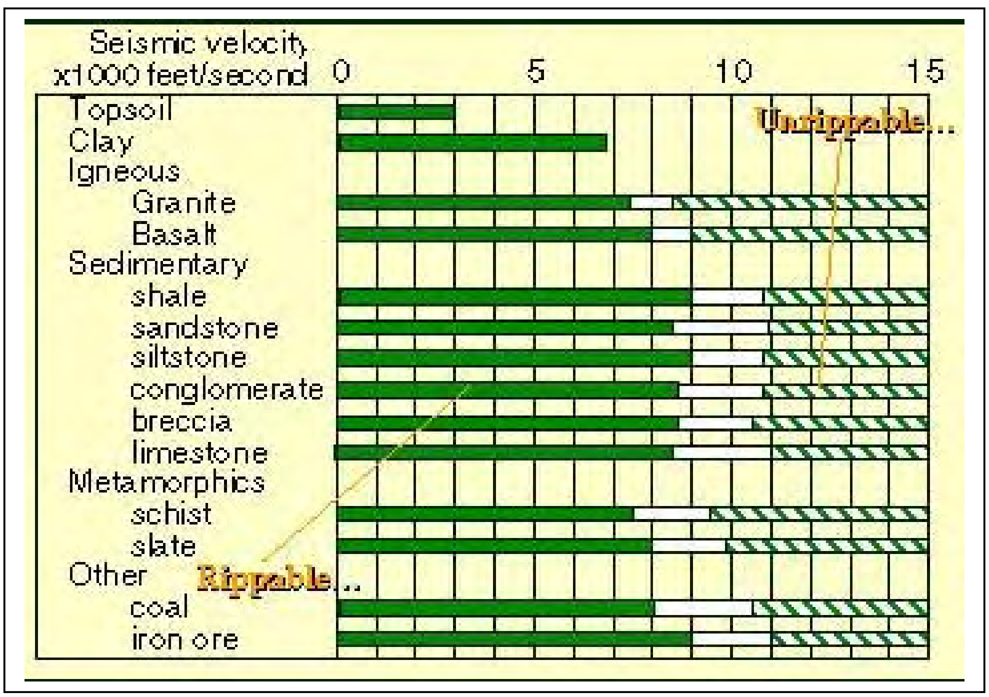



Figure 4-11. Rippability (using a D9 Caterpillar tractor) versus seismic velocity (Caterpillar Handbook of Ripping, 8th Edition).

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

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

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

### 4.5.5 In-Situ Testing

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

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

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

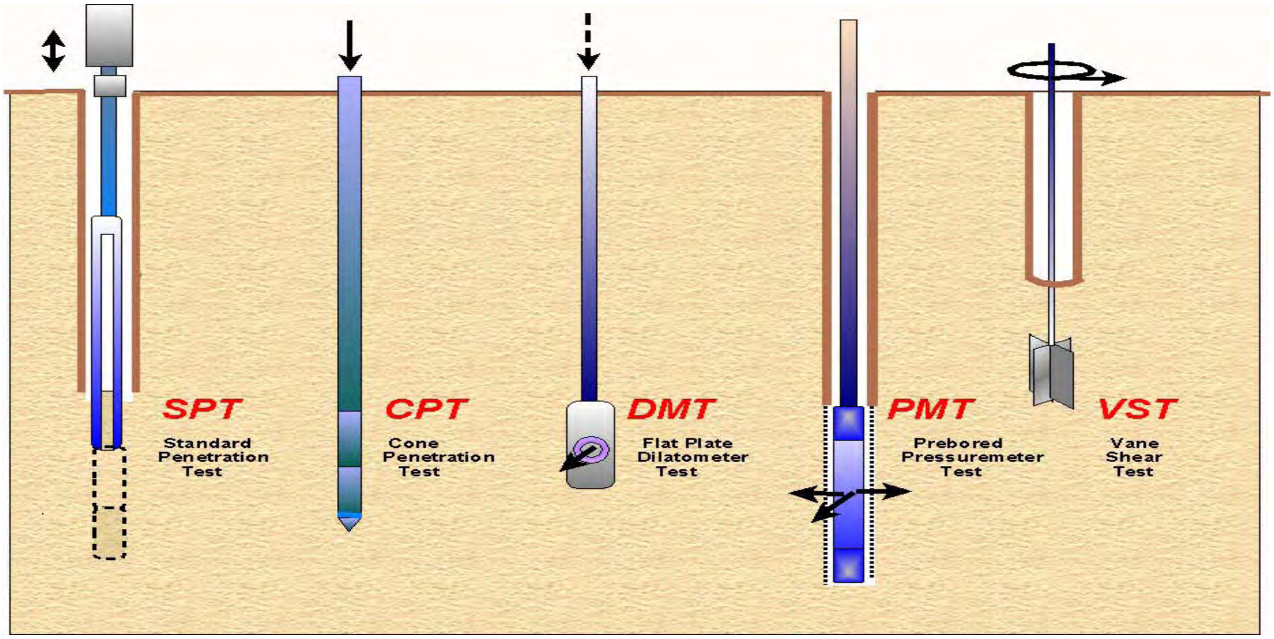



Figure 4-12. Common in-situ tests for pavement evaluation (Mayne et al., 2002).

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

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

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

### 4.5.6 Borings and Sampling

The final exploration method includes drilling bore holes or, in some cases, making excavations to obtain samples. This is the most complex and expensive part of the exploration program, and requires a great degree of care. Disturbed and undisturbed samples of the subsurface materials must be obtained for laboratory analyses (and/or tested in the field) to determine their engineering properties and verify geophysical and in-situ exploration results.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

| | |
|---|---|
| Reference Procedures | ASTM D 6951.<br>Standard Test Method for Use of the Dynamic Core Penetrometer in Shallow Pavement Applications.<br>The method is also described in FHWA TS-78-209. |
| Purpose | Another type of test that can be performed in the field to measure the strength of soils in-place, and is being used more commonly for pavement design purposes to estimate the in-place strength of both fine- and coarse-grained soils. |
| Procedure | The principle behind the DCP is that a direct correlation exists between the strength of a soil and its resistance to penetration by solid objects, such as cones (Newcombe and Birgisson, 1999). The DCP consists of a cone attached to a rod that is driven into soil by means of a drop hammer that slides along the penetrometer shaft. The mass of the hammer can be adjusted to 4.6 and 8 kg (10 and 18 lbs) with the lighter weight applicable for weaker soils. According to NCHRP Synthesis 278 (Newcomb and Birgisson, 1999), more recent versions of the DCP have a cone angle of 60 degrees, with a diameter of 20 mm (0.8 in.). |
| Commentary | A number of empirical correlations exist to relate the DCP penetration index (DPI) to subgrade strength parameters required for pavement design. The most widely used is (Webster et al., 1994):<br><br>$CBR = 292/(DPI)^{1.12}$ for gravel, sand, and silt<br>$CBR = 1/0.002871\ DCP$ for highly plastic clays<br>$CBR = 1/(0.017\ DCP)^2$ for low plasticity clays<br><br>The above methods were based on a database of field CBR versus DCP penetration rate values collected for many sites and different soil types, and correlated to test results by others (*e.g.,* log CBR = 2.61 – 1.26 log DCP as developed by Kleyn, 1975, and currently used by the Illinois DOT). For DCPs with automatic release hammers (*e.g.,* the Israeli automated DCP), CBR values are about 15% greater than the above correlations for manual hammers (after Newcomb and Birgisson, 1999). |

| ADVANTAGES | DISADVANTAGES |
|---|---|
| - Can be operated by one or two people<br>- Site access for testing not a problem<br>- Equipment is simple, rugged, and inexpensive<br>- Continuous record of soil properties with depth & immediate results<br>- Can be used in pavement core holes<br>- Suitable in many soil types & can perform in weak rocks<br>- Fair reliability (COV ~ 15 – 22 %)*<br>- Available throughout the U.S. | - Does not obtain a sample<br>- Index tests only<br>- High variability and uncertainty in gravelly soils<br>- Limited depth to 1 m (3.3 ft) (however, adequate for most rehab projects and good for rapid surficial characterization).<br>- Extraction of cone can be difficult. |

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

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

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

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

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

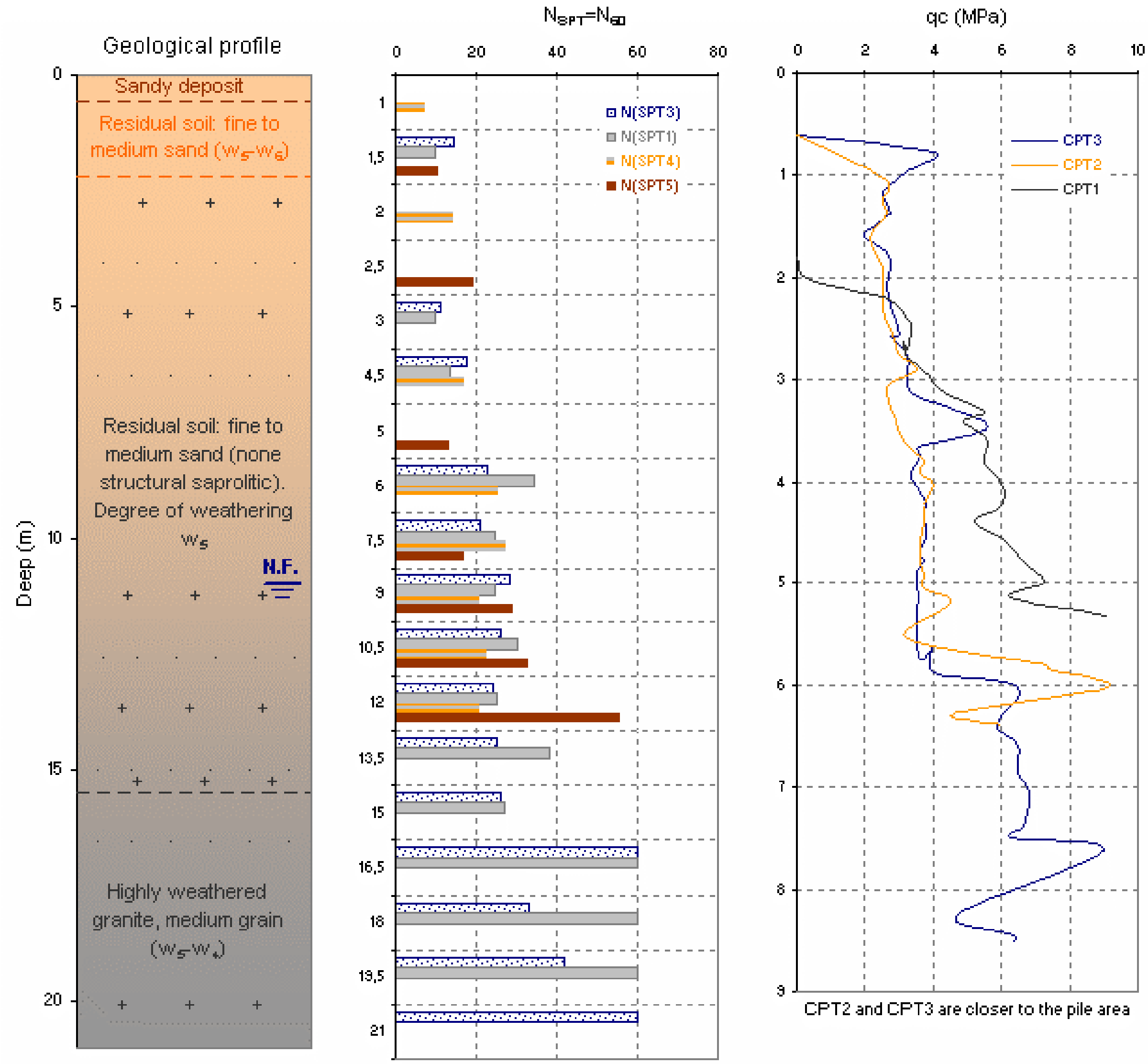



Figure 4-13. CPT log in comparison to SPT data from several locations.

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

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

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

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

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

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

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

***Frequency (number/spacing) of Borings***

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

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

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

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

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

As previously indicated in the introduction of Section 4.5.6, borings should be taken to a minimum depth of 1.5 – 2 m (5 – 7 ft) below the proposed pavement subgrade elevation, with at least a few borings taken to 6 m (20 ft) below the grade line. These deeper borings should also be used to determine the water table depth and occurrence of bedrock. Deeper borings are not generally required for rehabilitation projects, unless the previous section experienced premature failure due to subgrade conditions or there is a change in vertical alignment. All borings should extend through unsuitable foundation strata (for example, unconsolidated fill, highly organic materials, or soft, fine-grained soils) to reach relatively hard or compact materials of suitable bearing capacity to support the pavement system. Borings should extend a minimum of 1.5 m (5 ft) into relatively stiff or dense soils beneath soft deposits. Borings in potentially compressible fine-grained strata of great thickness should extend to a depth where the stress from superimposed traffic loads or a thick embankment is so small (less than 10% of the applied surface stress) that consideration will not significantly influence surface settlement.

Greater depth of borings may be required where deep cuts are to be made, side hill cuts are required, large embankments are to be constructed, or subsurface information indicates the presence of weak (or water-saturated) layers. In those cases, the borings should be deep enough to provide information on any materials that may cause problems with respect to stability, settlement, and drainage. For side hill cuts, additional borings should be performed on the uphill side in uniform soil conditions and on the uphill and downhill side for nonuniform conditions. Additional borings may be required for slope stability considerations and analysis.

Where stiff or compact soils are encountered at the surface and the general character and location of rock are known, borings should extend into sound rock. Where the location and character of rock are unknown or where boulders or irregularly weathered materials are likely to be found, the boring penetration into rock should be increased (NCHRP 1-37A, 2003), as discussed later in this section.

Take sufficient and appropriate auger, split tube, or undisturbed samples of all representative subsoil layers, as discussed in the next section. The soil samples must be properly sealed and stored to prevent moisture loss prior to laboratory testing. Prepare boring logs and soil profiles from this data.

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

***Soil Sampling (after NCHRP 1-37A)***

Sampling will vary with the type of pavement project. For new construction projects, a majority of the samples taken will most likely be the disturbed type, such as those obtained by split barrel samplers. This will permit visual identification and classification of the soils encountered, as well as identification by means of grain size, water content, and Atterberg limit tests. In rehabilitation projects, sampling to determine the potential of full depth reclamation or the potential for rubbilization of asphalt pavements is somewhat different, requiring the sampling of the in-place base, subbase, and surface pavement to determine its suitability for reuse and/or rubblizing. The condition survey, as discussed in section 4.2.3, will help in identifying areas requiring sampling and the types of samples required. In general, sampling of the subgrade is not as intensive as is needed for new pavements. Detailed sampling of the base, subbase, and surface pavement will be required to determine if there is a large amount of variability in materials along the project and the condition of those materials for reuse (*e.g.,* base and subbase that has been contaminated with large quantities of fines would not be desirable).

Sampling at each boring location may be either continuous or intermittent. In the former case, samples are obtained throughout the entire length of the hole; in the latter (primarily used in areas of deep cuts), samples are taken about every 1.5 m (5 ft) and at every change in material. Initially, it is preferable to have a few holes with continuous sampling so that all major soil strata present can be identified. Every attempt should be made to obtain 100 percent recovery where conditions warrant. The horizontal and vertical extent of these strata can then be established by intermittent sampling in later borings, if needed.

To obtain a basic knowledge of the engineering properties of the soils that will have an effect on the design, undisturbed samples (such as those obtained with thin-wall samplers or double tube core barrel rock samplers) should be taken, if possible. The actual number taken should be sufficient to obtain information on the shear strength, consolidation characteristics, and resilient modulus of each major soil stratum. Undisturbed samples should comply with the following criteria:

1. The samples should contain no visible distortion of strata, or opening or softening of materials.
2. Specific recovery ratio (length of undisturbed sample recovered divided by length of sampling push) should exceed 95 percent.
3. The samples should be taken with a sampler with an area ratio (cross sectional area of sampling tube divided by full area or outside diameter of sampler) less than 15 percent.

At least one representative undisturbed sample should be obtained in cohesive soil strata, in each boring for each 1.5-m (5-ft) depth interval, or just below the planned surface elevation of the subgrade. Recommended procedures for obtaining undisturbed samples are described in AASHTO Standard T207, *Thin-Walled Tube Sampling of Soils*. If undisturbed samples cannot be recovered, disturbed samples should be taken.

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

***Rock Sampling***

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

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

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

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

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

***Groundwater***

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

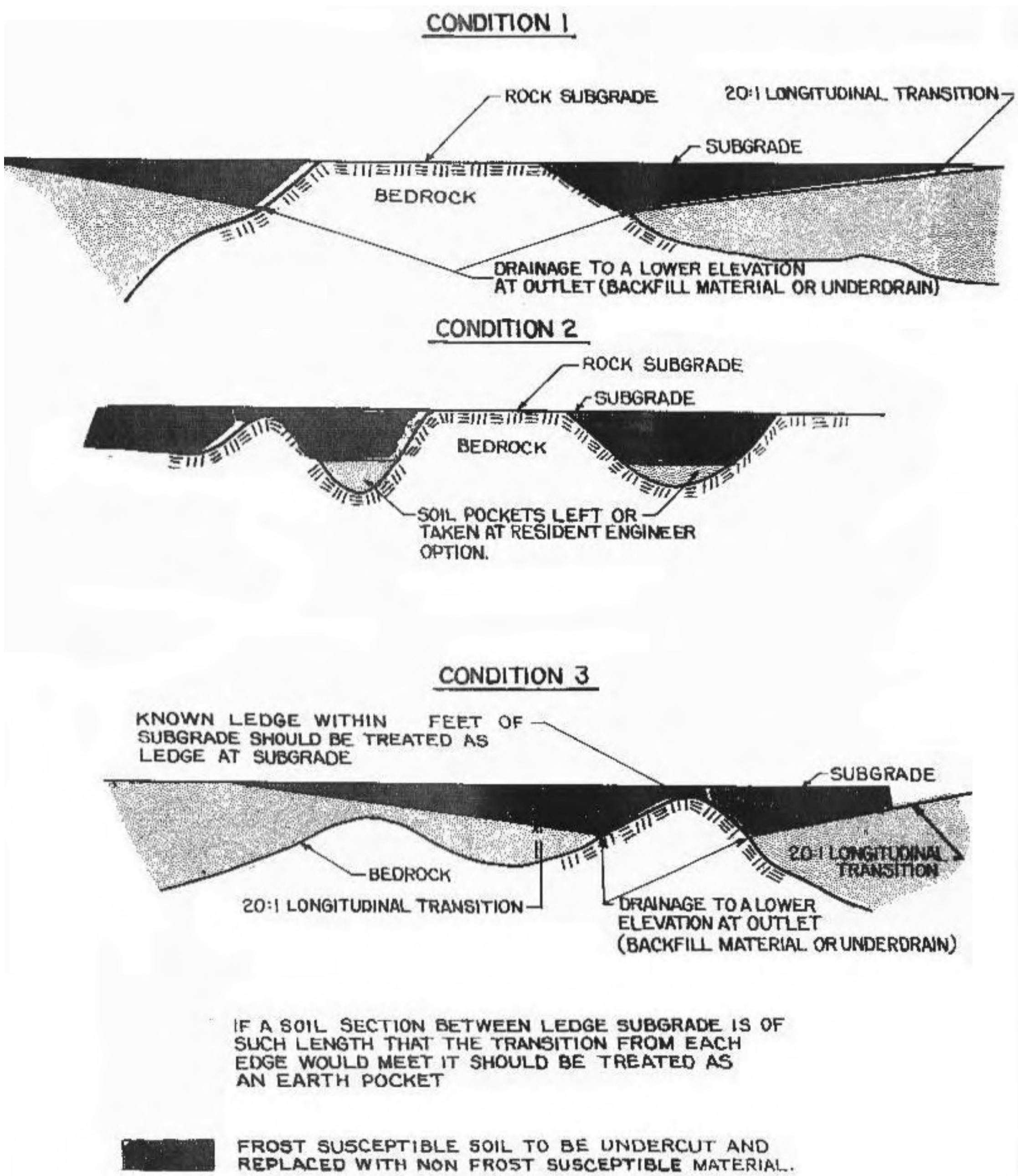



Figure 4-14. Excavation requirements for frost susceptible soils over undulating rock.

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

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

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

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

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

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

***Test Pits***

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

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

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

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

***Sampling for Fill/Borrow Materials***

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

***Standards and Guidelines***

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

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

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

### 4.5.7 Guidelines for Idealized Subsurface Exploration Program

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

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

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

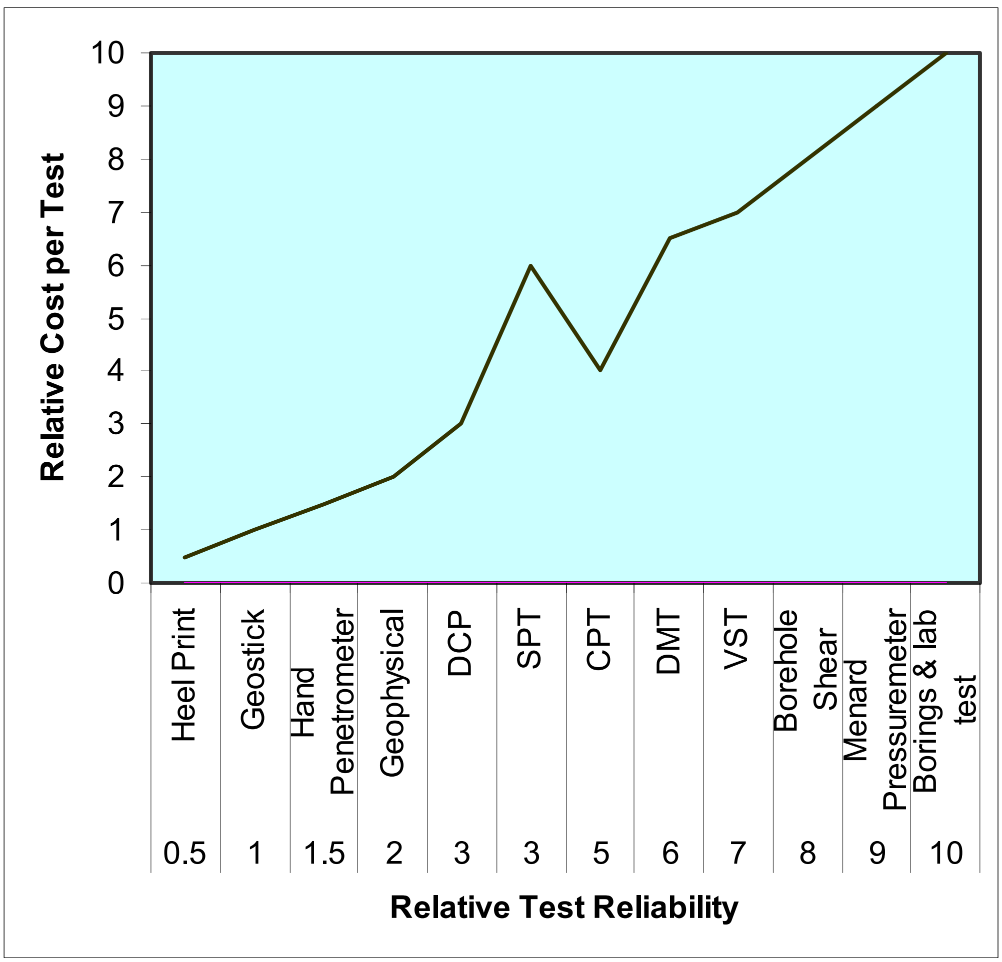
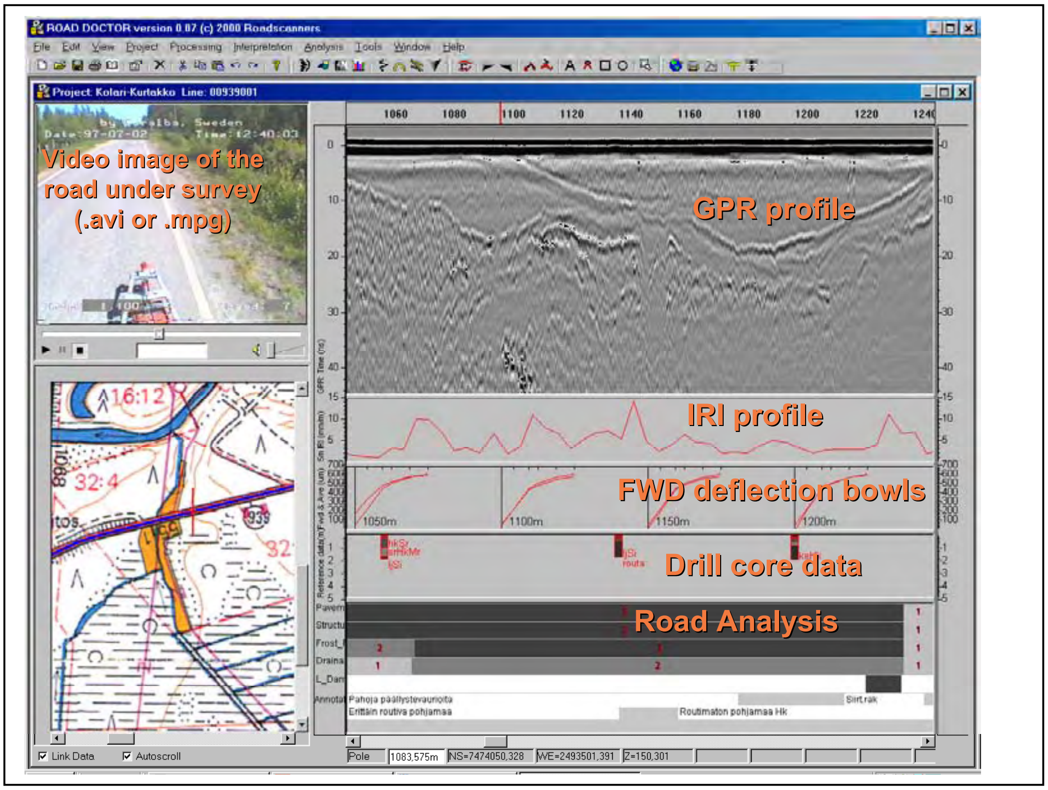
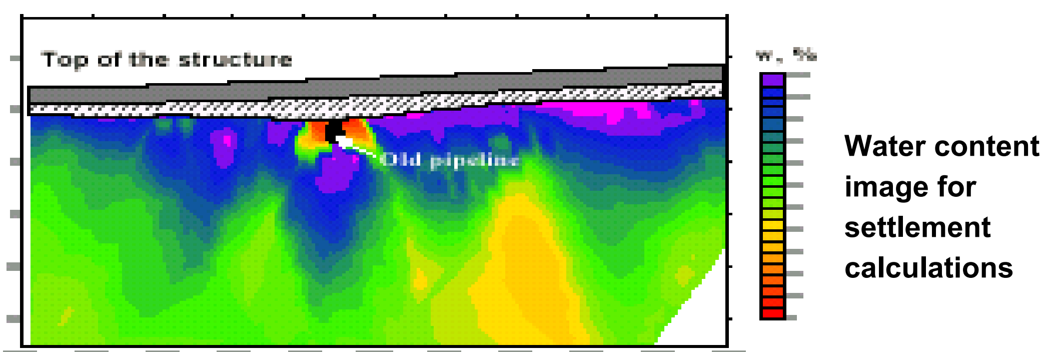



Figure 4-15. Qualitative relationship between relative subsurface exploration cost and reliability (after Handy, 1980).




**a)**




**b)**

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

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

## 4.6 IDENTIFY SOURCE FOR OTHER GEOTECHNICAL COMPONENTS

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

## 4.7 SUBGRADE CHACTERIZATION

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

### 4.7.1 Boring Logs

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

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

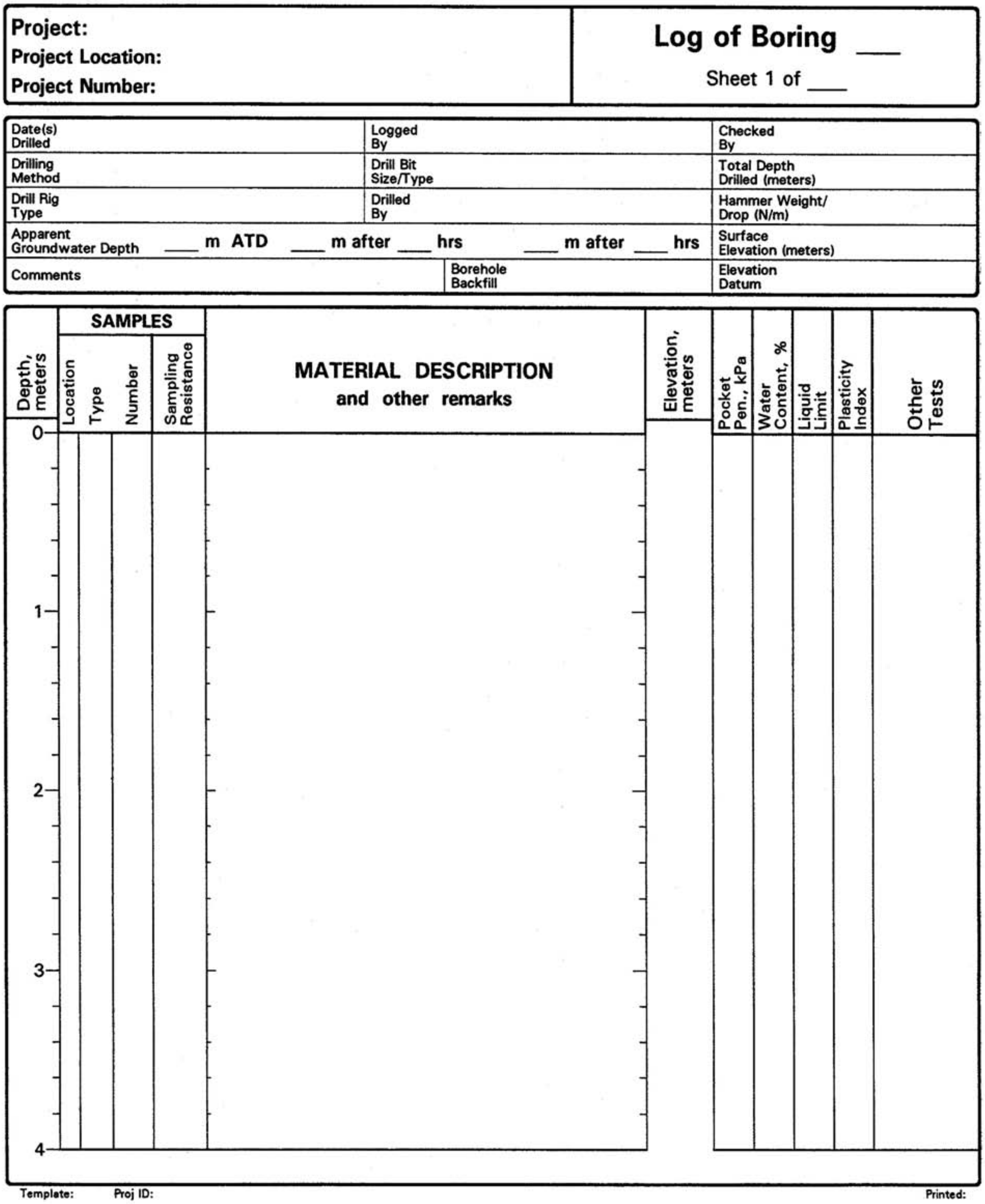
- Topographic survey data, including boring location and surface elevation, and bench mark location and datum, if available.
- An accurate record of any deviation in the planned boring locations.
- Identification of the subsoils and bedrock, including density, consistency, color, moisture, structure,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

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

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

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

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



Project:
Project Location:
Project Number:

Log of Boring ___
Sheet 1 of ___

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

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

Template: Proj ID: Printed:

Figure 4-17. Subsurface exploration log.

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

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

### 4.7.2 Soil Classification

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

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

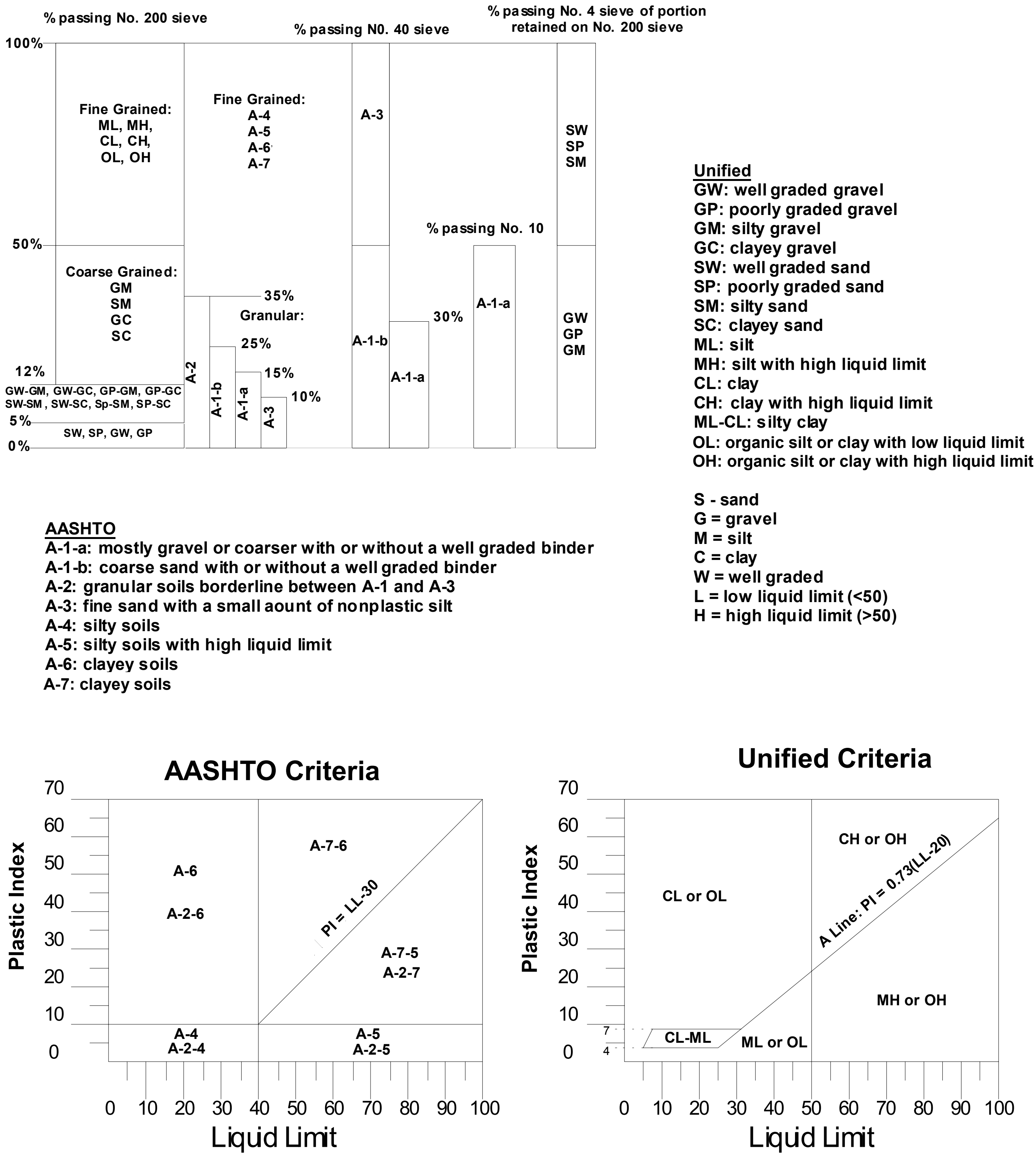



Figure 4-18. The AASHTO and the Unified Soil Classification System (after Utah DOT, 1998).

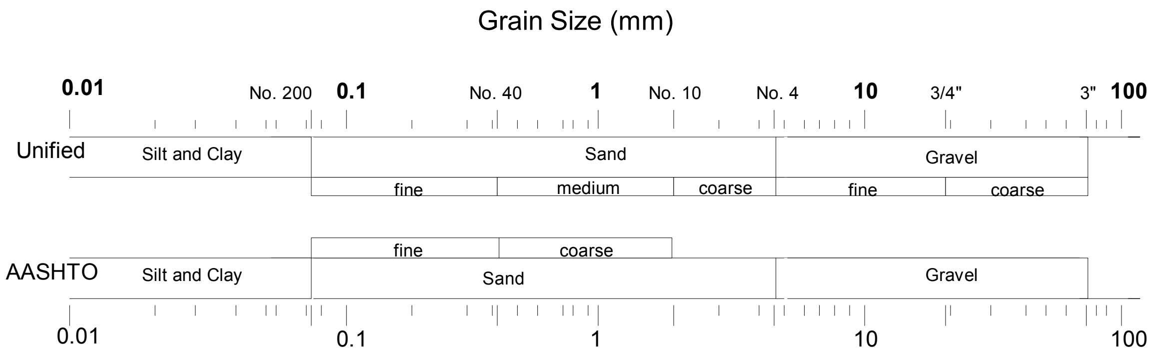



Figure 4–19. Particle size limit by different classifications systems.

**Table 4-13. Classification of soils.**

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

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

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

e.

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

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

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

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

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

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

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

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

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

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

### 4.7.3 Subsurface Profile

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

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

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

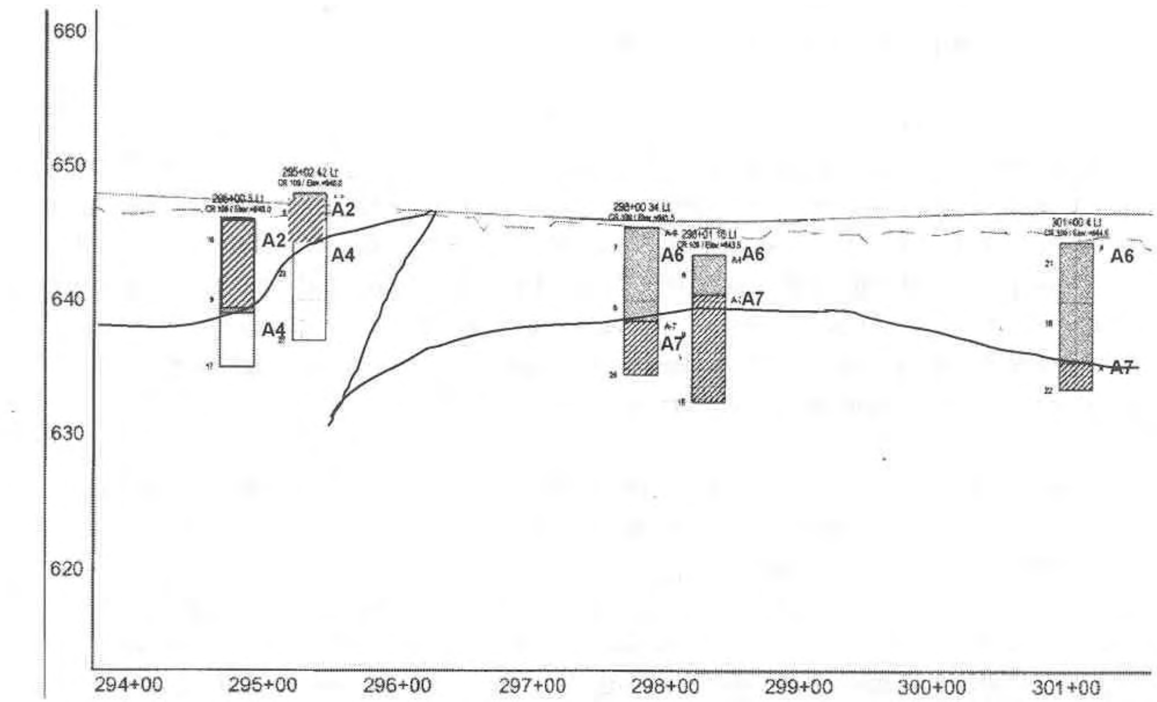



Figure 4-20. Subsurface profile based on boring data showing cross-sectional view.

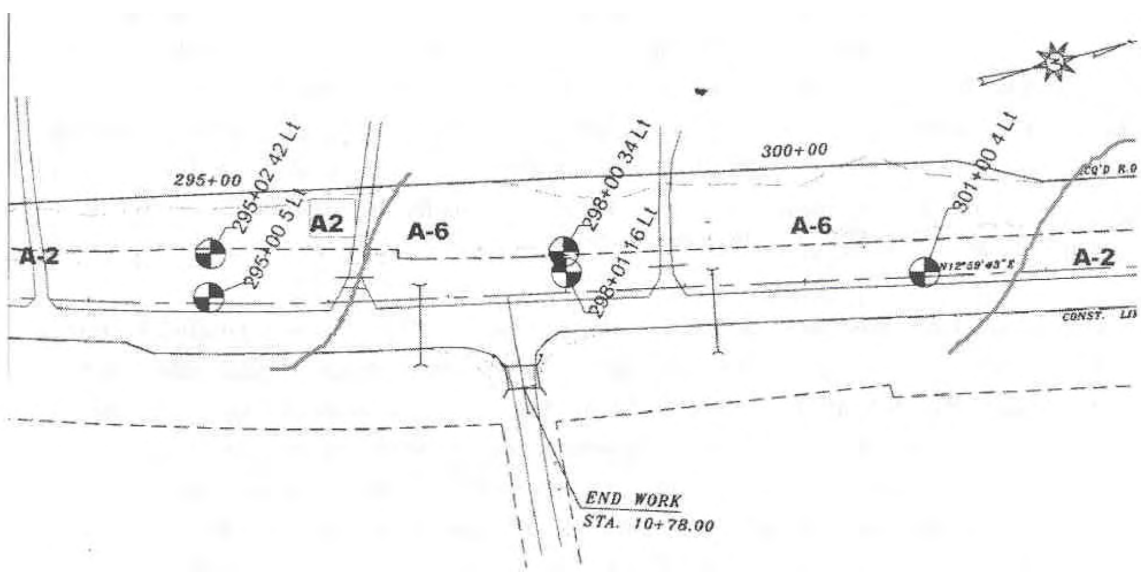



Figure 4-21. Plan view of subsurface information.

### 4.7.4 Select Samples for Laboratory Testing

A program of laboratory tests will be required on representative samples of the foundation soils or soils to be used as construction materials so that pertinent properties can be determined. The extent of the laboratory program depends on the criticality of the design and on the complexity of the soil conditions. Those laboratory tests and analyses that are typically performed or required for an analysis and selection of the pavement type and thickness are listed in Table 4-15. A deep cut or high embankment, as used in the table, general implies greater than a few meters (6 ft or more).

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

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

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

The number of test specimens depends on the number of different soils identified from the borings, as well as the condition of those soils. The availability of geophysical and/or in-situ tests will also affect the number and type of tests. Most of the subgrade test specimens should be taken from as close to the top of the subgrade as possible, extending down to a depth of 0.6 m (2 ft) below the planned subgrade elevation. However, some tests should be performed on the soils encountered at a greater depth, especially if those deeper soils are softer or weaker. No guidelines are provided regarding the number of tests, except that all of the major soil types encountered near the surface should be tested with replicates, if possible. Stated simply, resilient modulus tests or other design tests (*e.g.,* CBR) should be performed on any soil type that may have a detrimental impact on pavement performance (NCHRP 1-37A Pavement Design Guide). Other properties, such as shrink/swell and consolidation, will be required for evaluating stabilization requirements and long-term performance (*e.g.,* potential deformation).

For construction, as was discussed in section 4.5.6, moisture-density tests will be required on each soil type that will be used as fill in the pavement section, as well as the roadway embankment. AASHTO T99 should be used for medium to high plasticity fine-grained soils, whereas AASHTO T180 should be used for coarse-grained and low plasticity fine-grained soils. The degree of compaction required for the in-place density should be expressed as a percentage of the maximum density from the specified test procedure. Design tests (*e.g.,* resilient modulus, CBR, etc.) are also required on the compacted subgrade material.

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

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

**Student Exercise 4-1.**

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

**Student Exercise 4-2.**

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

## 4.8 REFERENCES

AASHTO (1972). *AASHTO Interim Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (1986). *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (1988). *Manual on Subsurface Investigations*, Developed by the Subcommittee on Materials, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (1993). *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (2002). *Standard Specifications for Transportation Materials and Methods of Sampling and Testing: Part II: Tests*, American Association of State Highway and Transportation Officials Sixteenth Edition, Washington, D.C.

Al-Qadi, I.L., Lahouar, S. and Loulizi, A (2003). "Ground Penetrating Radar Evaluation for Flexible Pavement Thickness Estimation," Journal of Transportation Engineering, ASCE, Vol. 129, No.1, Jan./Feb., pp. 93-99.

American Society for Testing & Materials (2003). *ASTM Book of Standards*, *Vol. 4, Section 03*, Road and Paving Materials; Vehicle-Pavement Systems, ASTM International, West Conshohocken, PA.

American Society for Testing & Materials (2000). *ASTM Book of Standards*, *Vol. 4, Section 08 and 09*, Construction Materials: Soils & Rocks, ASTM International, West Conshohocken, PA.

Boudreau, R. L. (2003). "Repeatability of the Resilient Modulus Test Procedure," *ASTM STP 1437, Resilient Modulus Testing for Pavement Components,* G.N. Durham, A.W. Marr, and W.L. DeGroff, Eds., American Society for Testing and Materials, West Conshohocken, PA.

Caterpillar Handbook of Ripping, 8th Edition, Caterpillar Company, Peoria, IL.

Chassie, R.G. (2000) *Soils and Foundations Workshop Manual*, FHWA NHI-00-045, National Highway Institute. Federal Highway Administration, Washington, D.C.

Cheney, R. and Chassie, R.G. (2000). Soils and Foundations Workshop, FHWA NHI-00-045, NHI Course No. 132012, Federal Highway Administration, Washington, D.C.

Dumas, C., Cannon, R.R., Lwin, M.M., Mansukhani, S., McLain, K.W., Pelnik, T.W., Porbaha, A., Putcha, S., Brown, D.A., Short, R.D., Macnab, A., and Christopher, B.R. (2003). FHWA Scan Tour Report on *The Innovative Technology for Accelerated Construction of Bridge and Embankment Foundations*, FHWA International Technology Scanning Program, Federal Highway Administration, Washington, D.C.

Dunnicliff, J. (1988). *Geotechnical Instrumentation for Monitoring Field Performance*, John Wiley & Sons, Inc., New York.

Dunnicliff, J. (1998). *Geotechnical Instrumentation Reference Manual*, FHWA-NHI-98-034, National Highway Institute, Washington D.C.

FHWA-NHI-98-034 (1998) *Geotechnical Instrumentation*, author: Dunnicliff, J., National Highway Institute, 238 p.

FHWA TS-78-209 (1978). *Guidelines for Cone Penetration Test: Performance and Design*, prepared by J.H. Schmertmann for the Federal Highway Administration, Washington, D.C.

FHWA NHI-01-031 (2001) *Subsurface Investigations*, authors: Mayne, P.W., Christopher, B.R., DeJong, J., National Highway Institute, 394 p.

Greenhouse, J.P., Slaine, D.D., and Gudjurgis, P. (1998). Application of Geophysics in Environmental Investigations, Matrix Multimedia Publishing, Toronto.

Handy, R.L. (1980). "Realism in Site Exploration: Past, Present, Future, and Then Some – All Inclusive," <u>Proceedings, Symposium on Site Exploration in Soft Ground Using In-Situ Techniques</u>, Report FHWA-TS-80-202, Federal highway Administration, Washington D.C.

Huang, Y.H. (1993). *Pavement Analysis and Design*. Englewood Cliffs, NJ: Prentice Hall.

Kleyn, E.G., "The Use of the Dynamic Cone Penetrometer (DCP)," Transvaal Roads Department, Report Number L2/74, Pretoria, 1975.

Kulhawy, F.H. and Mayne, P.W. (1990). *Manual on Estimating Soil Properties for Foundation for Design*, Prepared by Cornell University for the Electric Power Research Institute, Report Number EPRI EL-6800, Electric Power Research Institute, Palo Alto, CA.

*LTPP Manual for Falling Weight Deflectometer Measurement: Operational Field Guidelines, Version 3.1* (August 2000) *Prepared by:* LAW PCS, A Division of Law Engineering and Environmental Services, Inc., Office of Infrastructure R&D, Long-Term Pavement Performance Team, HRDI-13. Federal Highway Administration, McLean, Virginia.

Mayne, P.W., Christopher, B.R., Dejong, J. (2002). *Subsurface Investigations*, FHWA-NHI-01-031, Federal Highway Administration, Washington, D.C.

Morey, R.M. (1998). *Ground Penetrating Radar for Evaluating Subsurface Conditions for Transportation Facilities, NCHRP Synthesis 278*, Transportation Research Board, Washington, D.C.

Muhammad, L.N., Titi, H.H., and Herath, A. (2002). *Effect of Moisture Content and Dry Unit Weight on the Resilient Modulus of Subgrade Predicted by Cone Penetration Test*, FHWA/LA.00/355, Louisiana Transportation Research Center, Baton Rouge, LA.

NCHRP 1-37A Design Guide (2002). *2002 Design Guide – Design of New and Rehabilitated Pavement Structures*, Draft Final Report, Part 1 – Introduction and Part 2 – Design Inputs, Prepared for the National Cooperative Highway Research Program by ERES Division of ARA.

Newton, T., Anderson, N., and Hatheway, A. (1998). "Application of Innovative Geophysical Methods for Subgrade Investigations in Karst Terrain," Report published by the University of Missouri, Rolla, for Missouri Department of Transportation and the Federal Highway Administration

Newcombe, D.A., and B. Birgisson (1999). "Measuring In-situ Mechanical Properties of Pavement Subgrade Soils," *NCHRP Synthesis 278*, Transportation Research Board, Washington, D.C.

NHI (1998). *Techniques for Pavement Rehabilitation; A Training Course*, Participant's Manual, National Highway Institute course number 13108, Federal Highway Administration, Washington D.C.

Powers, J.P. (1992). *Construction Dewatering*, John Wiley & Sons, Inc., New York.

Roimela, P., Salmenkaita, S., Maijala, P., Saarenketo, T. (2000). "Road Analysis: a Tool for Cost-effective Rehabilitation Measures for Finnish Roads," Presented at GPR 2000, the Eighth International Conference on Ground Penetrating Radar, May 23-26, 2000, Gold Coast, Australia.

Rollings, M.P., and R.S. Rollings (1996). *Geotechnical Materials in Construction*, McGraw-Hill, NY.

Santamarina, J.C., Klein, K. and Fam, M.A. (2001). Soils and Waves, Particulate Materials Behavior Characterization, & Process Monitoring, John Wiley & Sons, Ltd., N.Y., 488 p.

Scullion, T. (2004). Implementation of Ground Penetrating Radar Technology, AASHTO's Technical Implementation Group, Demo CD Distributed at TRB 2004 Annual Meeting, Texas Transportation Institute.

Tolla, P. (2002). Presentation to the FHWA Scan Team on "Innovative Technology for Accelerated Construction of Bridge and Embankment Foundations."

*U.S. Department of Labor Construction Safety and Health Regulations* (current) Standard – 29 CFR, Part 1926: http://www.osha.gov/pls/oshaweb/owares.do_search?p_doc_type=STANDARDS&p_search_type=StandTextPolicy&p_search_str=PART+1926+

Utah D.O.T. (1998). "Pavement Management and Pavement Design Manual."

Vandre, B., Budge, A., Nussbaum, S. (1998). "DCP- A Useful Tool for Characterizing Engineering Properties of Soils at Shallow Depths," Proceedings of 34th Symposium on Engineering Geology and Geotechnical Engineering, Utah State University, Logan, UT.

Webster, S.L., Brown, R.W. and Porter, J.R. (1994). Force Projection Site Evaluation Using the Electric Cone Penotrometer (ECP) and the Dynamic Cone Penetrometer (DCP), Technical Report GL-94-17, U.S. Air Force, Tyndall AFB, FL.

Wightman, W.E., Jalinoos, F., Sirles, P., Hanna, K. (2003). *Application of Geophysical Methods to Highway Related Problems*, Report developed by Blackhawk GeoSciences, A Division Of Blackhawk GeoServices for FHWA under FHWA Contract Number DTFH68-02-P-00083, Federal Highway Administration, Washington D.C.

Wimsatt, A.J. and Scullion, T. (2003). "Selecting Rehabilitation Strategies for Flexible Pavement in Texas," presented and published at the annual meeting of the Transportation Research Board, TRB, Washington D.C.

www.fema.gov

www.ce.gatech.edu/~geosys/misc/links.htm

www.geoforum.com/info/geophysical/.

http://soils.usda.gov/survey/online_surveys/

www.usgs.gov

# CHAPTER 5.0 GEOTECHNICAL INPUTS FOR PAVEMENT DESIGN

## 5.1 INTRODUCTION

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

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

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

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

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

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

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

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

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

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

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

5.6 ENVIRONMENT/CLIMATE INPUTS

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

## 5.2 REQUIRED GEOTECHNICAL INPUTS

### 5.2.1 1993 AASHTO Design Guide

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

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

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

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

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

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

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

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

### 5.2.2 NCHRP 1-37A Design Guide

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

***Hierarchical Approach to Design Inputs***

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

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

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

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

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

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

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

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

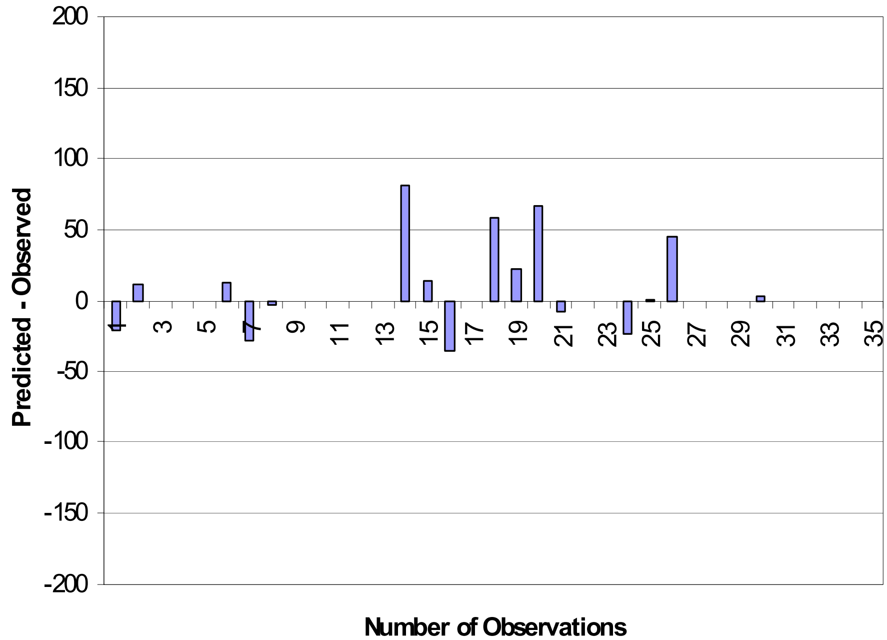



Figure 5-1. Thermal crack prediction from NCHRP 1-37A Design Guide using Level 1 material inputs.

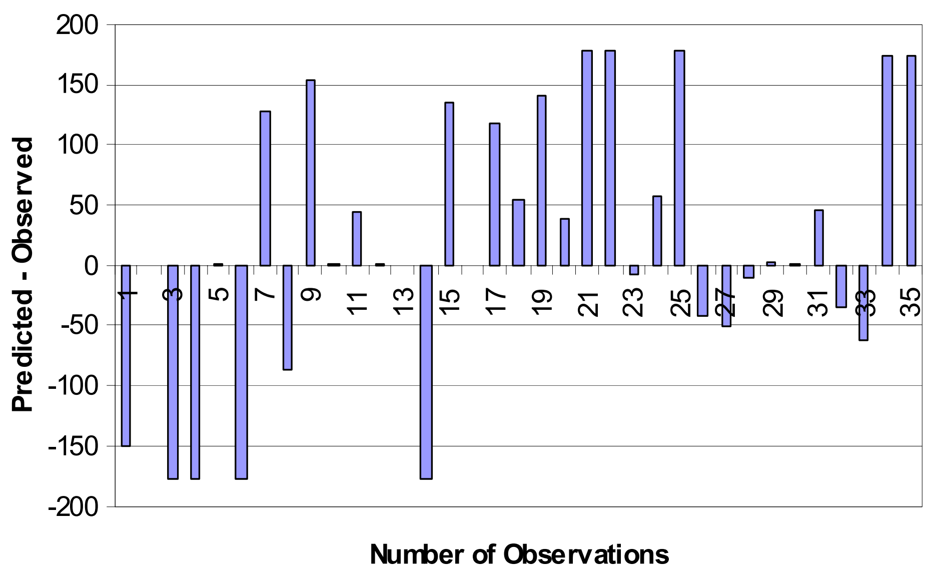



Figure 5-2. Thermal crack prediction from NCHRP 1-37A Design Guide using Level 3 material inputs.

***Required Geotechnical Inputs***

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

- *Mechanical* properties that are used in an analysis model to relate applied structural loads to structural response (Table 5-3 and Table 5-4).
- *Thermo-hydraulic* inputs that are used to relate environmental influences to the thermal and hydraulic state of the system (Table 5-5).
- *Distress model* properties that enter directly in the empirical models for pavement performance (Table 5-6).

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

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

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

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

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

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

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

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

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

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

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

### 5.2.3 Other Geotechnical Properties

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

## 5.3 PHYSICAL PROPERTIES

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

### 5.3.1 Weight-Volume Relationships

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

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

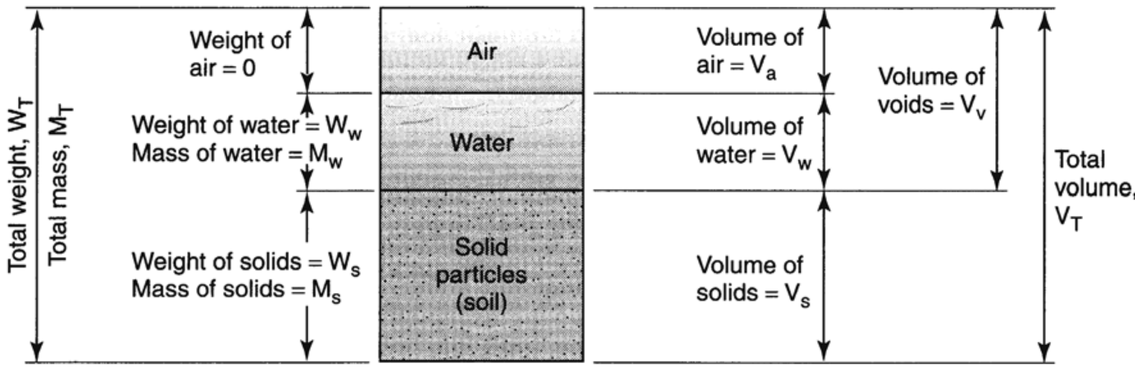



Figure 5-3. Relationships between volume and weight/mass of bulk soil (McCarthy, 2002).

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

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

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

**Table 5-8. Unit weight-volume relationships.**

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

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

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

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

### 5.3.2 Physical Property Determination

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

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

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

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

- Plasticity
  - Atterberg limits (Table 5-21)

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

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

***Volumetric Properties***

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

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

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

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

Typical Values (Coduto ,1999):

| Soil Type | $G_S$ |
|---|---|
| Clean, light colored sand (quartz, feldspar) | 2.65 |
| Dark colored sand | 2.72 |
| Sand-silt-clay mixtures | 2.72 |
| Clay | 2.65 |

**Table 5-11. Moisture content.**

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

**Table 5-12. Unit weight.**

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

**Table 5-13. Compaction characteristics.**

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

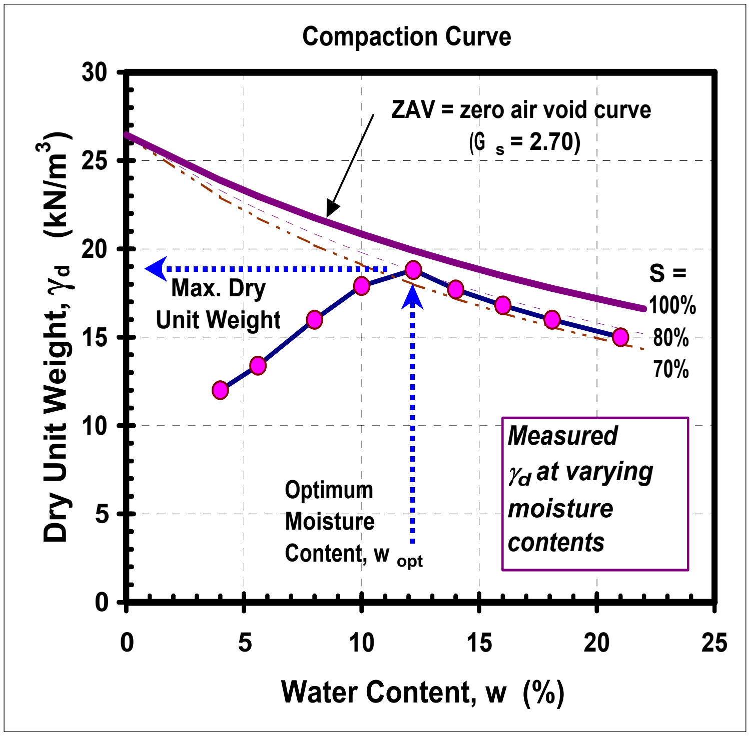



Figure 5-4. Typical moisture-density relationship from a standard compaction test.

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

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

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

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

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

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

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

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

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

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

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

***Gradation***

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

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

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

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

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

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

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

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

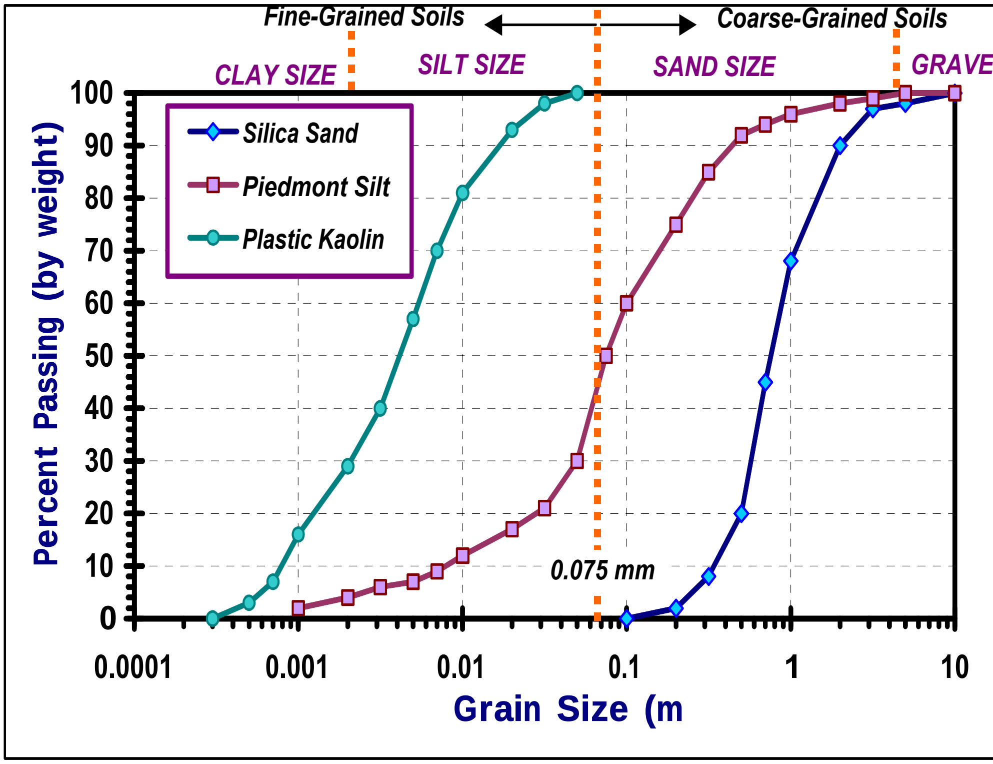



Figure 5-7. Representative grain size distributions for several soil types.

***Plasticity***

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

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

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

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

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

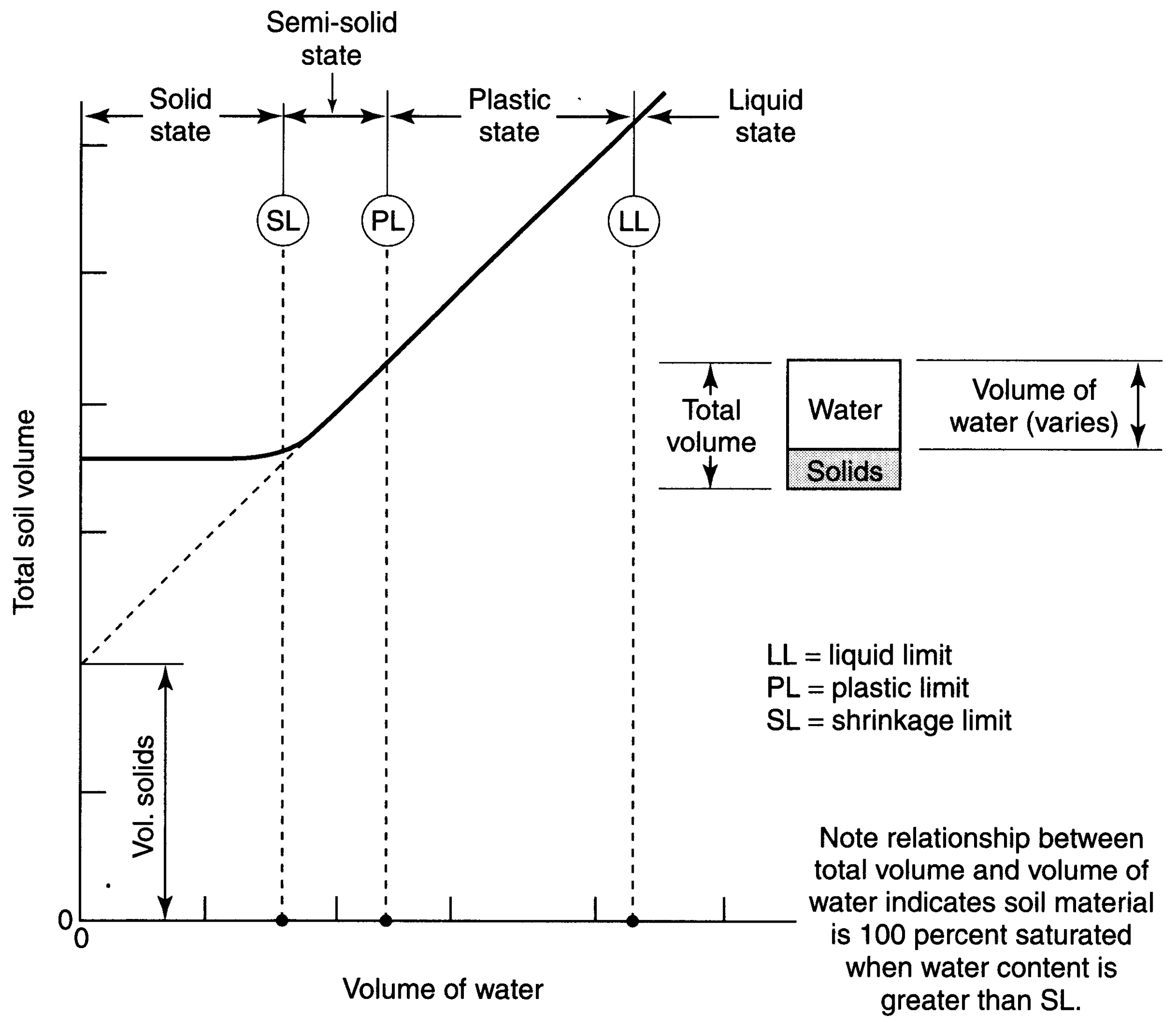



Figure 5-8. Variation of total soil volume and consistency with change in water content for a fine-grained soil (from McCarthy, 2002).

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

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

Figure 5-9. Liquid limit test device.

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

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

### 5.3.3 Problem Soil Identification

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

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

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

**Table 5-23. Swell potential of clays.**

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

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

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

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

**Table 5-25. Collapse potential of soils.**

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

### 5.3.4 Other Aggregate Tests

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

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

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

## 5.4 MECHANICAL PROPERTIES

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

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

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

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

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

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

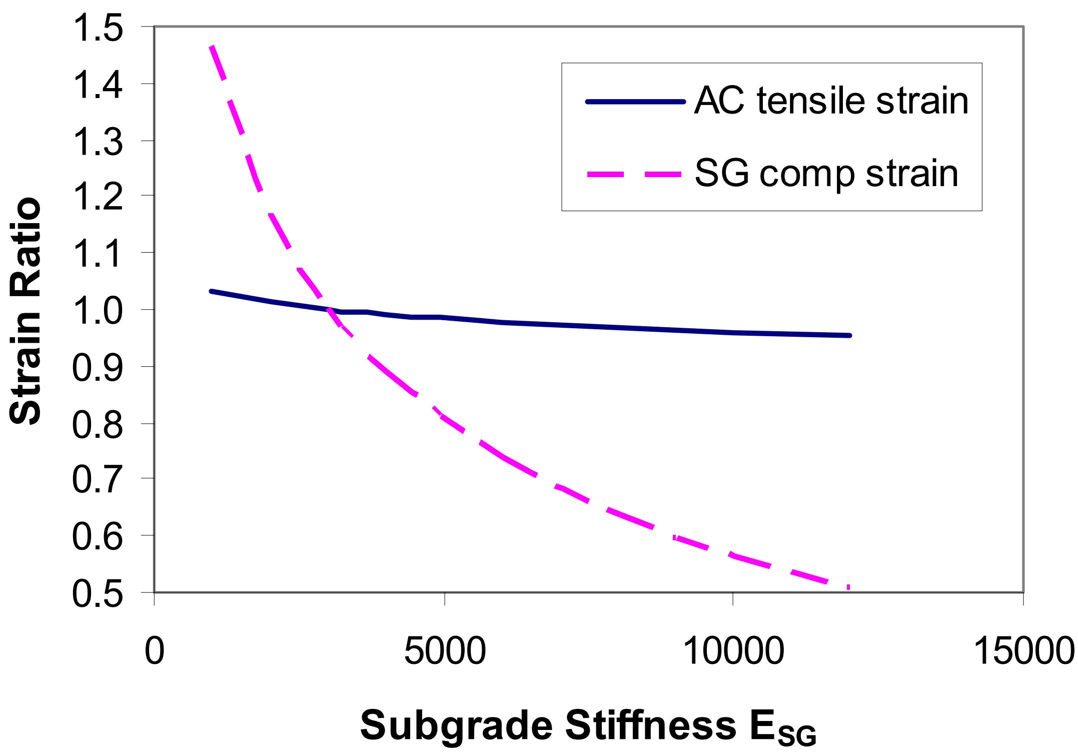



Figure 5-10. Influence of subgrade stiffness on critical pavement strains. (Elastic solution, 6 in./150 mm AC over 18 in./450 mm granular base. Reference elastic moduli: $E_{AC}$ = 500,000 psi/3450 MPa; $E_{BS}$ = 30,000 psi/207 Mpa; $E_{SG}$ = 3000 psi/20.7 MPa. Load: 10 kip/44.5 kN single-wheel load,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

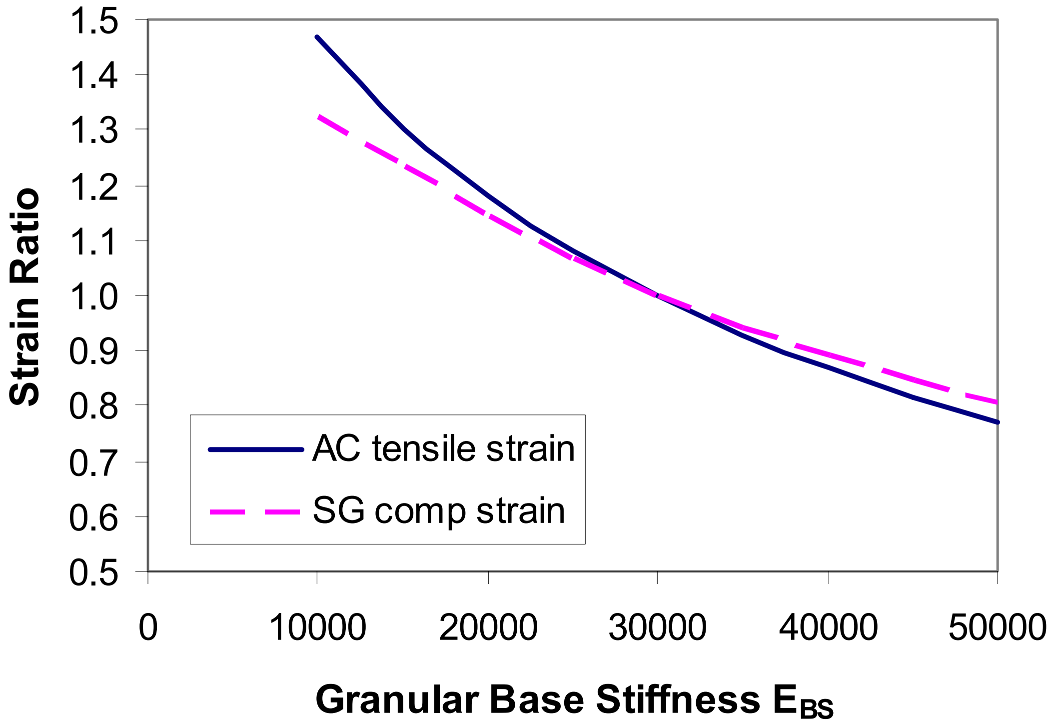



Figure 5-11. Influence of granular base stiffness on critical pavement strains. (Elastic solution, 6 in./150 mm AC over 18 in./450 mm granular base. Reference elastic moduli: EAC = 500,000 psi/3450 MPa; EBS = 30,000 psi/207 Mpa; ESG = 3000 psi/20.7 MPa. Load: 10 kip/44.5 kN single wheel load, 100 psi/690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

### 5.4.1 California Bearing Ratio (CBR)

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

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

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

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


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

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

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

### 5.4.2 Stabilometer (R-Value)

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

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

in which $R$ = resistance value

$P_v$ = applied vertical pressure (160 psi)

$P_h$ = transmitted horizontal pressure

$D_2$ = displacement of stabilometer fluid necessary to increase horizontal pressure from 5 to 100 psi, measured in revolutions of a calibrated pump handle

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

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

**Table 5-29. Stabilometer or R-Value.**

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

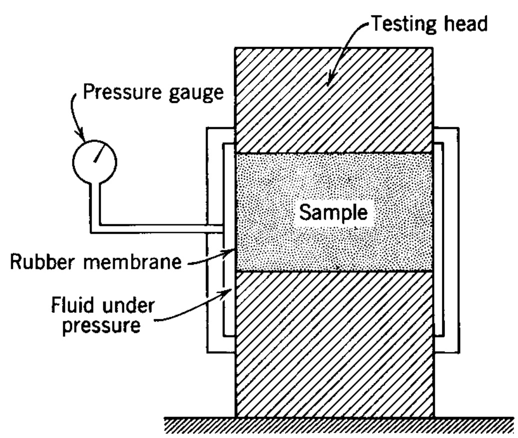



Figure 5-13. Schematic of stabilometer test setup for measuring R-value (Yoder and Witczak, 1975).

### 5.4.3 Elastic (Resilient) Modulus

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

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

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

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

***Unbound Materials***

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

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

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

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

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

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

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

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

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

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

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

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

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

in which

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

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

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

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

in which

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

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

$p_a$ = atmospheric pressure (to make equation dimensionless)

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

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

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

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

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

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

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

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

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

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

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

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

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

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

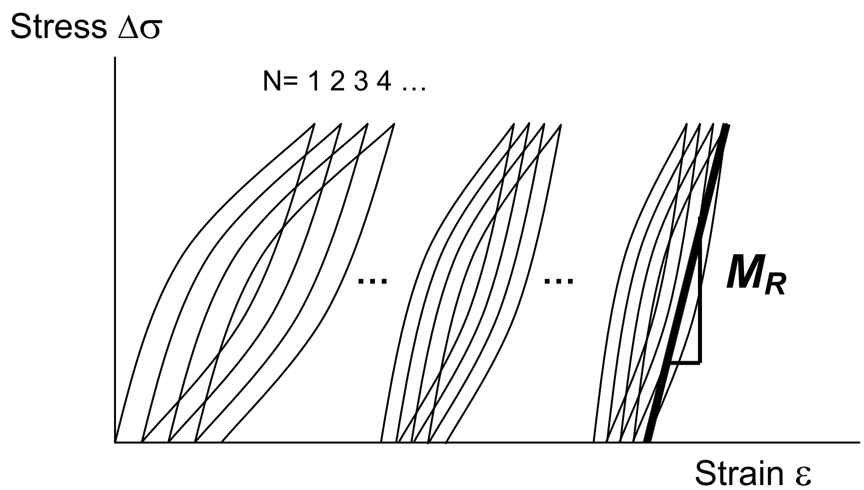


Figure 5-14. Resilient modulus under cyclic loading.

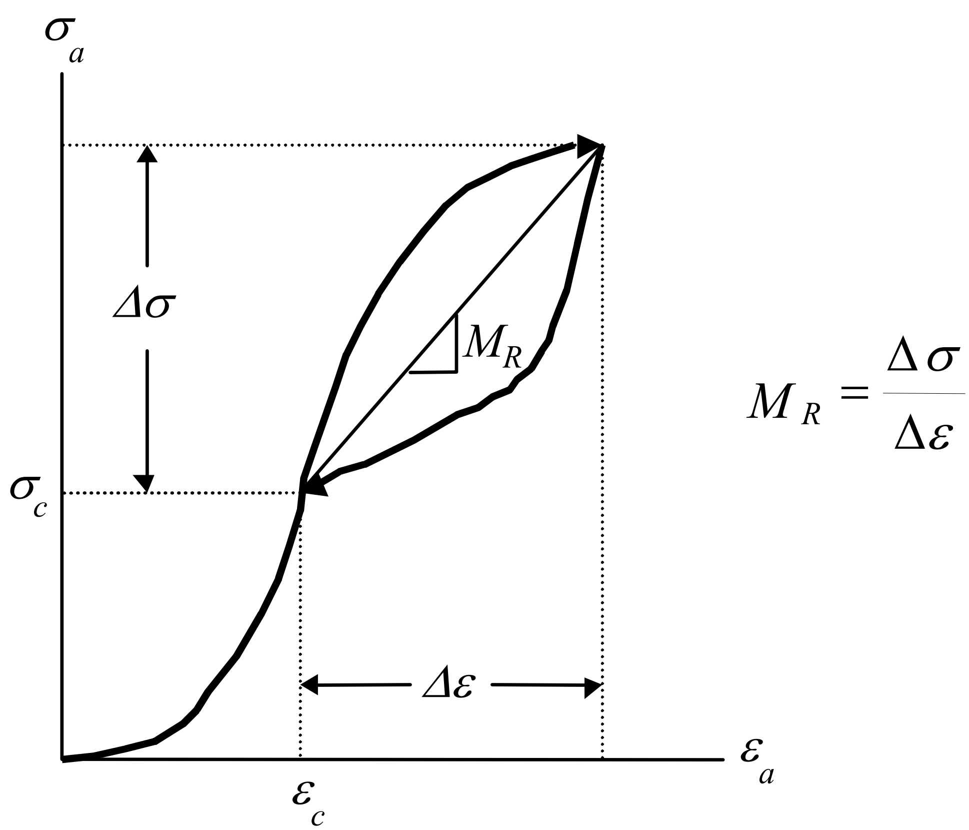


Figure 5-15. Definition of resilient modulus $M_R$ for cyclic triaxial loading.

**Table 5-31. Resilient modulus ($M_R$).**

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

| | |
|---|---|
| | subgrade soils that field $M_R$ values be multiplied by a factor of up to 0.33 for flexible pavements and up to 0.25 for rigid pavements to adjust to design $M_R$ values. NCHRP 1-37A recommends adjustment factors of 0.40 for subgrade soils and 0.67 for granular bases and subbases under flexible pavements. More detailed guidance for adjusting backcalculated modulus values to design $M_R$ values is given in Table 5-32.<br>The 1993 AASHTO Guide includes procedures for incorporating seasonal variations into an effective $M_R$ for the subgrade. Seasonal variations of material properties are included directly in the NCHRP 1-37A M-E design methodology.<br>The Level 1, 2, and 3 $M_R$ inputs in the NCHRP 1-37A design methodology are functions of pavement and construction type, as summarized in Table 5-33. |
| Typical Values | Correlations between $M_R$ and other soil properties include the following: |

*AASHTO 1993 Guide*

- Granular base and subbase layers:

| $\theta$ (psi) | $M_R$ (psi) |
|---|---|
| 100 | 100 740 × *CBR*<br>1000 + 780 × *R* |
| 30 | 440 × *CBR*<br>1000 + 450 × *R* |
| 20 | 340 × *CBR*<br>1000 + 350 × *R* |
| 10 | 250 × *CBR*<br>1000 + 250 × *R* |

- Subgrade (roadbed) soils

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

(Heukelom and Klomp, 1962)

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

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

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

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

*NCHRP 1-37A (Level 2 Inputs)*

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

*NCHRP 1-37A (Level 3 Inputs)*

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

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

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

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

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

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

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

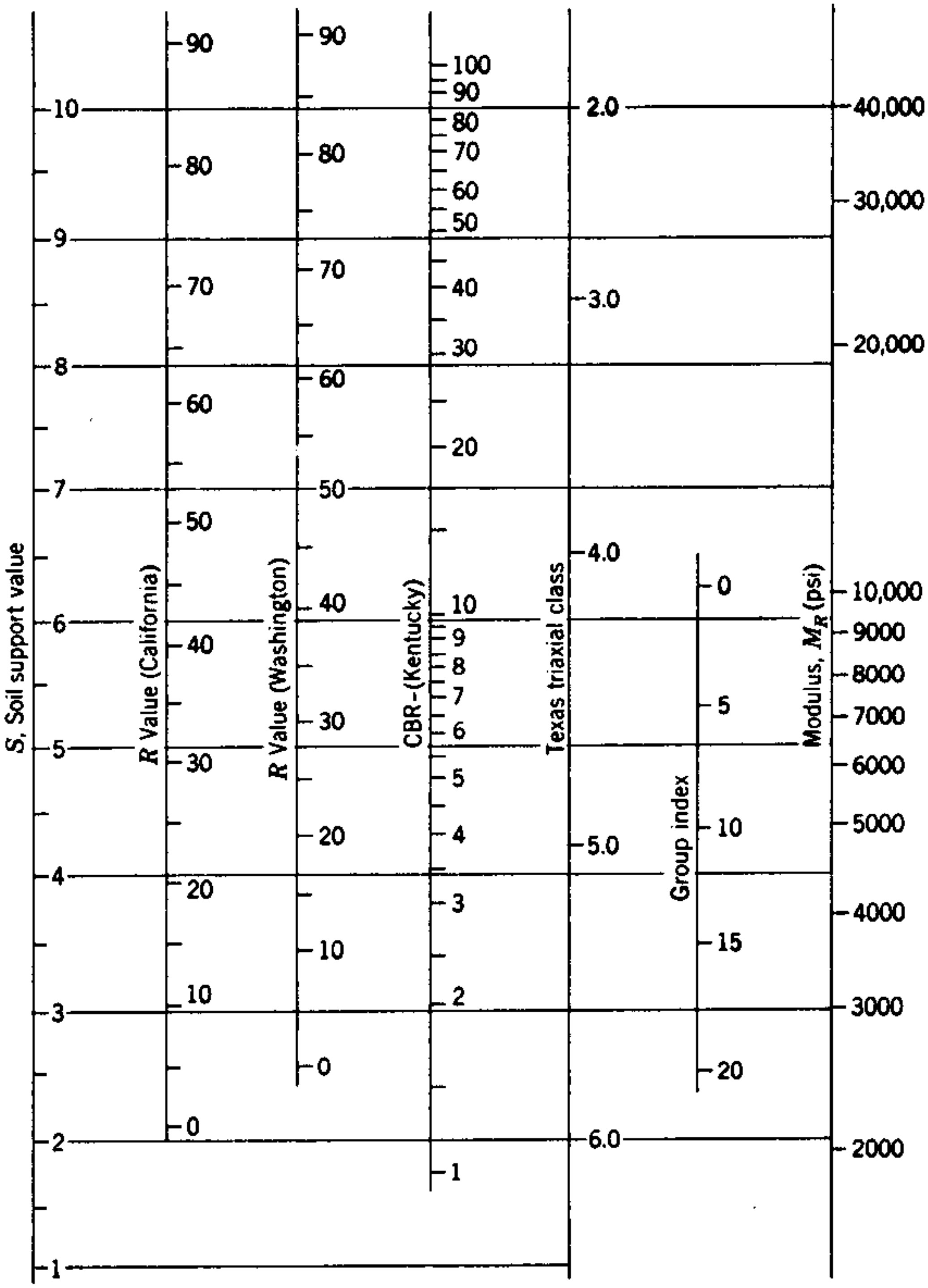



Figure 5-17. Correlations between subgrade resilient modulus and other soil properties (1 psi = 6.9 kPa; from Huang, 1993, after Van Til *et al.*, 1972).

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

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

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

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

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

*Multiply by 0.069 to convert to MPa.

***Fractured PCC Slabs***

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

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

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

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

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

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

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

***Bedrock***

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

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

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

### 5.4.4 Poisson's Ratio

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

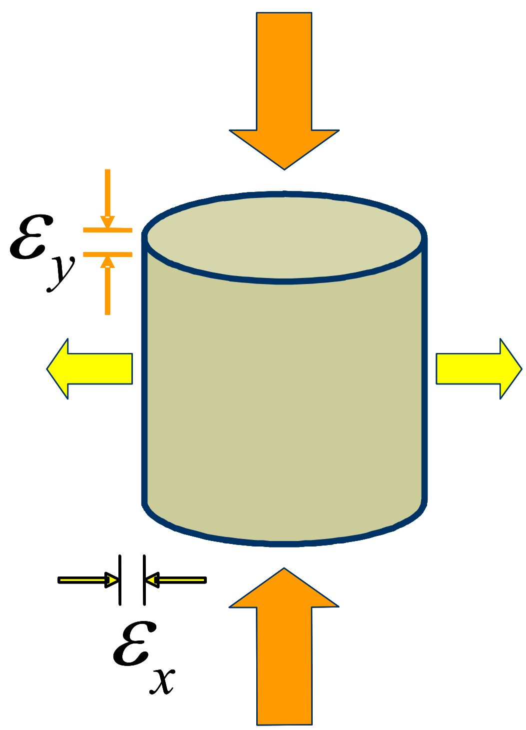



Figure 5-18. Illustration of Poisson's ratio.

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

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

### 5.4.5 Structural Layer Coefficients

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

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

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

***New Construction/Reconstruction***

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

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

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

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

in which

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

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

$k_1$ = 3000 to 8000 psi
$k_2$ = 0.5 to 0.7

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

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

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

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

*Range in $k_1$ and $k_2$ is a function of the material quality

**1 psi = 6.9 kPa

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

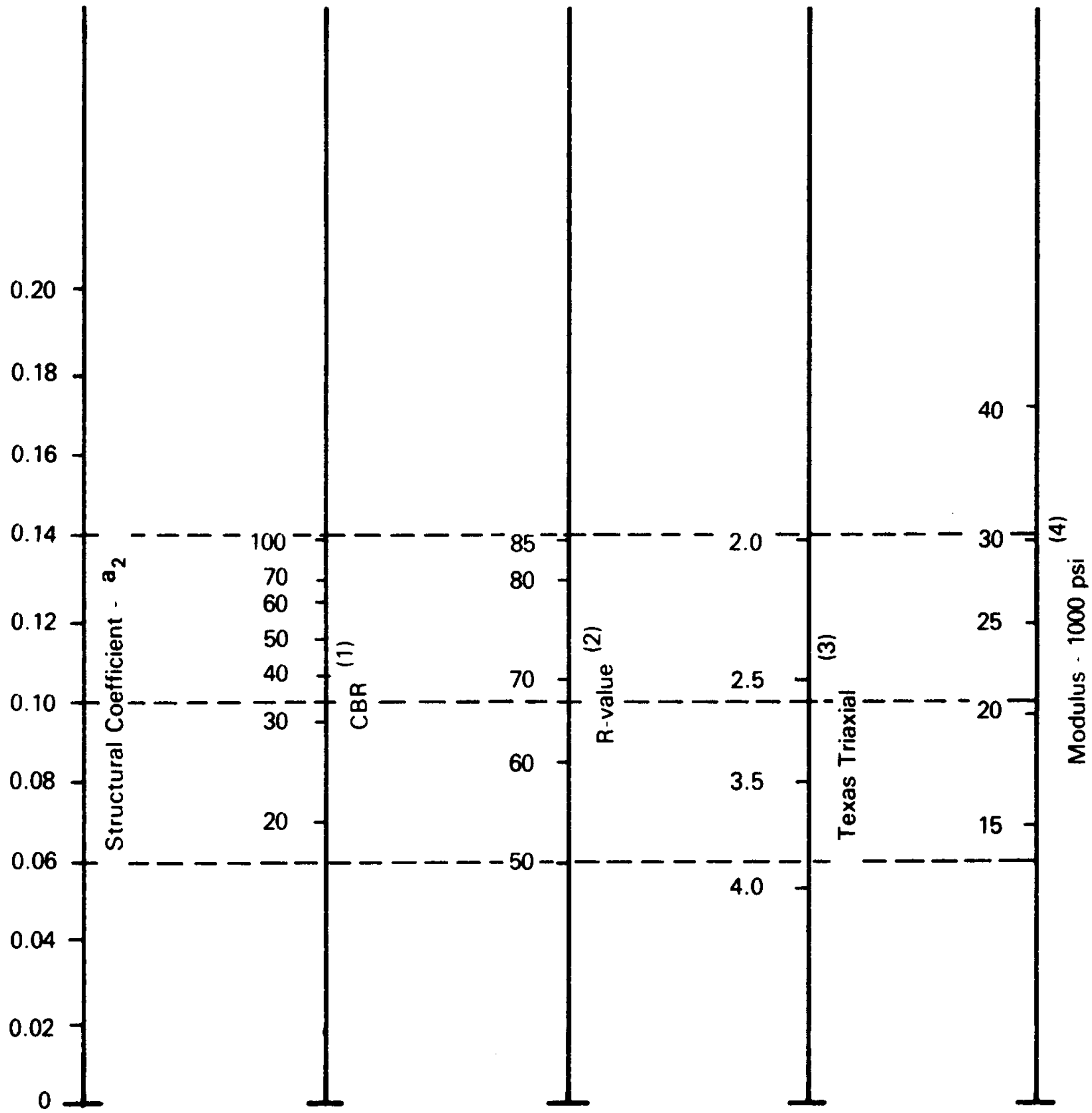
| Moisture State | Equation | Stress State (psi*) | | | |
|---|---|---|---|---|---|
| | | $\theta = 5$ | $\theta = 10$ | $\theta = 20$ | $\theta = 30$ |
| Dry | $8{,}000\theta^{0.6}$ | 21,012 | 31,848 | 48,273 | 61,569 |
| Damp | $4{,}000\theta^{0.6}$ | 10,506 | 15,924 | 24,136 | 30,784 |
| Wet | $3{,}200\theta^{0.6}$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

*1 psi = 6.9 kPa

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

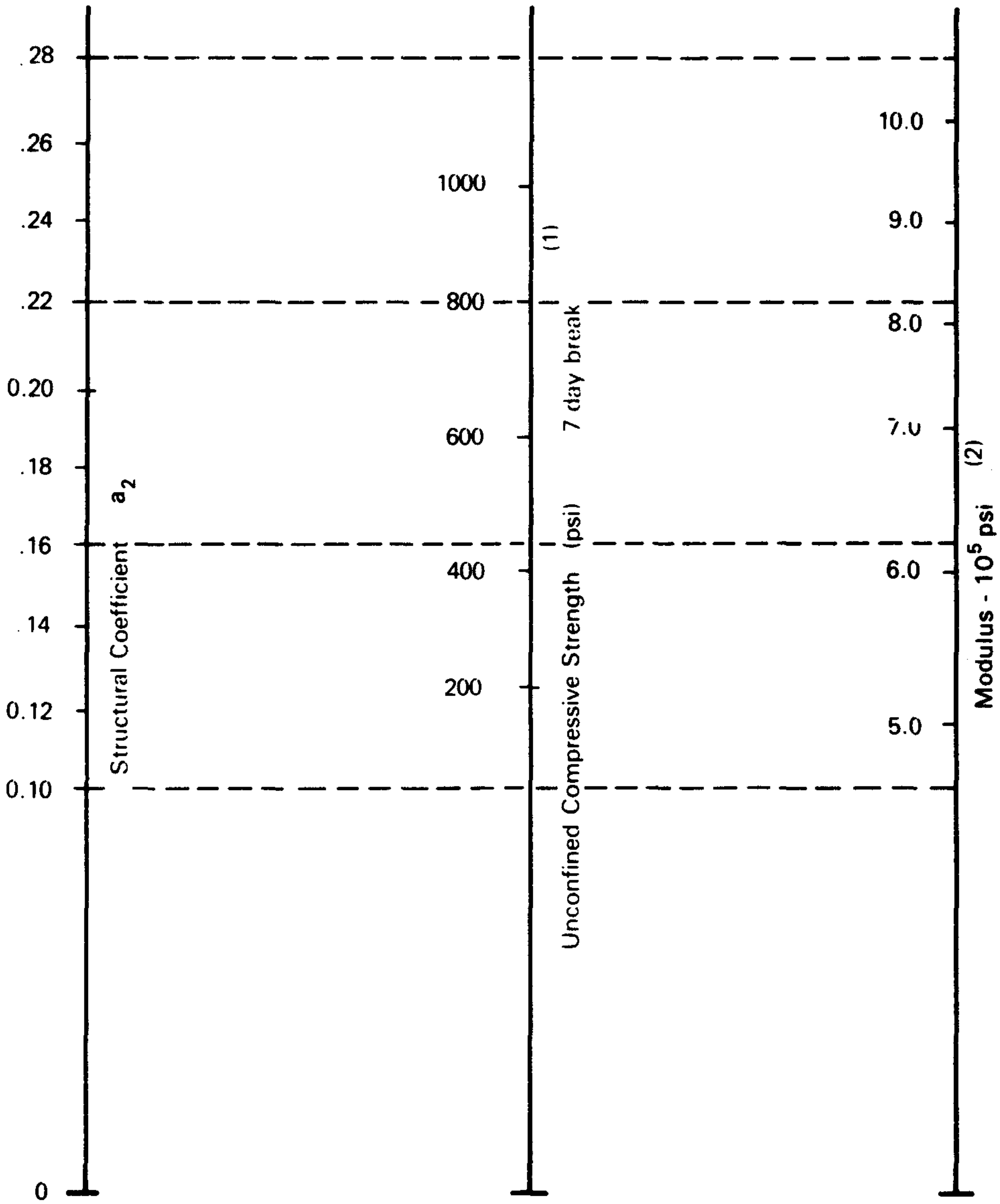
| Asphalt Concrete Thickness (inches*) | Roadbed Soil Resilient Modulus (psi*) | | |
|---|---|---|---|
| | 3,000 | 7,500 | 15,000 |
| < 2 | 20 | 25 | 30 |
| 2 – 4 | 10 | 15 | 20 |
| 4 – 6 | 5 | 10 | 15 |
| > 6 | 5 | 5 | 5 |

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




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

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




(1) Scale derived by averaging correlations from Illinois, Louisiana and Texas.

(2) Scale derived on NCHRP project (3).

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

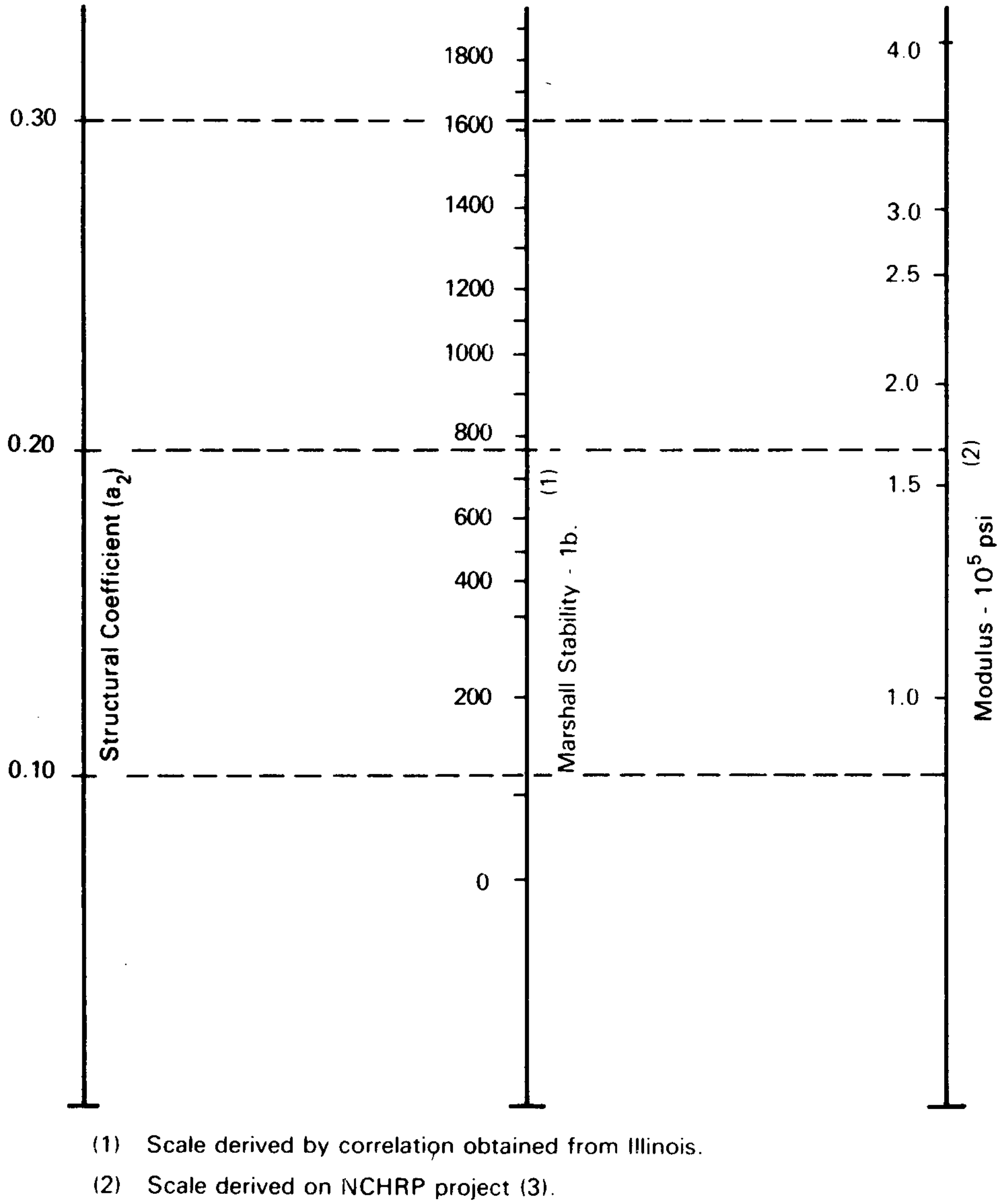



Figure 5-21. Correlations between structural layer coefficient $a_2$ and various strength and stiffness parameters for bituminous-treated granular bases (AASHTO, 1993).

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

$$a_3 = 0.227\log_{10} E_{SB} - 0.839 \tag{5.18}$$

The resilient modulus $E_{SB}$ for granular subbase layers is influenced by stress state in a manner similar to that for the base layer, as given in Eq. (5.17). Typical values for the $k_1$ and $k_2$ material properties for granular subbases are

$k_1$ = 1500 to 6000

$k_2$ = 0.4 to 0.6

The values of $E_{SB}$ from subbase layers in the original AASHO Road Test are summarized in Table 5-42. Note that the $E_{SB}$ values are not only functions of moisture, but also of stress state $\theta$, which in turn is a function of the pavement structure – *i.e.,* thickness of the asphalt concrete surface layer. Typical values of $\theta$ recommended in the 1993 AASHTO Guide for use in subbase design are summarized in Table 5-43. Figure 5-22 summarizes relationships between the $a_3$ structural layer coefficient for granular subbase layers and corresponding values of CBR, R-Value, Texas Triaxial strength, and resilient modulus.

**Table 5-42. Granular subbase resilient modulus $E_{SB}$ values (psi) from AASHO Road Test (AASHTO, 1993).**

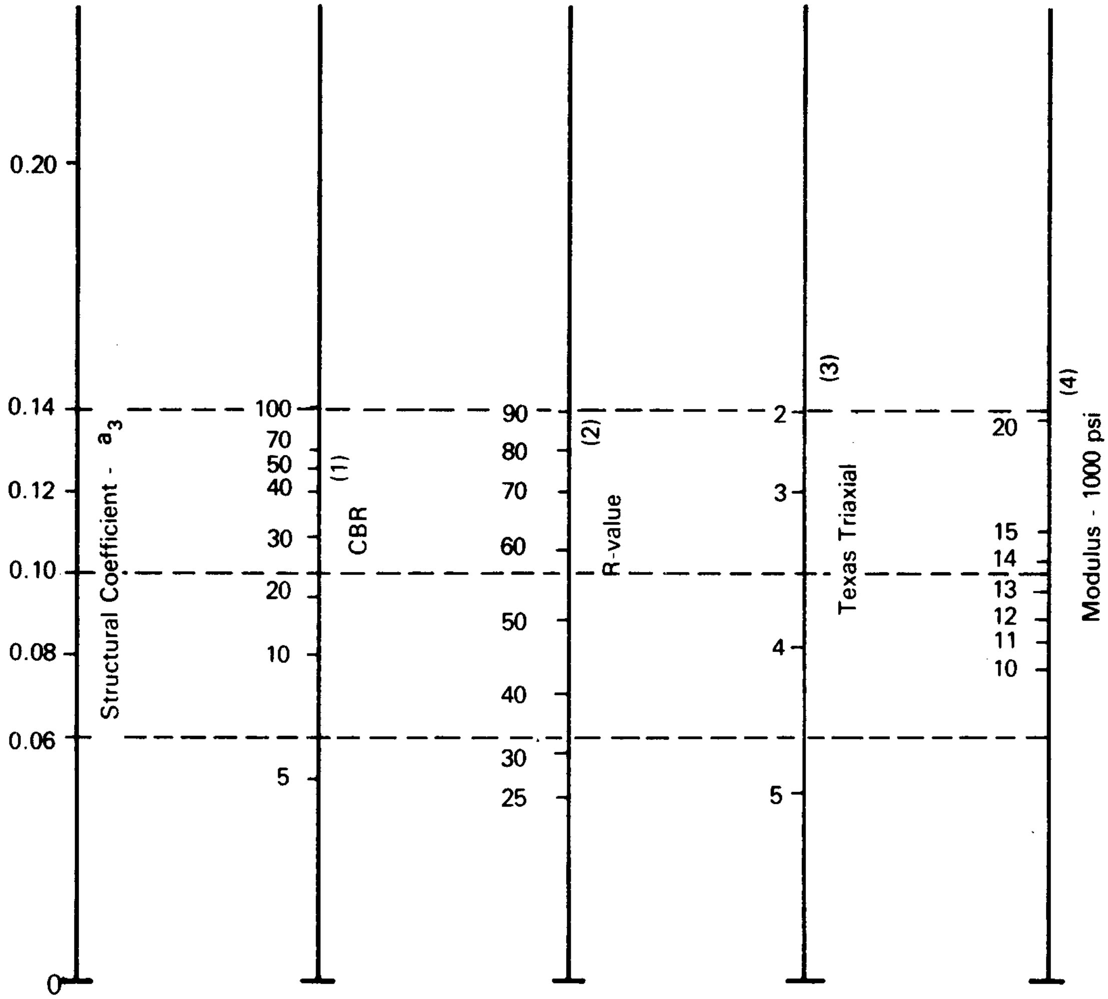
| Moisture State | Equation | Stress State (psi*) | | |
|---|---|---|---|---|
| | | $\theta = 5$ | $\theta = 7.5$ | $\theta = 10$ |
| Damp | $5{,}400\theta^{0.6}$ | 14,183 | 18,090 | 21,497 |
| Wet | $4{,}600\theta^{0.6}$ | 12,082 | 15,410 | 18,312 |

*1 psi = 6.9 kPa

**Table 5-43. Suggested bulk stress $\theta$ (psi) values for use in design of granular subbase layers (AASHTO, 1993).**

| Asphalt Concrete Thickness (inches*) | Stress State (psi*) |
|---|---|
| < 2 | 10.0 |
| 2 – 4 | 7.5 |
| > 4 | 4.0 |

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




(1) Scale derived from correlations from Illinois.

(2) Scale derived from correlations obtained from The Asphalt Institute, California, New Mexico and Wyoming.

(3) Scale derived from correlations obtained from Texas.

(4) Scale derived on NCHRP project *(3)*.

Figure 5-22. Correlations between structural layer coefficient $a_3$ and various strength and stiffness parameters for unbound granular subbases (AASHTO, 1993).

### *Rehabilitation*

Depending on the types and amounts of deterioration present, the layer coefficient values assigned to materials in in-service existing pavements should in most cases be less than the values that would be assigned to the same materials for new construction. Exceptions to this general rule would include unbound granular materials that show no sign of degradation or contamination.

Limited guidance is available for the selection of layer coefficients for in-service pavement materials. Recommendations from the 1993 AASHTO Pavement Design Guide are provided in Table 5-44. In addition to evidence of pumping noted during a visual condition survey, samples of base and subbase materials should be obtained and examined for evidence of erosion, degradation, and contamination by fines, as well as evaluated for drainability, and layer coefficients should be reduced accordingly. Coring and testing are recommended for evaluation of all materials and are strongly recommended for evaluation of stabilized layers.

**Table 5-44. Suggested layer coefficients for existing flexible pavement layer materials (AASHTO, 1993).**

| Material | Surface Condition | Coefficient |
|---|---|---|
| AC Surface | Little or no alligator cracking and/or only low-severity transverse cracking | 0.35 – 0.40 |
| | <10% low-severity alligator cracking and/or<br><5% medium- and high-severity transverse cracking | 0.25 – 0.35 |
| | >10% low-severity alligator cracking and/or<br><10 percent medium-severity alligator cracking and/or<br>>5-10% medium- and high-severity transverse cracking | 0.20 – 0.30 |
| | >10% medium-severity alligator cracking and/or<br><10% high-severity alligator cracking and/or<br>>10% medium- and high-severity transverse cracking | 0.14 – 0.20 |
| | >10% high-severity alligator cracking and/or<br>>10% high-severity transverse cracking | 0.08 – 0.15 |
| Stabilized Base | Little or no alligator cracking and/or only low-severity transverse cracking | 0.20 – 0.35 |
| | <10% low-severity alligator cracking and/or<br><5% medium- and high-severity transverse cracking | 0.15 – 0.25 |
| | >10% low-severity alligator cracking and/or<br><10% medium-severity alligator cracking and/or<br>>5-10% medium- and high-severity transverse cracking | 0.15 – 0.20 |
| | >10% medium-severity alligator cracking and/or<br><10% high-severity alligator cracking and/or<br>>10% medium- and high-severity transverse cracking | 0.10 – 0.20 |
| | >10% high-severity alligator cracking and/or<br>>10% high-severity transverse cracking | 0.08 – 0.15 |
| Granular Base/Subbase | No evidence of pumping, degradation, or contamination by fines | 0.10 – 0.14 |
| | Some evidence of pumping, degradation, or contamination by fines | 0.00 – 0.10 |

### 5.4.6 Modulus of Subgrade Reaction

Mechanistic solutions for the stresses and strains in rigid pavements have historically characterized the stiffness of the foundation soil in terms of the modulus of subgrade reaction *k* (Figure 5-23). However, the modulus of subgrade reaction is not a true engineering property for the foundation soil because it depends not only upon the soil stiffness, but also upon the slab (or footing) size and stiffness. For an example of a square footing on a homogeneous isotropic elastic foundation soil, *k* can be expressed as

$$k = \frac{0.65E}{B\left(1-\nu^2\right)} \sqrt[12]{\frac{EB^4}{E_f I}} \tag{5.19}$$

in which

| | | |
|---|---|---|
| $B$ | = | width of footing |
| $E$ | = | elastic modulus of soil |
| $\nu$ | = | Poisson's ratio of soil |
| $E_f$ | = | elastic modulus of footing |
| $I$ | = | moment of inertia of footing = $\frac{Bt^3}{12}$, $t$ = footing thickness |

For a given slab/footing size and stiffness, *k* is directly proportional to the effective elastic modulus of the foundation soil in Eq. (5.19).

The effective modulus of subgrade reaction is a direct input in the AASHTO design procedures for rigid pavements (see Section 3.5.2). The modulus of subgrade reaction was first introduced in the 1972 version of the Guide, with the recommendation that its value be determined from plate loading tests. Beginning with the 1986 Guide, the recommended procedure for estimating *k* for new/reconstruction designs is through correlations with subgrade $M_R$ plus various adjustments for base layer stiffness and thickness, presence of shallow rock, potential loss of slab support due to erosion, and seasonal variations.[2] The recommended procedure for determining *k* for rehabilitation designs is backcalculation from FWD test results.

[2] The 1998 supplement to the 1993 AASHTO Guide provides an alternate approach for determining the effective modulus of subgrade reaction for rigid pavements.

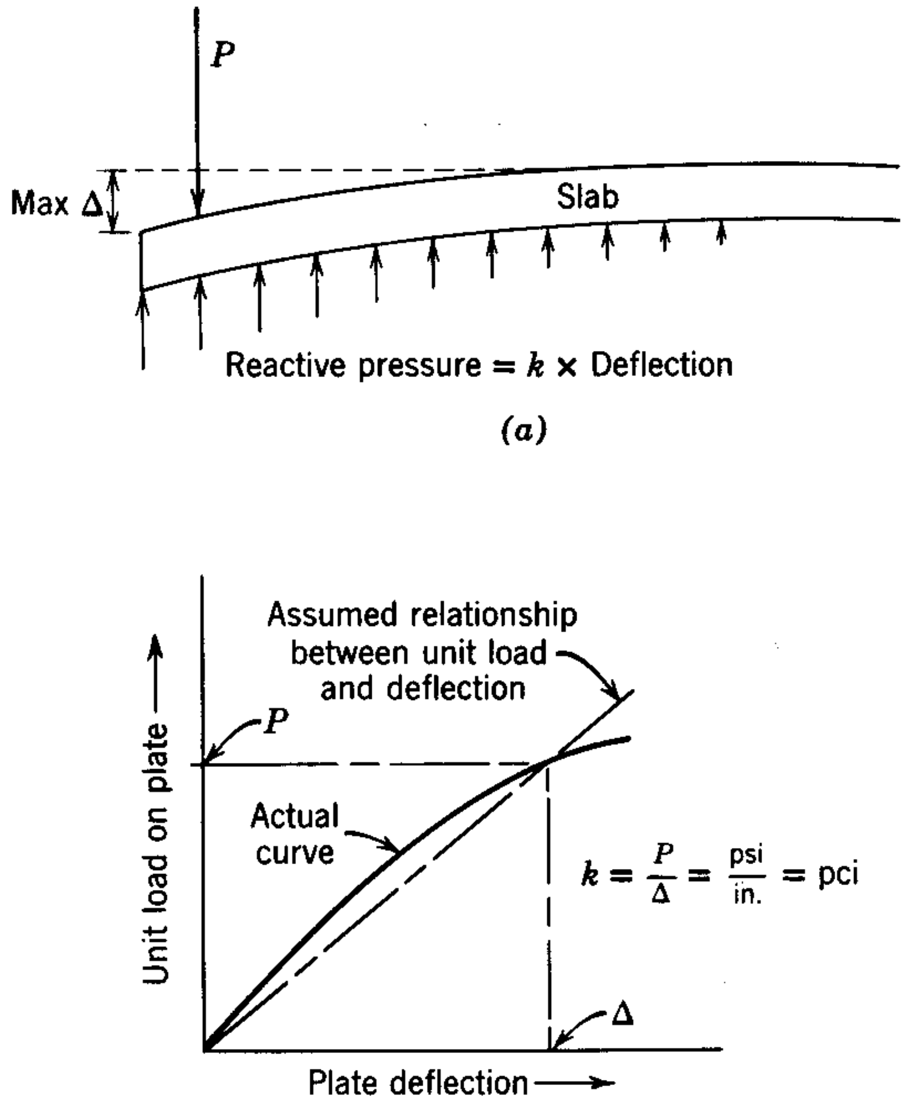



Figure 5-23. Coefficient of subgrade reaction *k* (Yoder and Witczak, 1975).

The subgrade, base, and subbase resilient moduli values are the direct inputs in the NCHRP 1-37A design methodology. These values are adjusted internally within the NCHRP 1-37A Design Guide software for environmental effects and then converted into an average monthly effective k-value for structural response calculation and damage analysis.

The detailed procedures used in the 1993 AASHTO and NCHRP 1-37A Design Guides to determine *k* for new/reconstruction and rehabilitation designs are described in the following subsections.

### *1993 AASHTO Guide*

New Construction/Reconstruction

The steps recommended in the 1993 AASHTO Design Guide for determining the effective modulus of subgrade reaction for new/reconstruction designs are as follows:

1. Identify the subgrade and subbase type(s), thicknesses, and other properties.

2. Determine the subgrade resilient modulus $M_R$ values for each season. Appropriate techniques for this are similar to those described earlier in Section 5.4.3.

3. Determine the subbase[3] resilient modulus $E_{SB}$ for each season (similar to step 2). The 1993 AASHTO Guide recommends the following limits for the subbase resilient modulus:

$$15{,}000 \text{ (spring thaw)} < E_{SB} \text{ (psi)} < 50{,}000 \text{ (winter freeze)} \tag{5.20}$$

$$E_{SB} < 4M_R \text{ (psi)} \tag{5.21}$$

4. Using Figure 5-24, determine a composite $k$ value for each season that represents the combined stiffness of the subgrade and subbase. Figure 5-24 is based on the following model (see Appendix LL in Volume 2 of AASHTO, 1986):

$$\begin{aligned}\ln k_\infty = &-2.807 + 0.1253(\ln D_{SB})^2 + 1.062\left(\ln M_R\right) + 0.1282\left(\ln D_{SB}\right)\left(\ln E_{SB}\right) \\ &- 0.4114\left(\ln D_{SB}\right) - 0.0581\left(\ln E_{SB}\right) - 0.1317\left(\ln D_{SB}\right)\left(\ln M_R\right)\end{aligned} \tag{5.22}$$

in which

$k_\infty$ = composite modulus of subgrade reaction (pci) assuming a semi-infinite roadbed soil
$D_{SB}$ = subbase thickness (inches)
$E_{SB}$ = subbase elastic modulus (psi)
$M_R$ = subgrade resilient modulus (psi)

5. Using Figure 5-25, correct the composite $k$ values from step 4 for any effects of shallow bedrock. Figure 5-25 is based on the following model (see Appendix LL in Volume 2 of AASHTO, 1986):

[3] In the 1993 AASHTO Guide terminology, the subbase is defined as the granular layer between the PCC slab and the roadbed (subgrade) soil. This layer is termed the base layer in the 1998 supplement to the 1993 Guide.

$$\ln k_{rf} = 5.303 + 0.0710\left(\ln D_{SB}\right)\left(\ln M_R\right) + 1.366\left(\ln k_\infty\right) \\ - 0.9187\left(\ln D_{SG}\right) - 0.6837\left(\ln M_R\right) \tag{5.23}$$

in which

$k_{rf}$ = composite modulus of subgrade reaction (pci) considering the effect of a rigid foundation near the surface

$D_{SG}$ = depth to rigid foundation (inches)

and the other terms are as defined previously in Eq. (5.22).

6. Determine the seasonal average composite $k$ value using the following procedure:
   - Estimate the design thickness of the slab and use Figure 5-26 to determine the relative damage $u_{ri}$ for each season. Figure 5-26 is based on the following simplified damage model (see Appendix HH in Volume 2 of AASHTO, 1986):

     $$u_{ri} = \left[D^{0.75} - 0.39k_i^{0.25}\right]^{3.42} \tag{5.24}$$

     in which $D$ is the slab thickness (inches) and $k_i$ is the modulus of subgrade reaction for each season.
   - Compute the average relative damage $\overline{u}_r$ as the sum of the relative damage values for each season divided by the number of seasons.
   - Determine the seasonally averaged composite $k$ from Figure 5-26 using $\overline{u}_r$ and the estimated slab thickness. This seasonally averaged composite $k$ is termed the effective modulus of subgrade reaction $k_{eff}$.

7. Using Figure 5-27 and Table 5-45, correct the effective modulus of subgrade reaction $k_{eff}$ for loss of support due to subbase erosion. This corrected $k_{eff}$ is the value to be used for design. Table 5-46 summarizes the recommended design values for the modulus of subgrade reaction from the low-volume road section of the 1993 Design Guide.

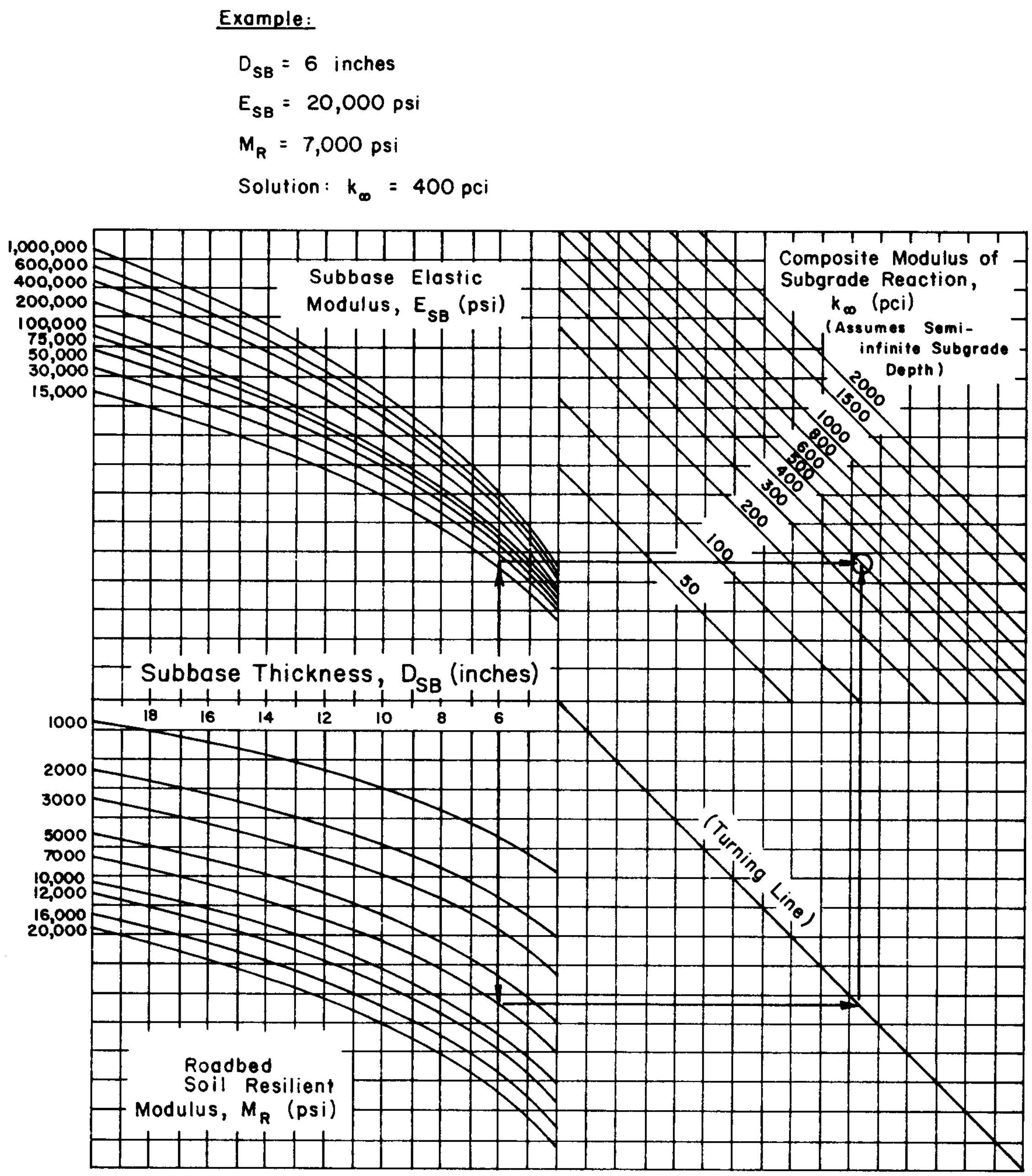



Figure 5-24. Chart for estimating composite modulus of subgrade reaction $k_\infty$, assuming a semi-infinite subgrade depth (AASHTO, 1993).

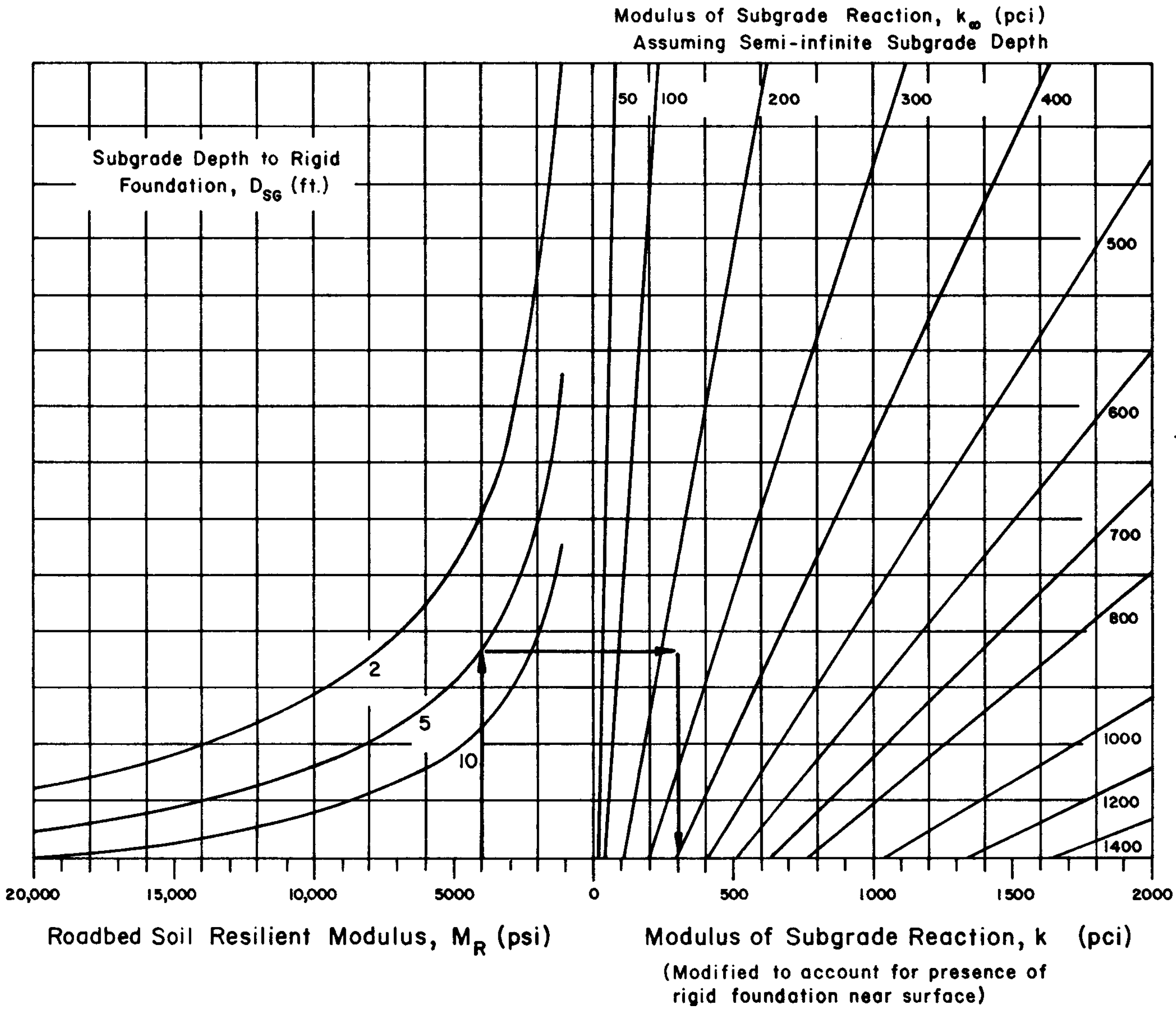



Figure 5-25. Chart to modify modulus of subgrade reaction to consider effects of rigid foundation near surface (within 10 ft) (AASHTO, 1993).

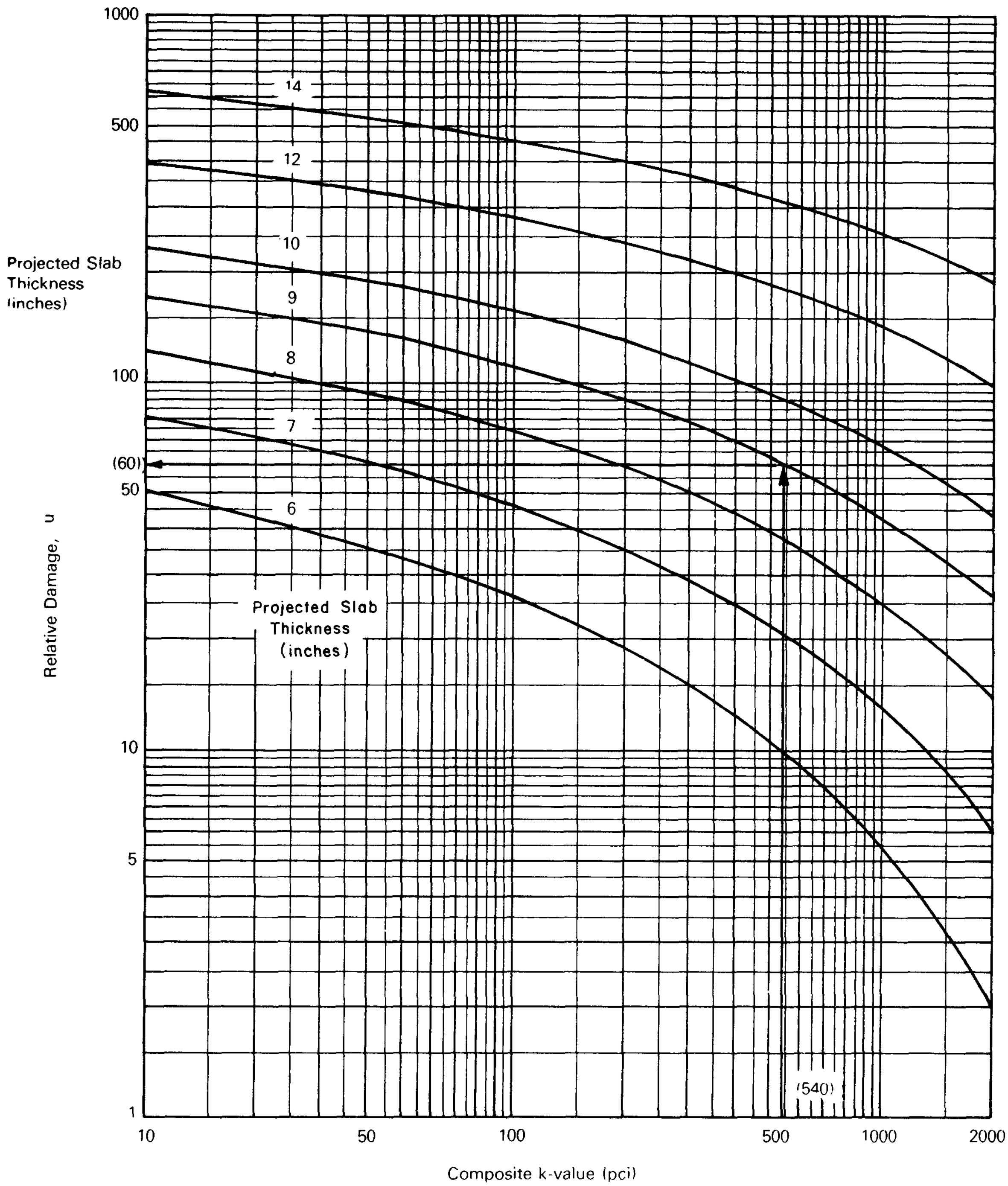



Figure 5-26. Chart for estimating relative damage to rigid pavements based on slab thickness and underlying support (AASHTO, 1993).

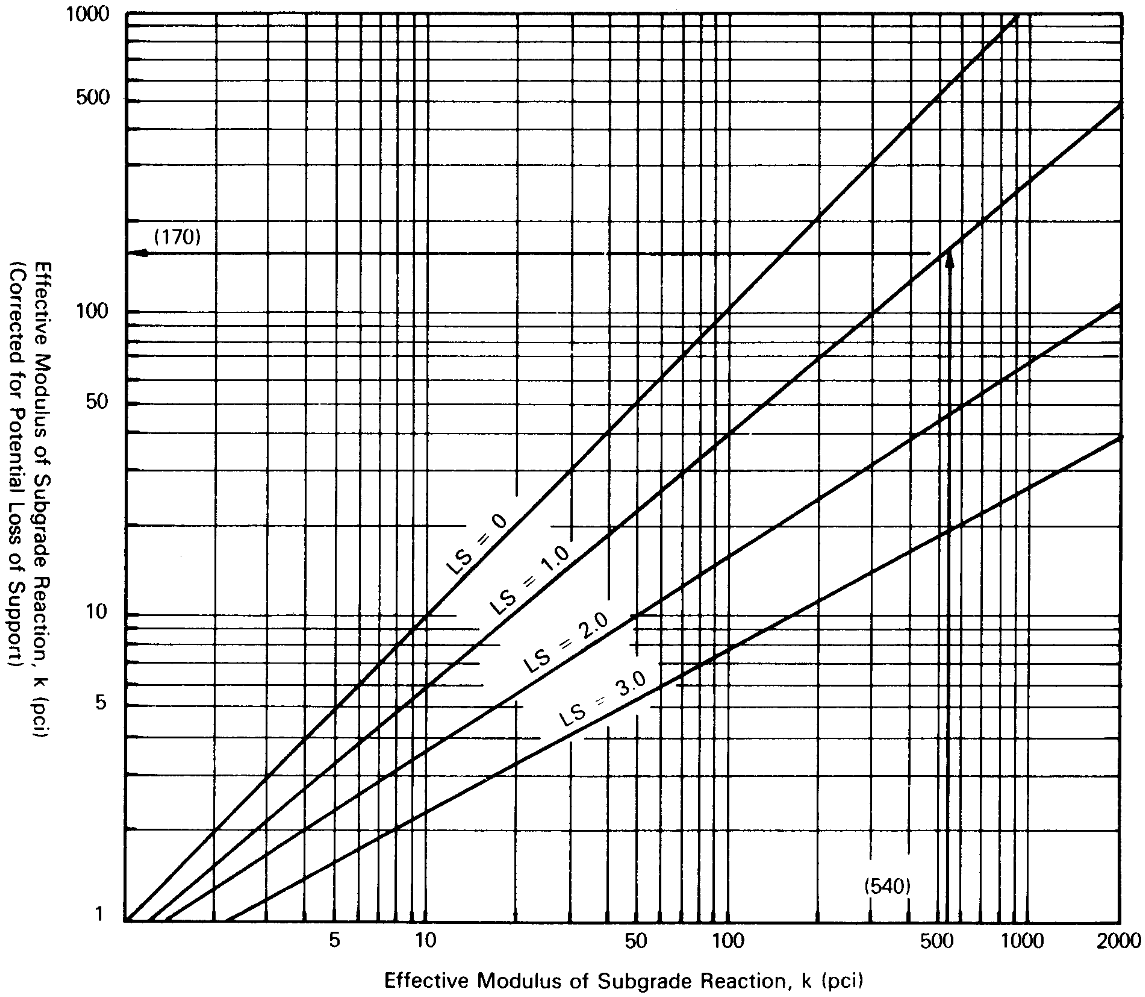



Figure 5-27. Correction of effective modulus of subgrade reaction for potential loss of subbase support (AASHTO, 1993).

**Table 5-45. Typical ranges of loss of support *LS* factors for various types of materials (AASHTO, 1993).**

| Type of Material | Loss of Support (LS) |
|---|---|
| Cement treated granular base (E = 1,000,000 to 2,000,000 psi) | 0.0 to 1.0 |
| Cement aggregate mixtures (E = 500,000 to 1,000,000 psi) | 0.0 to 1.0 |
| Asphalt treated base (E = 350,000 to 1,000,000 psi) | 0.0 to 1.0 |
| Bituminous stabilized mixtures (E = 40,000 to 300,000 psi) | 0.0 to 1.0 |
| Lime stabilized (E = 20,000 to 70,000) | 1.0 to 3.0 |
| Unbound granular materials (E = 15,000 to 45,000 psi) | 1.0 to 3.0 |
| Fine grained or natural subgrade materials (E = 3,000 to 40,000 psi) | 2.0 to 3.0 |

**Table 5-46. Suggested ranges for modulus of subgrade reaction for design (AASHTO, 1993).**

| Roadbed Soil Quality | Range for $k_{eff}$ (pci) |
|---|---|
| Very Good | > 550 |
| Good | 400 – 500 |
| Fair | 250 – 350 |
| Poor | 150 – 250 |
| Very Poor | < 150 |

<u>Rehabilitation</u>

For rehabilitation projects, the modulus of subgrade reaction *k* can be determined from FWD deflection testing of the existing PCC pavement. An FWD with a load plate radius of 5.9 inches and a load magnitude of 9000 pounds is recommended, with deflections measured at sensors located at 0, 12, 24, and 36 inches from the center of the load along the outer wheel path. For each slab tested, a dynamic $k_{dynamic}$ value (pci) can be determined from Figure 5-28 based on the deflection at the center of the loading plate, $d_0$ (inches) and the AREA of the deflection basin as computed by[4]

$$AREA = 6\left[1 + 2\left(\frac{d_{12}}{d_0}\right) + 2\left(\frac{d_{24}}{d_0}\right) + \left(\frac{d_{36}}{d_0}\right)\right] \tag{5.25}$$

in which the $d_i$ values are the deflections at $i$ inches from the plate center. The static $k_{eff}$ value for design is then determined as:

$$k_{eff} = \frac{k_{dynamic}}{2} \tag{5.26}$$

As is the case for new/reconstruction, this $k_{eff}$ value may need to be adjusted for seasonal effects.

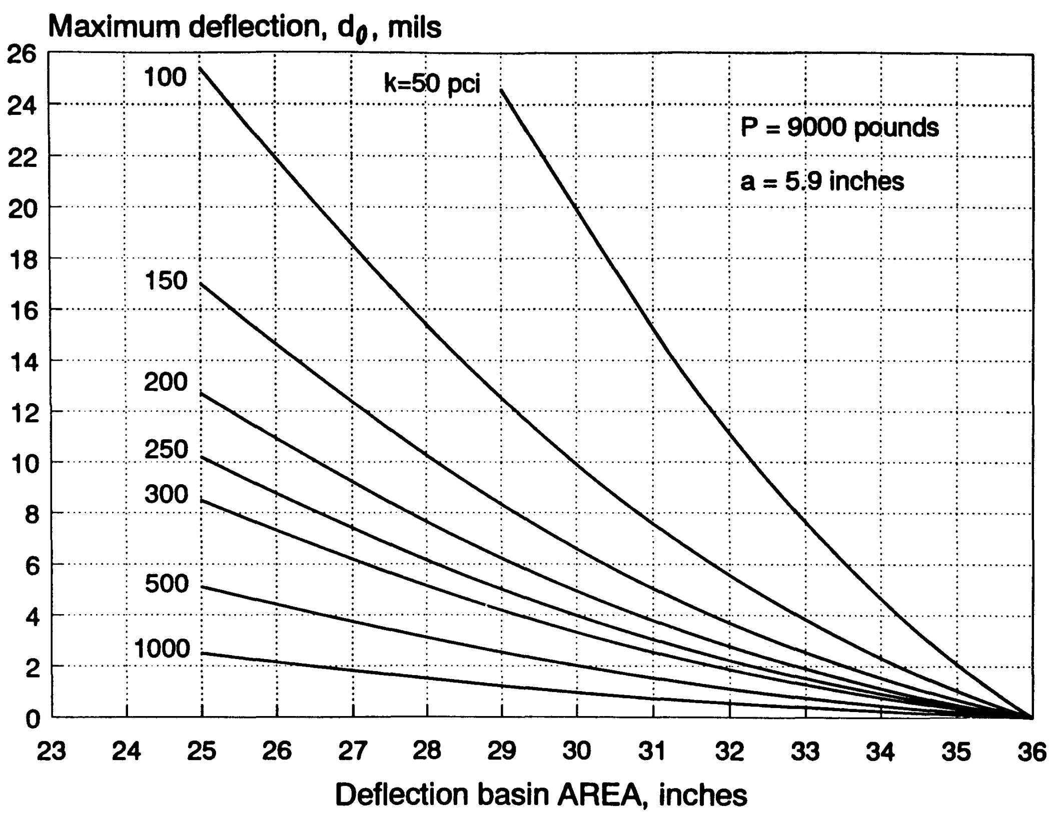



Figure 5-28. Effective dynamic $k$ value determination from $d_0$ and $AREA$ (AASHTO, 1993).

[4] For loads within 2000 pounds more or less of the 9000 pound desired value, deflections may be linearly scaled to equivalent 9000-pound deflections.

### *NCHRP 1-37A Design Guide*

New Construction/Reconstruction

All subgrade and unbound pavement layers for all pavement types are characterized using $M_R$ in the NCHRP 1-37A design methodology. The pavement response model for rigid pavement design, however, is based on a Winkler-spring foundation model that requires a value for the modulus of subgrade reaction $k_{dynamic}$ (see Appendix D for more details on the rigid pavement response model). The modulus of subgrade reaction is obtained from the subgrade and subbase $M_R$ values and the subbase thickness through a conversion process that transforms the actual multilayer pavement structure into an equivalent three-layer structure consisting of the PCC slab, base, and an effective dynamic *k*, as shown in Figure 5-29. This conversion is performed internally in the NCHRP 1-37A Design Guide software as a part of input processing.

The procedure to obtain the effective value of $k_{dynamic}$ for each time increment in the analysis can be summarized in the following steps:

1. Assign initial estimates of the stiffness parameters $M_R$ and $\nu$ to each unbound layer in the pavement structure.
2. Using multilayer elastic theory, simulate an FWD load and compute the stresses in the subgrade and subbase.
3. Adjust the subgrade and subbase $M_R$ values to account for the stress states determined in Step 2.
4. Using multilayer elastic theory, again simulate an FWD load using the updated subgrade and subbase $M_R$ values from Step 3. Calculate the PCC surface deflections at specified radii from the center of the load plate.
5. Using the rigid pavement response model, determine the $k_{dynamic}$ value that gives the best fit to the PCC surface deflections from Step 4.

The $k_{dynamic}$ value represents the compressibility of all layers beneath the base layer.
It is a computed quantity and not a direct input to the NCHRP 1-37A design procedure for new/reconstruction. Note also that $k_{dynamic}$ is a dynamic value, which should be distinguished from the traditional static *k* values used in previous design procedures.

The $k_{dynamic}$ value is calculated for each month of the year. It is used directly in the computation of the critical stresses, strains, and deflections for the incremental damage accumulation algorithms in the NCHRP 1-37A performance forecasting procedure. Environmental factors like water table depth, depth to bedrock, and freeze/thaw that can significantly affect the value for $k_{dynamic}$ are all considered in the NCHRP 1-37A calculations via the Enhanced Integrated Climate Model (EICM). Additional details of these algorithms are provided in Appendix D.

Rehabilitation

The modulus of subgrade reaction is a direct input for rigid pavement rehabilitation designs in the NCHRP 1-37A procedure. Measured surface deflections from FWD testing are used to backcalculate a $k_{dynamic}$ for design. The mean backcalculated $k_{dynamic}$ for a given month is input to the NCHRP 1-37A Design Guide software, and the $k_{dynamic}$ values for the remaining months of the year are seasonal adjustment factors computed by the EICM.

### 5.4.7 Interface Friction

1993 AASHTO Guide

The reinforcement design of jointed reinforced concrete pavements (JCRP) is dependent upon the frictional resistance between the bottom of the slab and the top of the underlying subbase or subgrade. This frictional resistance is characterized in the 1993 AASHTO Guide by a friction factor $F$ that is related (but not equal) to the coefficient of friction between the slab and the underlying material. Recommended values for natural subgrade and a variety of subbase materials are presented in Table 5-47. The friction factor is required only for JCRP design.

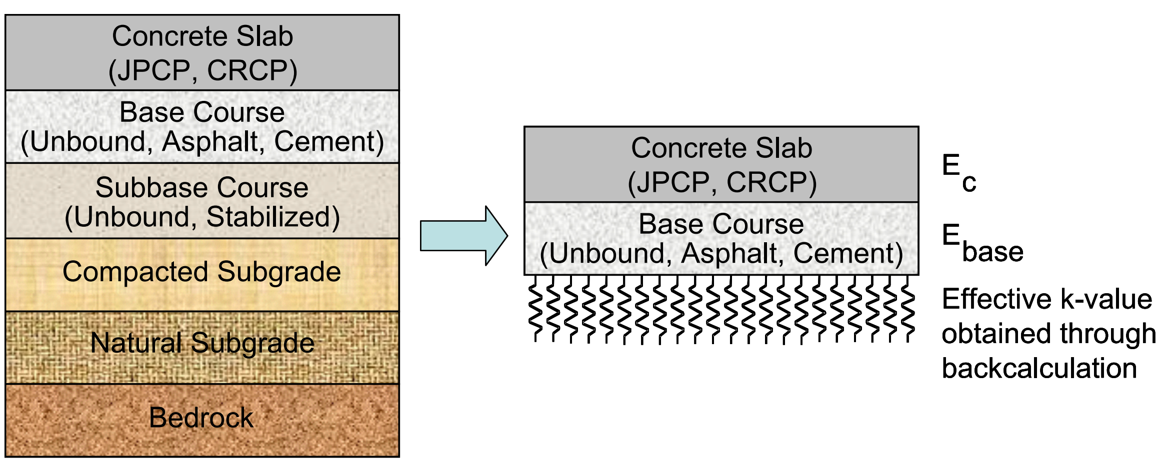



Figure 5-29. Structural model for rigid pavement structural response computations.

**Table 5-47. Recommended friction factor values (AASHTO, 1993).**

| Type of Material Beneath Slab | Friction Factor *F* |
|---|---|
| Surface treatment | 2.2 |
| Lime stabilization | 1.8 |
| Asphalt stabilization | 1.8 |
| Cement stabilization | 1.8 |
| River gravel | 1.5 |
| Crushed stone | 1.5 |
| Sandstone | 1.2 |
| Natural subgrade | 0.9 |

NCHRP 1-37A Procedure

The NCHRP 1-37A procedure for flexible pavements permits specification of the degree of bonding between each layer and the layer immediately beneath. The degree of bonding is characterized by an interface coefficient that varies between the limits of 1 for fully bonded conditions and 0 for a full slip interface. No guidance is provided at present in the NCHRP 1-37A procedure for specifying intermediate values for the interface coefficient to represent partial bond conditions between layers in flexible pavements.

The NCHRP 1-37A procedure for jointed plain concrete pavements (JPCP) requires specification of fully bonded for fully unbonded interface conditions between the bottom of the slab and the underlying layer. No provision is provided for intermediate bond conditions. The friction conditions at the bottom of continuously reinforced concrete pavements (CRCP) are specified in terms of a base/slab friction coefficient. Guidelines for specifying this coefficient are provided in Table 5-48. Jointed reinforced concrete pavement design is not included in the NCHRP 1-37A design procedures.

**Table 5-48. Typical values of base/slab friction coefficient recommended for CRCP design in the NCHRP 1-37A procedure (NCHRP 1-37A, 2004).**

| Subbase/Base Type | Friction Coefficient (low – medium – high) |
|---|---|
| Fine grained soil | 0.5 – 1.1 – 2.0 |
| Sand* | 0.5 – 0.8 – 1.0 |
| Aggregate | 0.5 – 2.5 – 4.0 |
| Lime stabilized clay* | 3.0 – 4.1 – 5.3 |
| Asphalt treated base | 2.5 – 7.5 – 15 |
| Cement treated base | 3.5 – 8.9 – 13 |
| Soil cement | 6.0 – 7.9 – 23 |
| Lime-cement-flyash | 3.0 – 8.5 - 20 |
| Lime-cement-flyash, not cured* | > 36 |

*Base type did not exist or was not considered in the NCHRP 1-37A calibration process.

### 5.4.8 Permanent Deformation Characteristics

The permanent deformation characteristics of unbound materials are used in the empirical rutting distress models in the NCHRP 1-37A design methodology. This information is not required for rigid pavement design in the NCHRP 1-37A Design Guide or at all in the 1993 AASHTO design procedure. Permanent deformation characteristics are measured via triaxial repeated load tests conducted for many cycles of loading; Figure 5-30 shows schematically the typical behavior measured in this type of test. The repeated load permanent deformation tests are very similar to the cyclic triaxial tests used to measure resilient modulus (see 5.4.3), except that the cyclic deviator stress magnitude is kept constant throughout the test. There are at present no ASTM or AASHTO test specifications for repeated load permanent deformation testing. However, the first 1000 conditioning cycles of the AASHTO T307-99 resilient modulus testing procedure are often used for permanent deformation modeling.

The NCHRP 1-37A design methodology characterizes the permanent deformation behavior of unbound base, subbase, and subgrade materials using a model based on work by Tseng and Lytton (1989):

$$\delta_a(N) = \xi_1 \left( \frac{\varepsilon_o}{\varepsilon_r} \right) e^{-\left(\frac{\rho}{N}\right)^{\xi_2 \beta}} \varepsilon_v h \tag{5.27}$$

in which

$\delta_a$ = Permanent deformation for the layer/sublayer
$N$ = Number of traffic repetitions
$\varepsilon_o$, $\beta$, $\rho$ = Material properties
$\varepsilon_r$ = Resilient strain imposed in laboratory test to obtain material properties $\varepsilon_o$, $\beta$, and $\rho$
$\varepsilon_v$ = Average vertical resilient strain in the layer/sublayer as obtained from the primary response model
$h$ = Thickness of the layer/sublayer
$\xi_1$, $\xi_2$ = Field calibration functions

Tseng and Lytton provide regression equations for the $\varepsilon_o/\varepsilon_r$, $\rho$, and $\beta$ terms. These regression equations were substantially revised during development of the NCHRP 1-37A design methodology. The revised equations implemented in the NCHRP 1-37A procedure are as follows:

$$\log\left(\frac{\varepsilon_o}{\varepsilon_r}\right) = 0.74168 + 0.08109W_c - 0.000012157M_R \tag{5.28}$$

$$\log\beta = -0.61119 - 0.017638W_c \tag{5.29}$$

$$\log\rho = 0.622685 + 0.541524W_c \tag{5.30}$$

In Eq. (5.28) through Eq. (5.30), $M_R$ is the resilient modulus in psi, and $W_c$ is an estimate of the average in-situ gravimetric water content in percent. The NCHRP 1-37A procedure proposes the following equation for determining $W_c$ in the absence of measured values:

$$W_c = 51.712\left(CBR\right)^{-0.3586(GWT)^{0.1192}} \qquad W_c \le W_{sat} \tag{5.31}$$

In Eq. (5.31), GWT is the depth to the groundwater table in feet, and CBR can be estimated from resilient modulus using

$$CBR = \left(\frac{M_R}{2555}\right)^{(1/0.64)} \tag{5.32}$$

The $W_{sat}$ limit in Eq. (5.31) can be determined from

$$W_{sat} = \left( \frac{2.75}{SPG} - 1 \right) * 100 / 2.75 \tag{5.33}$$

where *SPG* is the saturated specific gravity of the soil. For laboratory test conditions, $W_c$ is presumably equal to the tested water content.

Although fine-tuning of the calibration is still underway and therefore the expressions for $\xi_1$, $\xi_2$ may yet change, the current best estimates are as follows:

$$\xi_1 = 1.2 - 1.39 e^{-0.058(M_R/1000)} \qquad \le 1 \times 10^{-7} \tag{5.34}$$

$$\xi_2 = 0.7 \tag{5.35}$$

In Eq. (5.34), a lower bound of 2.6 is set for $M_R/1000$.

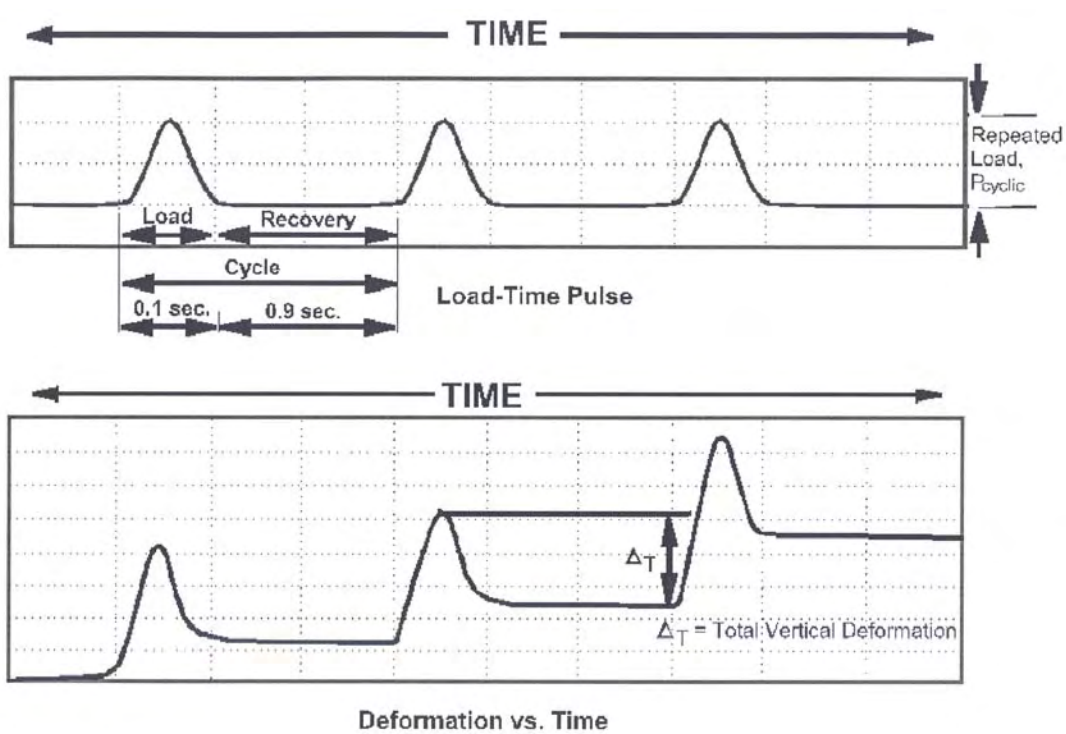



Figure 5-30. Accumulation of permanent deformations with repeated cyclic loading (LTPP, 2003).

### 5.4.9 Coefficient of Lateral Pressure

The coefficient of lateral earth pressure $K_0$ is defined as the ratio of the horizontal to vertical in-situ effective stress:

$$K_0 = \frac{\bar{\sigma}_{ho}}{\bar{\sigma}_{vo}} \tag{5.36}$$

The coefficient of lateral earth pressure is an input in the NCHRP 1-37A design procedure. It is used to compute the combined in-situ and induced stress states within the pavement system.

Elasticity theory can be used to estimate $K_0$ based on the confined Poisson expansion:

$$K_0 = \frac{\nu}{1-\nu} \tag{5.37}$$

in which $\nu$ is Poisson's ratio. Values of $K_0$ predicted by Eq. (5.37) for typical geomaterials range between 0.4 and 0.6.

A common empirical correlation for $K_0$ for cohesionless and normally consolidated cohesive soils is the Jaky relationship:

$$K_0 = 1 - \sin\phi \tag{5.38}$$

in which $\phi$ is the friction angle. Overconsolidation in cohesive soils will increase the value for $K_0$ above that given in Eq. (5.38). Figure 5-31 shows the typical relationship between $K_0$, the overconsolidation ratio OCR, and the plasticity index *PI*.

Loading followed by unloading and reloading, such as occurs during compaction of unbound materials in pavements, often results in an increase in $K_0$. The relative magnitudes of horizontal and vertical stress during a load-unload-reload path are shown schematically in Figure 5-32. Mayne and Kulhawy (1982) proposed the following model for $K_0$ after loading-unloading-reloading:

$$K_0 = \left(1-\sin\phi\right)\left[\frac{OCR}{OCR_{\max}^{(1-\sin\phi)}} + \left(\frac{3}{4}\right)\frac{OCR}{OCR_{\max}}\right] \tag{5.39}$$

in which $OCR_{max}$ is the maximum overconsolidation ratio achieved in the load path and the other terms are as defined previously.

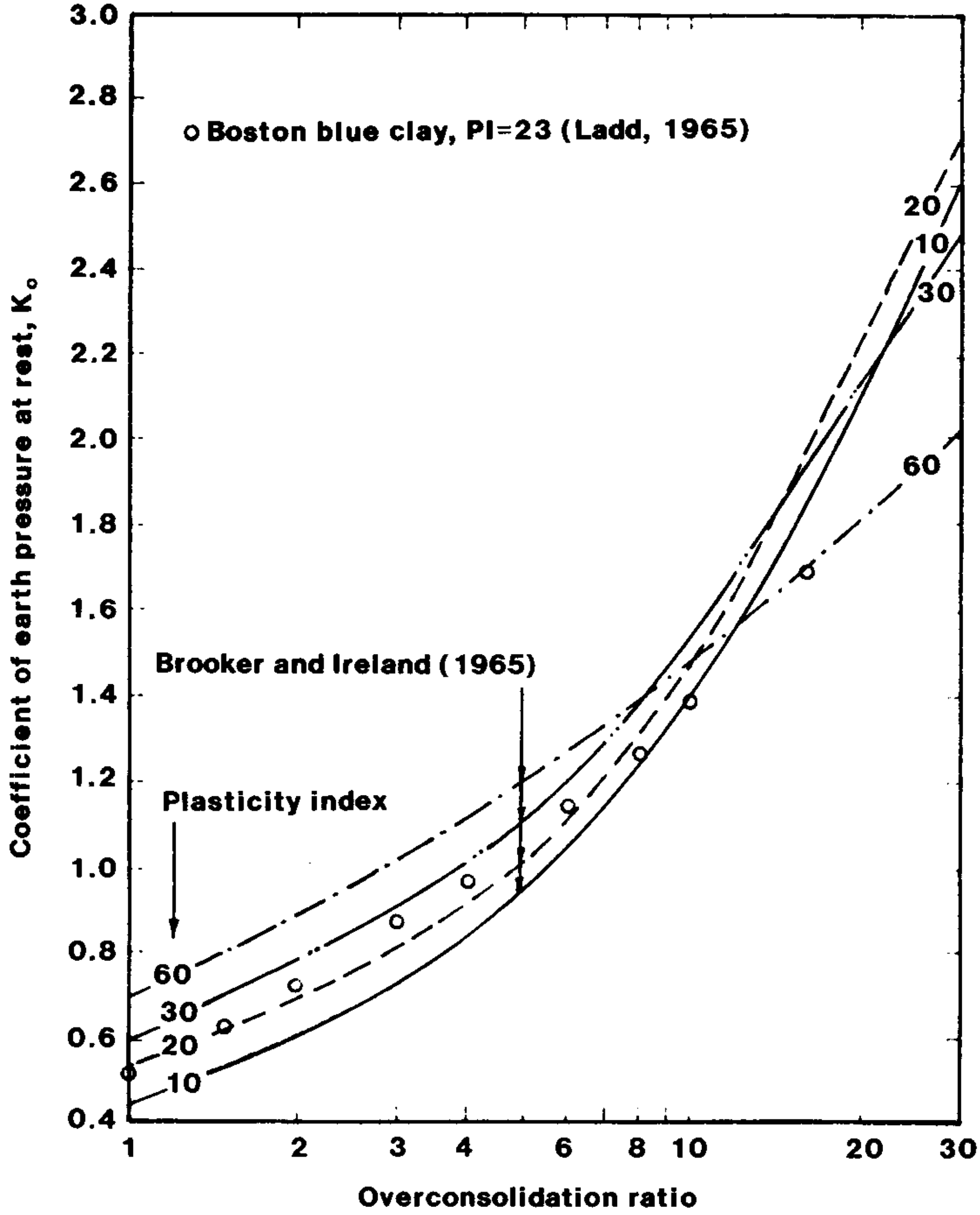



Figure 5-31. Correlation between coefficient of lateral earth pressure and overconsolidation ratio for clays of various plasticity indices (Carter and Bentley, 1991).

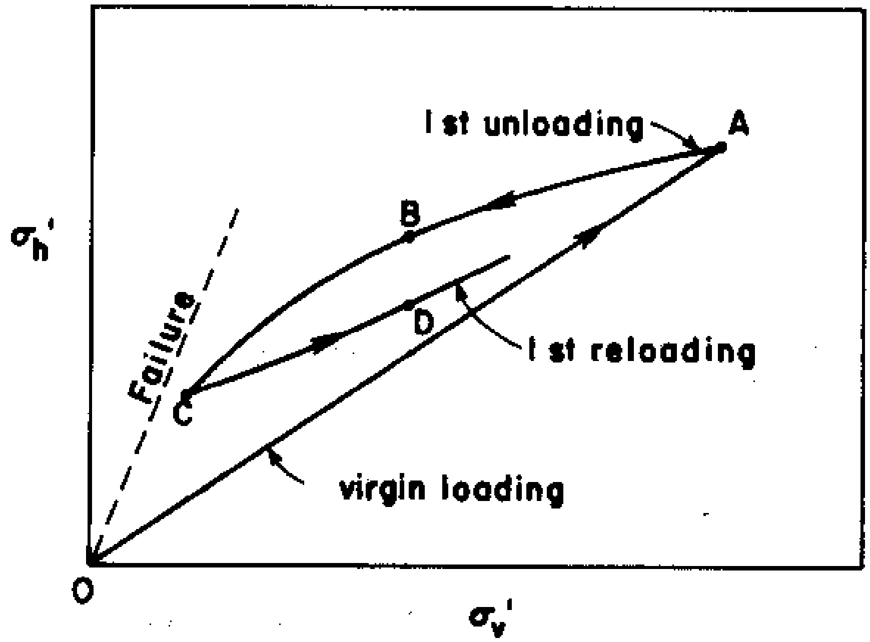



Figure 5-32. Horizontal and vertical in-situ stresses during a load-unload-reload path (Mayne and Kulhawy, 1982).

## 5.5 THERMO-HYDRAULIC PROPERTIES

Thermo-hydraulic material properties are required to evaluate the temperature and moisture conditions in a pavement system and their effects on the material behavior. Temperature has significant effects on the stiffness of asphalt concrete, and temperature gradients can induce thermal curling and stresses in rigid pavement slabs. Moisture content influences the stiffness and strength of unbound materials, and moisture gradients can induce warping of rigid pavement slabs. Combined temperature and moisture effects can cause detrimental freeze/thaw cycles in unbound materials.[5]

The empirical 1993 AASHTO Design Guide and the mechanistic-empirical NCHRP 1-37A design procedure have drastically different input requirements for thermo-hydraulic properties. The thermo-hydraulic design inputs in the 1993 AASHTO Guide are largely empirical coefficients grouped in the following categories:

- Drainage coefficients (for unbound layers)
- Swelling parameters (for expansive subgrade soils)
- Frost heave parameters (for frost-susceptible subgrade soils)

[5] Moisture and freeze/thaw are also important factors behind stripping of asphalt concrete, but this material phenomenon is beyond our scope.

These empirical properties in the 1993 Guide often mix material property and climate factors. For example, drainage coefficients are functions of both climate-determined moisture conditions and material-related drainage quality.

The thermo-hydraulic properties required as input to the NCHRP 1-37A Design Guide tend to be more fundamental material properties. These include

- Groundwater table depth
- Infiltration and drainage properties
- Physical properties
- Soil water characteristic curve
- Hydraulic conductivity (permeability)
- Thermal conductivity
- Heat capacity

These thermo-hydraulic properties are used in the mechanistic Enhanced Integrated Climate Model (EICM) along with climate inputs (discussed separately in Section 5.6) to predict temperature and moisture distributions in the pavement as functions of depth and time. Appendix D provides details on algorithms embedded in the EICM.

Because of the substantial differences in these thermo-hydraulic inputs to the two design methods, each design method is discussed separately in the following subsections.

### 5.5.1 1993 AASHTO Guide

The environment-related aspects in the 1993 AASHTO Design Guide are grouped into two general categories: drainage and subgrade swelling/frost heave. As described in Section 3.5.2 in Chapter 3, drainage is incorporated via adjustment to the unbound structural layer coefficients for flexible pavements or via a drainage factor in the design equation for rigid pavements. Swelling and/or frost heave, on the other hand, is incorporated via a partitioning of the total allowable serviceability loss $\Delta PSI$; part of $\Delta PSI$ is allocated to environment-induced deterioration due to swelling and/or frost heave, and the remainder of $\Delta PSI$ is allocated to traffic-induced deterioration.

<u>Drainage Coefficients</u>

The 1993 AASHTO Guide provides guidance for the design of subsurface drainage systems and modifications to the flexible and rigid pavement design procedure to take advantage of improvements in performance due to good drainage. For flexible pavements, the benefits of drainage are incorporated into the structural number via empirical drainage coefficients:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \quad (5.40)$$

in which $m_2$ and $m_3$ are the drainage coefficients for the base and subbase layers, respectively, and all other terms are as defined previously. Table 5-49 summarizes the recommended values for $m_i$ in the 1993 AASHTO Guide as functions of qualitative descriptions of drainage quality and climate conditions.

For rigid pavements, the benefits of drainage are incorporated via an empirical drainage coefficient $C_d$ in the rigid pavement design equation. Table 5-50 summarizes the recommended values for $C_d$ in the 1993 AASHTO Guide as a function of qualitative descriptions of drainage quality and climate conditions.

**Table 5-49. Recommended $m_i$ values for modifying structural layer coefficients of untreated base and subbase materials in flexible pavements (AASHTO, 1993).**

| Quality of Drainage | Water Removed Within | Percent of Time Pavement is Exposed to Moisture Levels Approaching Saturation | | | |
|---|---|---|---|---|---|
| | | <1% | 1-5% | 5-25% | >25% |
| Excellent | 2 hours | 1.40-1.35 | 1.35-1.30 | 1.30-1.20 | 1.20 |
| Good | 1 day | 1.35-1.25 | 1.25-1.15 | 1.15-1.00 | 1.00 |
| Fair | 1 week | 1.25-1.15 | 1.15-1.05 | 1.00-0.80 | 0.80 |
| Poor | 1 month | 1.05-0.80 | 1.05-0.80 | 0.80-0.60 | 0.60 |
| Very Poor | no drainage | 0.95-0.75 | 0.95-0.75 | 0.75-0.40 | 0.40 |

**Table 5-50. Recommended values of drainage coefficient $C_d$ values for rigid pavement design (AASHTO, 1993).**

| Quality of Drainage | Water Removed Within | Percent of Time Pavement is Exposed to Moisture Levels Approaching Saturation | | | |
|---|---|---|---|---|---|
| | | <1% | 1-5% | 5-25% | >25% |
| Excellent | 2 hours | 1.25-1.20 | 1.20-1.15 | 1.15-1.10 | 1.10 |
| Good | 1 day | 1.20-1.15 | 1.15-1.10 | 1.10-1.00 | 1.00 |
| Fair | 1 week | 1.15-1.10 | 1.10-1.00 | 1.00-0.90 | 0.90 |
| Poor | 1 month | 1.10-1.00 | 1.00-0.90 | 0.90-0.80 | 0.80 |
| Very Poor | no drainage | 1.00-0.90 | 0.90-0.80 | 0.80-0.70 | 0.70 |

Swelling Parameters

The 1993 AASHTO Guide includes three empirical parameters for estimating potential serviceability loss due to swelling:

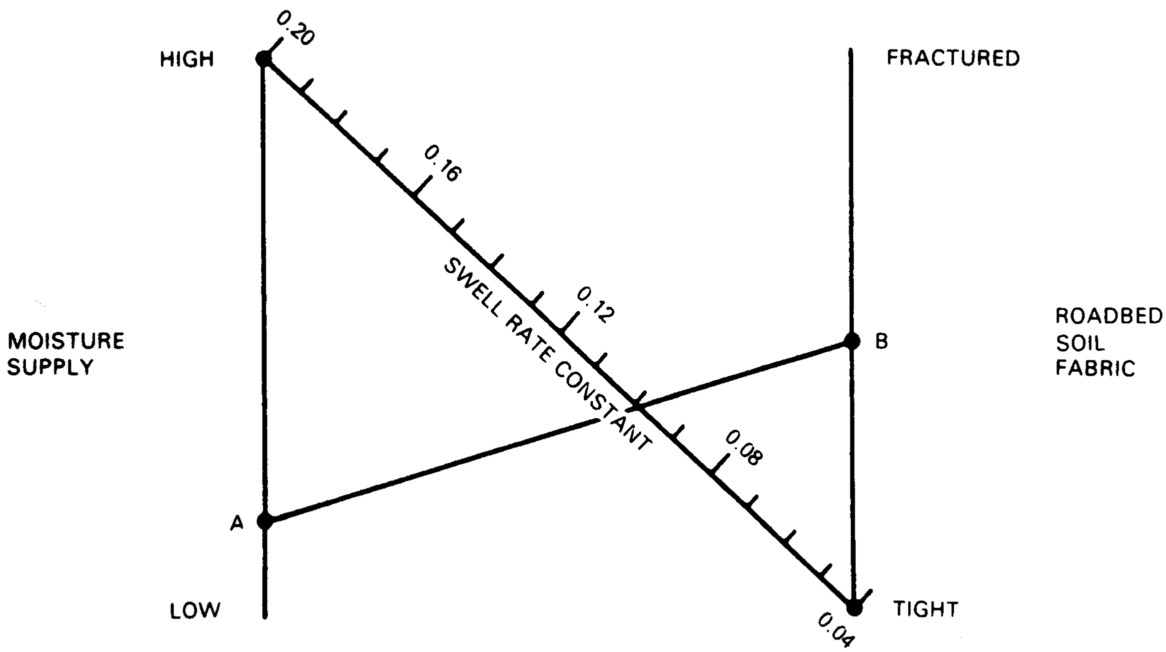
- Swell rate constant $\theta$
- Potential vertical rise $V_R$
- Swell probability $P_S$

The swell rate constant $\theta$ is used to estimate the rate at which swelling will take place. It varies between 0.04 and 0.20, with higher values appropriate for soils exposed to a large moisture supply either due to high rainfall, poor drainage, or some other source. Figure 5-33 provides a nomograph for subjectively estimating the rate of subgrade soil swelling based upon qualitative descriptions of moisture supply and soil fabric. Little guidance beyond that in Figure 5-33 is provided in the 1993 Guide for estimating the values for moisture supply and soil fabric.

The potential vertical rise $V_R$ is a measure of the vertical expansion that may occur in the subgrade soil under extreme swell conditions. Although it is possible to measure $V_R$ from laboratory swell tests, this is not commonly done in practice. Instead, $V_R$ is estimated using the chart in Figure 5-34 based on the soil's plasticity index, moisture condition, and overall thickness of the layer. The moisture condition is a subjective estimate of the difference between the in-situ moisture conditions during construction and moisture conditions at a later date.

The swell probability ($P_S$) is a measure of the proportion (percent) of the project length that is subject to swell. The probability of swelling at a given location is assumed to be 100% if the subgrade soil plasticity index is greater than 30 and the layer thickness is greater than 2 feet (or if $V_R$ is greater than 0.20 inches). These criteria can be used to separate the project length into swelling and nonswelling sections, from which a length-averaged estimate of $P_S$ can be determined.

These three swelling parameters are used in a nomograph (see Appendix C) along with the design life to determine the expected serviceability loss due to swelling $\Delta PSI_{SW}$. However, it should be clear from the empirical and highly subjective procedures used to determine the input parameters that the predicted $\Delta PSI_{SW}$ will be only a very approximate estimate.




NOTES: a) LOW MOISTURE SUPPLY:

Low rainfall
Good drainage

b) HIGH MOISTURE SUPPLY

High rainfall
Poor drainage
Vicinity of culverts, bridge abutments, inlet leads

c) SOIL FABRIC CONDITIONS (self explanatory)

d) USE OF THE NONOGRAPH

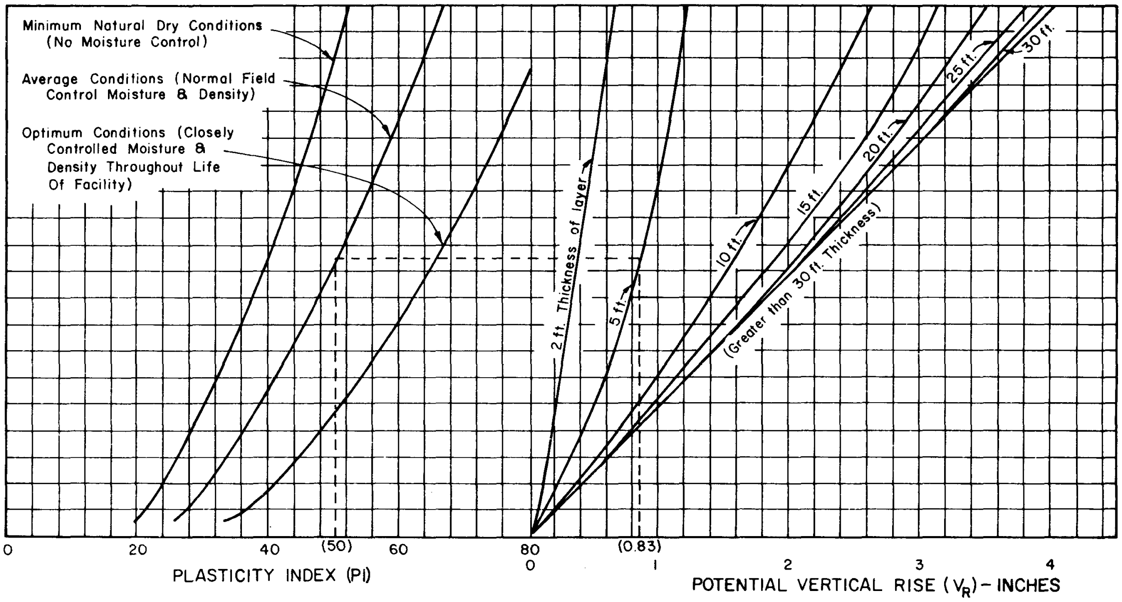
1) Select the appropriate moisture supply condition which may be somewhere between low and high (such as A).

2) Select the appropriate soil fabric (such as B). This scale must be developed by each individual agency.

3) Draw a straight line between the selected points (A to B).

4) Read swell rate constant from the diagonal axis (read 0.10).

Figure 5-33. Nomograph for estimating swell rate constant (AASHTO, 1993).




NOTES:

1. This figure is predicated upon the following assumptions:
   a. The subgrade soils for the thickness shown all are passing the No. 40 mesh sieve.
   b. The subgrade soil has a uniform moisture content and plasticity index throughout the layer thickness for the conditions shown.
   c. A surcharge pressure from 20 inches of overburden (± 10 inches will have no material effect).
2. Calculations are required to determine $V_R$ for other surcharge pressures.

Figure 5-34. Chart for estimating potential vertical rise of natural soils (AASHTO, 1993).

<u>Frost Heave Parameters</u>

The 1993 AASHTO Guide includes three empirical parameters for estimating potential serviceability loss due to frost heave:

- Frost heave rate $\phi$
- Maximum potential serviceability loss $\Delta PSI_{MAX}$
- Frost heave probability $P_F$

The frost heave rate $\phi$ is a measure of the rate of increase of frost heave in millimeters per day. The rate of frost heave depends on the type of subgrade material, in particular the percentage of fine-grained material. Figure 5-35 can be used to estimate the rate of frost heave based on the USCS class for the subgrade and the percentage of material finer than 0.02 mm.

The maximum potential serviceability loss $\Delta PSI_{MAX}$ due to frost heave is dependent on the quality of drainage and the depth of frost penetration. Figure 5-36 can be used to estimate the maximum potential serviceability loss due to these two factors. The drainage quality parameter in Figure 5-36 is the same as that used to define the drainage coefficients in Table

5-49 and Table 5-50. See Yoder and Witczak (1975) for methods for determining the depth of frost penetration.

The frost heave probability $P_F$ is the designer's estimate of the percentage length of the project that will experience frost heave. This estimate will depend upon the extent of frost-susceptible subgrade material, moisture availability, drainage quality, number of freeze-thaw cycles during the year, and the depth of frost penetration. Past experience is valuable here, as there is no clear method for approximating the frost heave probability.

These three frost heave parameters are used in a nomograph (see Appendix C) along with the design life to determine the expected serviceability loss due to frost heave $\Delta PSI_{FH}$. However, it should be clear from the empirical and highly subjective procedures used to determine the input parameters that the predicted $\Delta PSI_{FH}$ will be only a very approximate estimate.

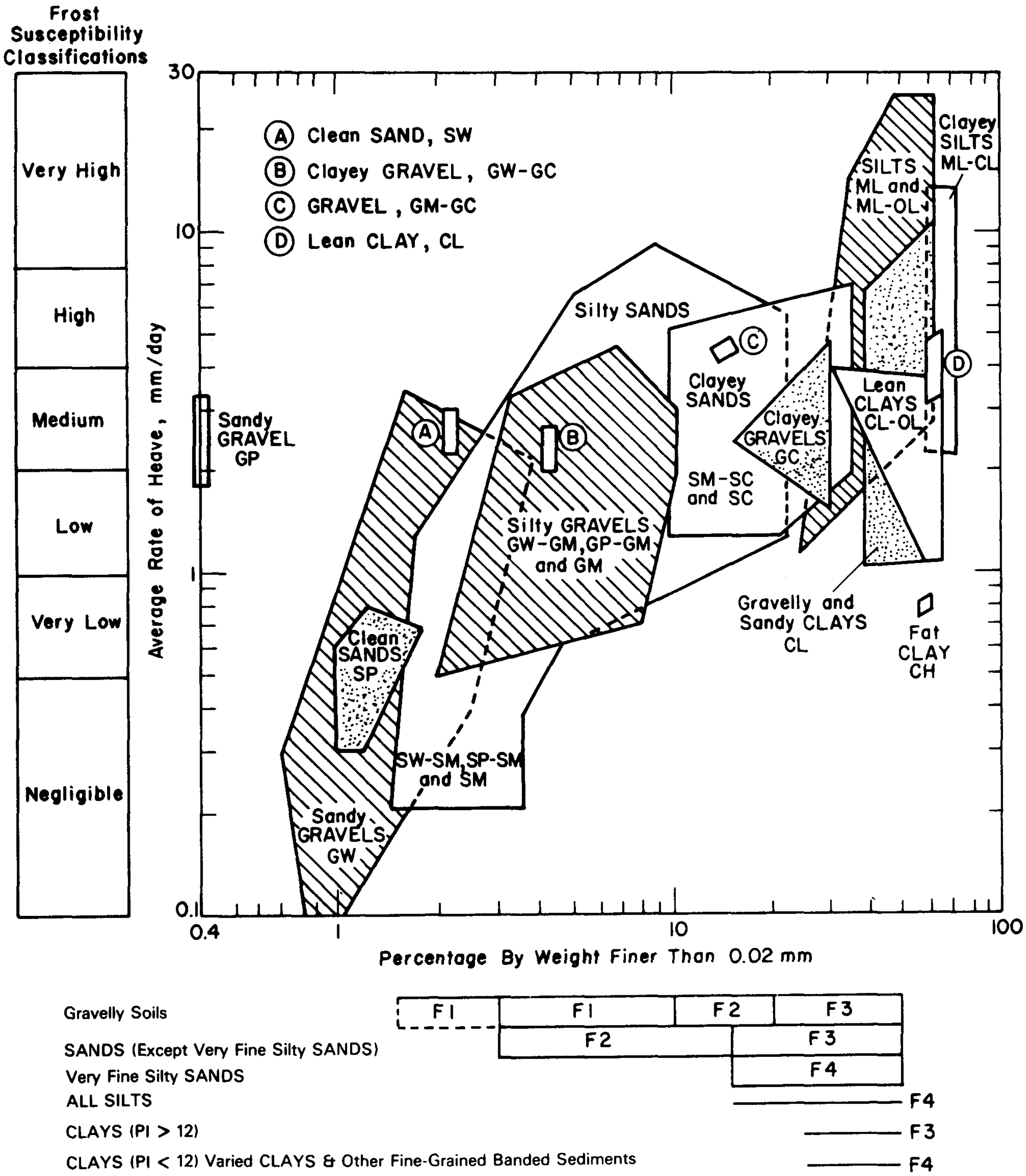



Figure 5-35. Chart for estimating frost heave rate for subgrade soil (AASHTO, 1993).

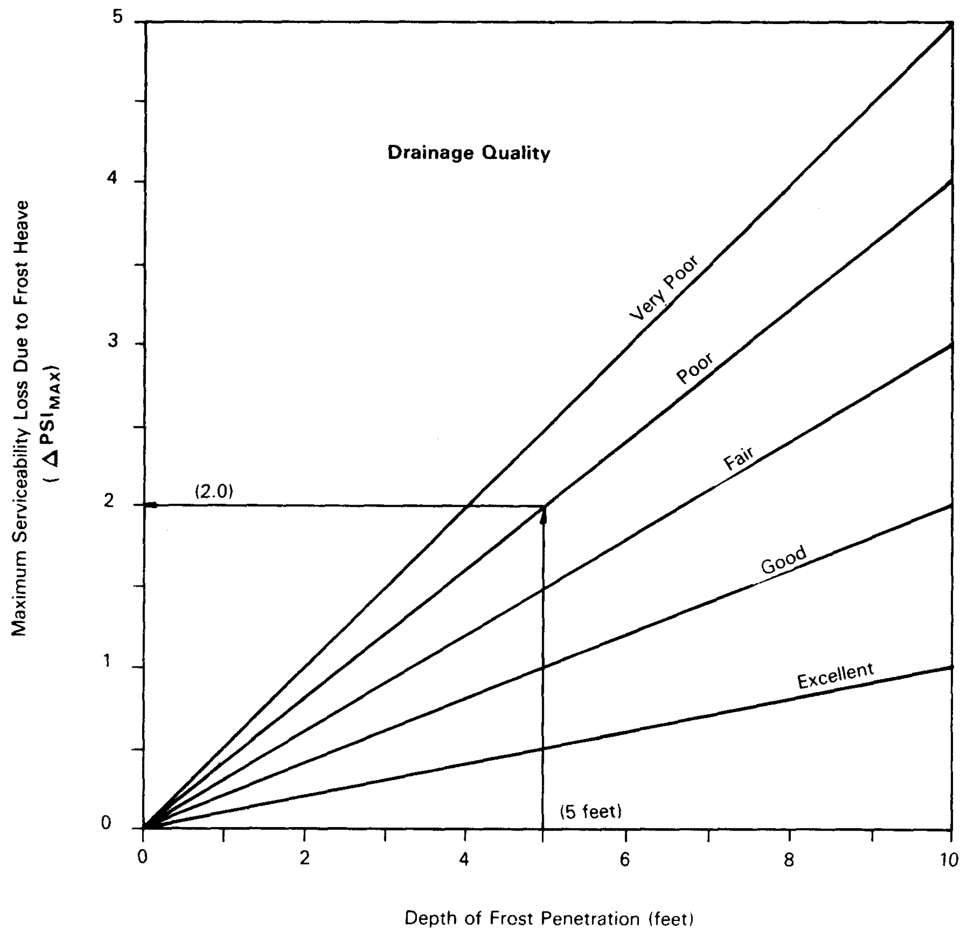



Figure 5-36. Graph for estimating maximum serviceability loss due to frost heave (AASHTO, 1993).

### 5.5.2 NCHRP 1-37A Design Guide

The thermo-hydraulic properties required as input to the NCHRP 1-37A Design Guide can be grouped into the following categories:

- Groundwater depth
- Infiltration and drainage properties
- Physical/index properties
- Soil water characteristic curve
- Hydraulic conductivity (permeability)
- Thermal conductivity
- Heat capacity

Methods for determining the design inputs in each of these categories are described in the following subsections. In some cases, the design inputs are determined by direct measurement in the laboratory or the field. However, other design inputs (*e.g.,* soil water characteristic curve) are much less commonly measured in geotechnical practice. Recognizing this, the NCHRP 1-37A project team expended substantial effort to develop robust correlations between these properties and other more conventional soil properties (*e.g.,* gradation and plasticity). These correlations are also detailed in the following subsections as appropriate.

***Groundwater Depth***

The groundwater depth plays a significant role in the NCHRP 1-37A Design Guide predictions of moisture content distributions in the unbound pavement materials and thus on the seasonal resilient modulus values. The input value is intended to be the best estimate of the annual average groundwater depth. Groundwater depth can be determined from profile characterization borings during design (see Section 4.7.1) or estimated. The county soil reports produced by the National Resources Conservation Service can often be used to develop estimates of groundwater depth.

***Infiltration and Drainage***

Three input parameters related to infiltration and drainage are required in the NCHRP 1-37A design methodology:

- Amount of infiltration
- Pavement cross slope
- Drainage path length

<u>Amount of Infiltration</u>

The amount of infiltration will be a function of rainfall intensity and duration (determined from the climate inputs, see Section 5.6), pavement condition, shoulder type, and drainage features. The NCHRP 1-37A Design Guide qualitatively divides infiltration into four categories, as summarized in Table 5-51. These categories are used at all hierarchical input levels. The infiltration category is based upon shoulder type, generally the largest single source of moisture entry into the pavement structure, and edge drains, since these shorten the drainage path and provide a positive drainage outlet. Note that if a drainage layer is present in addition to edge drains, its influence is automatically accounted for within the EICM moisture calculations.

**Table 5-51. Infiltration categories in the NCHRP 1-37A Design Guide (NCHRP 1-37A, 2004).**

| Infiltration Category | Conditions | % Precipitation Entering Pavement |
|---|---|---|
| None | | 0 |
| Minor | This option is valid when tied and sealed concrete shoulders (rigid pavements), widened PCC lanes, or full-width AC paving (monolithic main lane and shoulder) are used or when an aggressive policy is pursued to keep the lane-shoulder joint sealed. This option is also applicable when edge drains are used. | 10 |
| Moderate | This option is valid for all other shoulder types, PCC restoration, and AC overlays over old and cracked existing pavements where reflection cracking will likely occur. | 50 |
| Extreme | Generally not used for new or reconstructed pavement levels. | 100 |

Most designs and maintenance activities, especially for higher functional class pavements, should strive to achieve zero infiltration or reduce it to a minimum value. This can be done by proper design of surface drainage elements (cross slopes, side ditches, etc.), adopting construction practices that reduce infiltration (*e.g.,* eliminating cold lane/shoulder joints, use of tied joints for PCC pavements, etc.), proactive routine maintenance activities (*e.g.,* crack and joint sealing, surface treatments, etc.), and providing adequate subsurface drainage (*e.g.,* drainage layers, edge drains). Chapter 7 provides more information on pavement drainage systems.

Pavement Cross Slope

The pavement cross slope is the slope of the surface perpendicular to the direction of traffic. This input is used in computing the drainage path length, as described in the next subsection.

Drainage Path Length

The drainage path length is the resultant of the cross and longitudinal slopes of the pavement. It is measured from the highest point in the pavement cross section to the drainage outlet. This input is used in the EICM's infiltration and drainage model to compute the time required to drain an unbound base or subbase layer from an initially wet condition.

The DRIP computer program (Mallela *et al*., 2002) can be used to compute the drainage path length based on pavement cross and longitudinal slopes, lane widths, edge drain trench widths (if applicable, and cross section crown and superelevation). The DRIP program is provided as part of the NCHRP 1-37A Design Guide software.

***Physical Properties***

Several physical properties are required for the internal calculations in the EICM. For unbound materials, these are

- Specific gravity of solids $G_s$ (see Table 5-10)
- Maximum dry unit weight $\gamma_{d\ max}$ (see Table 5-13)
- Optimum gravimetric moisture content $w_{opt}$ (see Table 5-13)

Table 5-52 describes the procedures to obtain these physical property inputs for hierarchical input levels 1 and 2 (level 3 inputs are not applicable for this input category). From these properties, all other necessary weight and volume properties required in the EICM can be computed. These include

- Degree of saturation at optimum compaction ($S_{opt}$)
- Optimum volumetric moisture content ($\theta_{opt}$)
- Saturated volumetric water content ($\theta_{sat}$)

For rehabilitation designs only, the equilibrium or in-situ gravimetric water content is also a required input. NCHRP 1-37A recommends that this value be estimated from direct testing of bulk samples retrieved from the site, or through other appropriate means.

Although the material properties of the lower natural subgrade layers are important to the overall response of the pavement, a lower level of effort is generally sufficient to characterize these deeper layers as compared to the overlying compacted materials. Level 1 inputs are thus generally not necessary for in-situ subgrade materials. NCHRP 1-37A recommends that only gradation properties and Atterberg limits be measured for the in-situ subgrade materials.

**Table 5-52. Physical properties for unbound materials required for EICM calculations (NCHRP 1-37A, 2004).**

<table>
<tr><th>Material Property</th><th>Input Level</th><th>Description</th></tr>
<tr><td rowspan="3">Specific gravity, $G_s$</td><td>1</td><td>A direct measurement using AASHTO T100 (performed in conjunction with consolidation tests – T180 for bases or T 99 for other layers).<br>See Table 5-10.</td></tr>
<tr><td>2</td><td>Determined from $P_{200}$[1] and $PI$[2] of the layer as below:<br>1. Determine $P_{200}$ and $PI$.<br>2. Estimate $G_s$:<br>$G_s = 0.041(P_{200} * PI)^{0.29} + 2.65$ (5.41)</td></tr>
<tr><td>3</td><td>Not applicable.</td></tr>
<tr><td rowspan="3">Optimum gravimetric water content, $w_{opt}$, and maximum dry unit weight of solids, $(\gamma_d)_{max}$</td><td>1</td><td>Typically, AASHTO T180 compaction test for base layers and AASHTO T99 compaction test for other layers. See Table 5-13.</td></tr>
<tr><td>2</td><td>Estimated from $D_{60}$[1], $P_{200}$[1] and $PI$[2] of the layer following these steps:<br>1. Determine $PI$, $P_{200}$, and $D_{60}$.<br>2. Estimate $S_{opt}$:<br>$S_{opt} = 6.752\,(P_{200} * PI)^{0.147} + 78$ (5.42)<br>3. Estimate $w_{opt}$:<br>If $P_{200} * PI > 0$<br>$w_{opt} = 1.3\,(P_{200} * PI)^{0.73} + 11$ (5.43)<br>If $P_{200} * PI = 0$<br>$w_{opt\,(T99)} = 8.6425\,(D_{60})^{-0.1038}$ (5.44)<br>If layer is not a base course<br>$w_{opt} = w_{opt\,(T99)}$ (5.45)<br>If layer is a base course<br>$\Delta w_{opt} = 0.0156[w_{opt(T99)}]^2 - 0.1465 w_{opt(T99)} + 0.9$ (5.46)<br>$w_{opt} = w_{opt\,(T99)} - \Delta w_{opt}$ (5.47)<br>4. Determine $G_s$ using the level 2 procedure described in this table above.<br>5. Compute $(\gamma_d)_{max\,comp}$ at optimum moisture and maximum compacted density:<br>$$\gamma_{d\,max\,comp} = \frac{G_s \gamma_{water}}{1 + \frac{w_{opt} G_s}{S_{opt}}} \quad (5.48)$$<br>6. Determine $(\gamma_d)_{max}$:<br>If layer is a compacted material:<br>$\gamma_{d\,max} = \gamma_{d\,max\,comp}$ (5.49)<br>If layer is a natural in-situ material:<br>$\gamma_{d\,max} = 0.9\gamma_{d\,max\,comp}$ (5.50)</td></tr>
<tr><td>3</td><td>Not applicable.</td></tr>
</table>

[1] $P_{200}$ and $D_{60}$ can be obtained from a grain-size distribution test (AASHTO T 27)—see Table 5-19.

[2] $PI$ can be determined from an Atterberg limit test (AASHTO T 90)—see Table 5-21.

***Soil Water Characteristic Curve***

The soil water characteristic curve (SWCC) defines the relationship between water content and matric suction $h$ for a given soil. Matric suction is defined as the difference between the pore air pressure $u_a$ and pore water pressure $u_w$ in a partially saturated soil:

$$h = \left(u_a - u_w\right) \tag{5.51}$$

This relationship is usually plotted as the variation of water content (gravimetric $w$, volumetric $\theta$, or degree of saturation $S$) vs. soil suction (Figure 5-37). The SWCC is one of the primary material inputs used in the EICM to compute moisture distributions with depth and time. Although the SWCC can be measured in the laboratory (e.g., see Fredlund and Rahardjo, 1993), this is quite uncommon and rather difficult. Instead, empirical models are used to express the SWCC in terms of other, more easily measurable parameters. The EICM algorithms in the NCHRP 1-37A analysis procedure are based on a SWCC model proposed by Fredlund and Xing (1994):

$$\theta_w = C(h) \times \left[ \frac{\theta_{sat}}{\left[ \ln\left[ EXP(1) + \left( \frac{h}{a_f} \right)^{b_f} \right] \right]^{c_f}} \right] \tag{5.52}$$

$$C(h) = \left[ 1 - \frac{\ln\left(1 + \frac{h}{h_r}\right)}{\ln\left(1 + \frac{1.45 \times 10^5}{h_r}\right)} \right] \tag{5.53}$$

in which

| | | |
|---|---|---|
| in which | $h$ | = matric suction (units of stress) |
| | $\theta_{sat}$ | = volumetric moisture content at saturation |
| | $a_f$, $b_f$, $c_f$, and $h_r$ | = model parameters ($a_f$, $h_r$ in units of stress) |

Table 5-53 summarizes the NCHRP 1-37A recommended approach for estimating the parameters of the SWCC at each of the three hierarchical input levels.

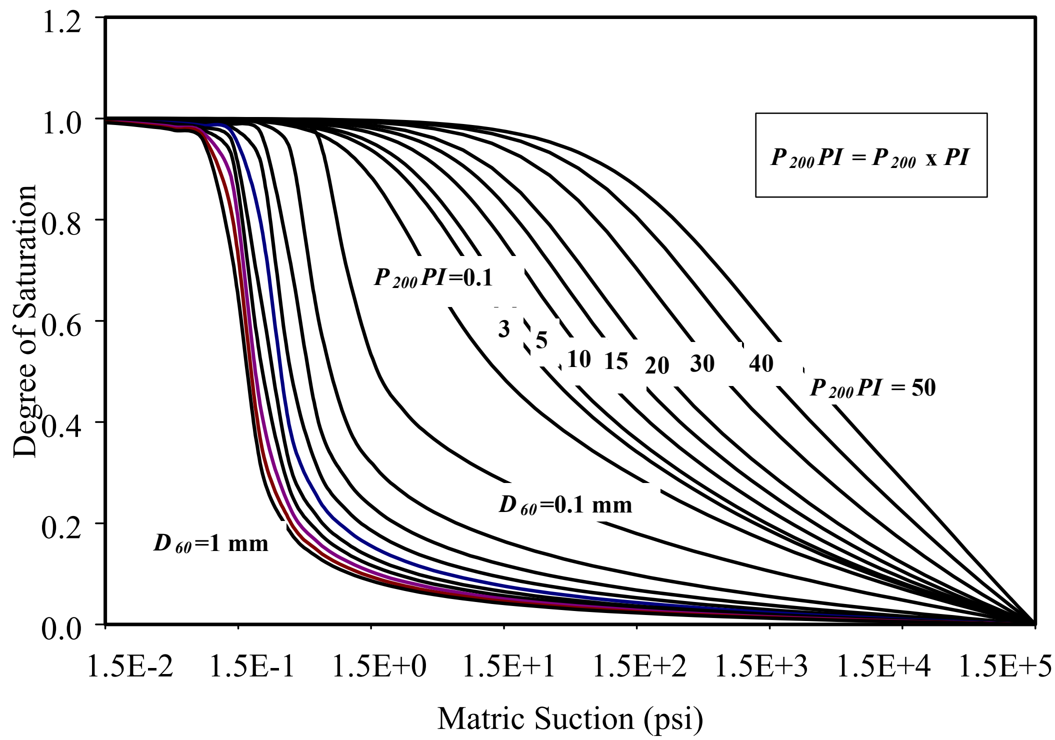



Figure 5-37. Soil water characteristic curves (NCHRP 1-37A, 2004).

***Hydraulic Conductivity (Permeability)***

Hydraulic conductivity (or permeability) $k$ describes the ability of a material to conduct fluid (water). It is defined as the quantity of fluid flow through a unit area of soil under a unit pressure gradient. Hydraulic conductivity is one of the primary material inputs to the environment model in the NCHRP 1-37A analysis procedure, where it is used to determine the transient moisture profiles in unbound materials and to estimate their drainage characteristics.

The unsaturated flow algorithms in the EICM require a complete specification of the unsaturated hydraulic conductivity as a function of matric suction $h$. Although the unsaturated hydraulic conductivity vs. matric suction relationship can be measured in the laboratory (*e.g.,* see Fredlund and Rahardjo, 1993), this is uncommon and difficult. At best, only the saturated hydraulic conductivity $k_{sat}$ is measured in practice. Consequently, within the EICM an empirical model proposed by Fredlund *et al.* (1994) is used to express the unsaturated hydraulic conductivity $k(h)$ vs. matric suction $h$ relationship in terms of the Fredlund and Xing (1994) SWCC model, Eqs. (5.52) and (5.53). The $k(h)$ model is expressed in terms of a relative hydraulic conductivity:

$$k_r(h) \equiv k(h)/k_{sat} \qquad \textbf{(5.54)}$$

**Table 5-53. Options for estimating the SWCC parameters (NCHRP 1-37A, 2004).**

| Input Level | Procedure to Determine SWCC parameters | Required Testing |
|---|---|---|
| 1 | 1) Direct measurement of suction ($h$) in psi, and volumetric water content ($\theta_w$) pairs of values.<br>2) Direct measurement of optimum gravimetric water content, $w_{opt}$ and maximum dry unit weight, $\gamma_{d\,max}$.<br>3) Direct measurement of the specific gravity of the solids, $G_s$.<br>4) Compute $\theta_{opt} = \dfrac{w_{opt}\gamma_{d\,max}}{\gamma_{water}}$ (5.55)<br>5) Compute $S_{opt} = \dfrac{\theta_{opt}}{1 - \dfrac{\gamma_{d\,max}}{\gamma_{water} G_s}}$ (5.56)<br>6) Compute $\theta_{sat} = \dfrac{\theta_{opt}}{S_{opt}}$ (5.57)<br>7) Using non-linear regression analysis, compute the SWCC model parameters $a_f$, $b_f$, $c_f$, and $h_r$ from Eqs. (5.52) and (5.53) and the ($h$, $\theta_w$) pairs of values obtained in Step 1. | Pressure plate, filter paper, and/or Tempe cell testing.<br>AASHTO T180 or T99 for $\gamma_{d\,max}$ (see Table 5-13).<br>AASHTO T100 for $G_s$ (see Table 5-10). |
| 2 | 1) Direct measurement of optimum gravimetric water content, $w_{opt}$ and maximum dry unit weight, $\gamma_{d\,max}$.<br>2) Direct measurement of the specific gravity of the solids, $G_s$.<br>3) Direct measurement of $P_{200}$, $D_{60}$, and $PI$.<br>The EICM will then internally do the following:<br>a) Calculate $P_{200} * PI$.<br>b) Calculate $\theta_{opt}$, $S_{opt}$, and $\theta_{sat}$, as described for level 1.<br>c) Determine the SWCC model parameters $a_f$, $b_f$, $c_f$, and $h_r$ in Eqs. (5.52) and (5.53) via correlations with $P_{200}PI$ and $D_{60}$.<br><br>If $P_{200}\ PI > 0$<br>$a_f = \dfrac{0.00364(P_{200}PI)^{3.35} + 4(P_{200}PI) + 11}{6.895}$ (psi) (5.58)<br>$\dfrac{b_f}{c_f} = -2.313(P_{200}PI)^{0.14} + 5$ (5.59)<br>$c_f = 0.0514(P_{200}PI)^{0.465} + 0.5$ (5.60)<br>$\dfrac{h_r}{a_f} = 32.44e^{0.0186(P_{200}PI)}$ (5.61)<br><br>If $P_{200}\ PI = 0$<br>$a_f = \dfrac{0.8627(D_{60})^{-0.751}}{6.895}$ (psi) (5.62)<br>$b_f = 7.5$ (5.63) | AASHTO T180 or T99 for $\gamma_{d\,max}$ (see Table 5-13).<br>T100 for $G_s$ (see Table 5-10).<br>AASHTO T88 for $P_{200}$ and $D_{60}$ (see Table 5-19).<br>AASTHO T90 for $PI$ (see Table 5-21). |

| | | |
|---|---|---|
| | $c_f = 0.1772\ln(D_{60}) + 0.7734$ (5.64)<br>$\frac{h_r}{a_f} = \frac{1}{D_{60} + 9.7e^{-4}}$ (5.65) | |
| 3 | Direct measurement and input of $P_{200}$, $PI$, and $D_{60}$, after which the EICM uses correlations with $P_{200}PI$ and $D_{60}$ to automatically generate the SWCC parameters for each soil as follows:<br>1) Compute $G_s$, as outlined in Table 5-52 for level 2.<br>2) Compute $P_{200}$ * $PI$<br>3) Estimate $S_{opt}$, $w_{opt}$, and $\gamma_{d\,max}$, as shown Table 5-52 for level 2.<br>4) Determine the SWCC model parameters $a_f$, $b_f$, $c_f$, and $h_r$ via correlations with $P_{200}PI$ and $D_{60}$, as shown in this table for level 2. | AASHTO T88 for $P_{200}$ and $D_{60}$ (see Table 5-19).<br>AASHTO T90 for $PI$ (see Table 5-21). |

Recommendations from NCHRP 1-37A for determining the $k_{sat}$ value needed in Eq. (5.54) are summarized in Table 5-54. The Fredlund *et al.* (1994) model for $k_r(h)$ is then expressed in integral form as

$$k_r(h) = \frac{\int_h^{h_r} \frac{\theta(x) - \theta(h)}{x^2}\theta'(x)dx}{\int_{h_{aev}}^{h_r} \frac{\theta(x) - \theta_s}{x^2}\theta'(x)dx} \tag{5.66}$$

in which:

$\theta(h)$ = volumetric water content as a function of matric suction, from the SWCC Eqs. (5.52) and (5.53)

$\theta_s$ = saturated volumetric water content

$\theta'(h)$ = derivative of the SWCC

$x$ = dummy integration variable corresponding to water content

$h_r$ = matric suction corresponding to the residual water content (i.e., the water content below which a large increase in suction is required to remove additional water)

$h_{ave}$ = the air-entry matric suction (i.e., the suction where air starts to enter the largest pores in the soil)

The procedures described in Table 5-53 are used in the EICM to determine the SWCC via Eqs. (5.52) and (5.53), which in turn is then used to determine the unsaturated hydraulic conductivity via Eq. (5.66). These calculations are performed internally within the EICM software.

**Table 5-54. Options for determining the saturated hydraulic conductivity for unbound materials (NCHRP 1-37A, 2004).**

| Material Property | Input Level | Description |
|---|---|---|
| Saturated hydraulic conductivity, $k_{sat}$ | 1 | Direct measurement using a permeability test (AASHTO T215)—see Table 5-55. |
| | 2 | Determined from $P_{200}$[1], $D_{60}$[1], and PI[2] of the layer as below:<br>1. Determine $P_{200}PI = P_{200} * PI$<br>2. If $0 \le P_{200}PI < 1$<br>$k_{sat} = 118.11 \times 10^{\left[-1.1275(\log D_{60}+2)^2+7.2816(\log D_{60}+2)-11.2891\right]}$ (5.67)<br>Units: ft/hr<br>Valid for $D_{60} < 0.75$ in<br>If $D_{60} > 0.75$ in, set $D_{60} = 0.75$ mm<br>3. If $P_{200}PI \ge 1$<br>$k_{sat} = 118.11 \times 10^{\left[0.0004(P_{200}PI)^2-0.0929(P_{200}PI)-6.56\right]}$ (ft/hr) (5.68) |
| | 3 | Not applicable. |

[1] $P_{200}$ and $D_{60}$ can be obtained from a grain-size distribution test (Table 5-19)

[2] *PI* can be determined from an Atterberg limit test (Table 5-21).

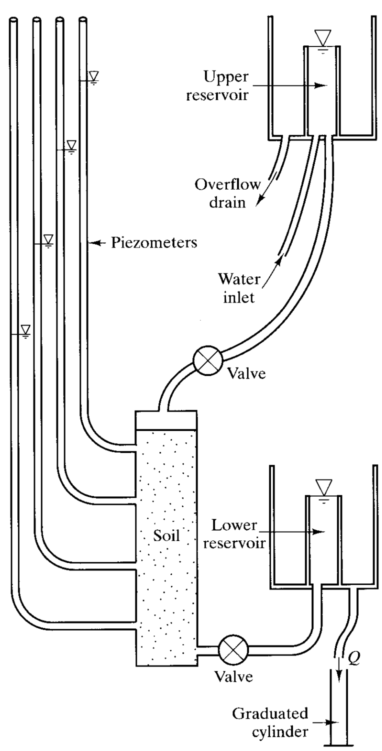



Figure 5-38. Schematic of a constant head permeameter (Coduto, 1999).

**Table 5-55. Saturated hydraulic conductivity.**

| | |
|---|---|
| Description | Quantity of fluid flow through a unit area of soil under a unit pressure gradient. |
| Uses in Pavements | Used in the EICM for predicting distributions of moisture with depth and time in the NCHRP 1-37A Design Guide. |
| Laboratory Determination | AASHTO T 215; ASTM D 2434 (Granular Soils), ASTM D 5084 (All Soils). There are two basic standard types of test procedures to directly determine permeability: (1) the constant-head test, normally used for coarse materials (Figure 5-38); and (2) the falling-head test, normally used for clays (Figure 5-39). Undisturbed, remolded, or compacted samples can be used in both procedures. |
| Field Measurement | Pumping tests can be used to measure hydraulic conductivity in-situ. |
| Commentary | Both test procedures determine permeability of soils under specified conditions. The geotechnical engineer must establish which test conditions are representative of the problem under consideration. As with all other laboratory tests, the geotechnical engineer has to be aware of the limitations of this test. The process is sensitive to the presence of air or gases in the voids and in the permeant or water. Prior to the test, distilled, de-aired water should be run through the specimen to remove as much of the air and gas as practical. It is a good practice to use de-aired or distilled water at temperatures slightly higher than the temperature of the specimen. As the water permeates through the voids and cools, it will have a tendency to dissolve the air and some of the gases, thus removing them during this process. The result will be a more representative, albeit idealized, permeability value.<br>The type of permeameter, (*i.e.,* flexible wall - ASTM D 5084 -versus rigid - ASTM D 2434 and AASHTO T215) may also affect the final results. For testing of fine-grained low-permeability soils, the use of flexible-wall permeameters is recommended, which are essentially very similar to the triaxial test apparatus. When rigid wall units are used, the permeant may find a route at the sample-permeameter interface. This will produce erroneous results.<br>It should be emphasized that permeability is sensitive to viscosity. In computing permeability, correction factors for viscosity and temperatures must be applied. The temperature of the permeant and the laboratory should be kept constant during testing.<br>Laboratory permeability tests produce reliable results under ideal conditions. Permeability of fine-grained soils can also be computed from one-dimensional consolidation test results, although these results are not as accurate as direct $k_{sat}$ measurements. |
| Typical Values | See Table 5-56 and Table 5-57. Saturated hydraulic conductivity for loose clean sands can also be estimated using the Hazen relationship:<br>$k_{sat} = C * D_{10}^2 \qquad (5.69)$<br>in which $k_{sat}$ is the saturated hydraulic conductivity in cm/sec; $C$ is Hazen's coefficient ranging between 0.8 and 1.2 (a value of 1.0 is commonly used); and $D_{10}$ is the effective particle size, defined as the largest particle diameter in the finest 10% fraction of the soil. |

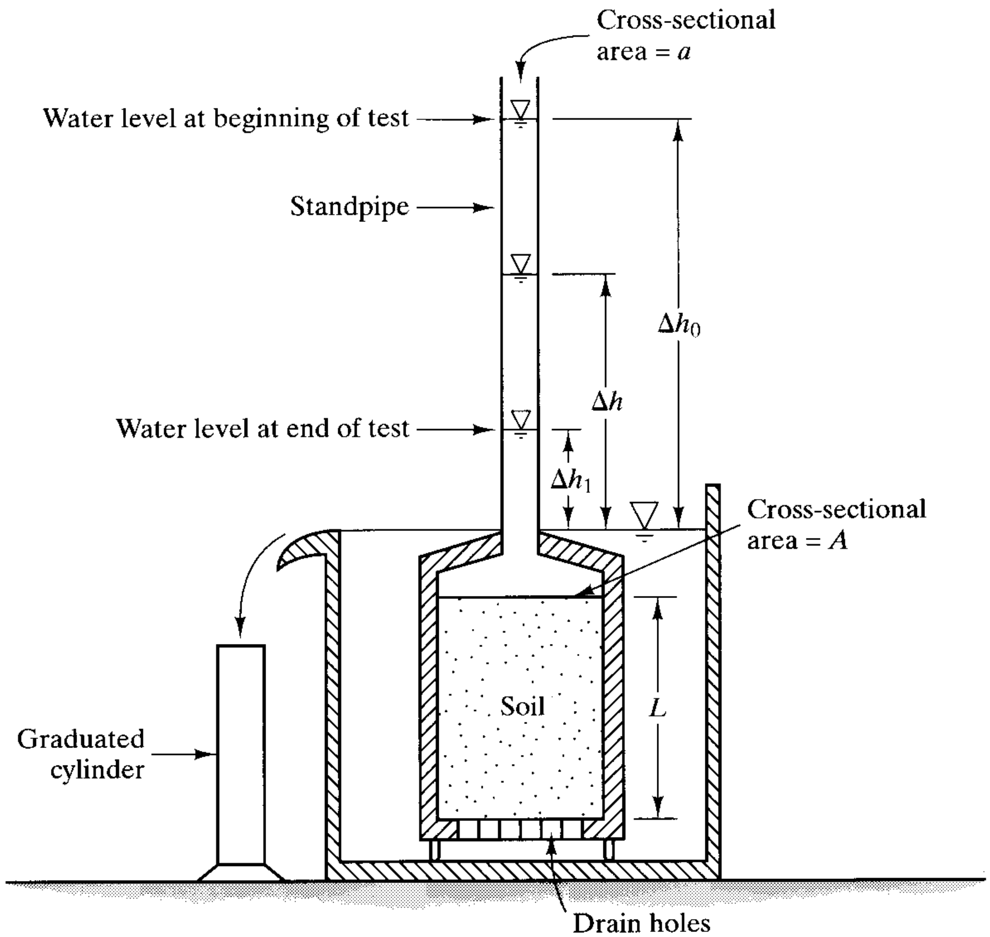


Figure 5-39. Schematic of a falling head permeameter (Coduto, 1999).

**Table 5-56. Typical values of saturated hydraulic conductivity for soils (Coduto, 1999).**

| Soil Description | Hydraulic Conductivity $k$ (cm/s) | Hydraulic Conductivity $k$ (ft/s) |
|---|---|---|
| Clean gravel | 1 – 100 | $3\text{x}10^{-2}$ – 3 |
| Sand-gravel mixtures | $10^{-2}$ – 10 | $3\text{x}10^{-4}$ – 0.3 |
| Clean coarse sand | $10^{-2}$ – 1 | $3\text{x}10^{-4}$ – $3\text{x}10^{-2}$ |
| Fine sand | $10^{-3}$ – $10^{-1}$ | $3\text{x}10^{-5}$ – $3\text{x}10^{-3}$ |
| Silty sand | $10^{-3}$ – $10^{-2}$ | $3\text{x}10^{-5}$ – $3\text{x}10^{-4}$ |
| Clayey sand | $10^{-4}$ – $10^{-2}$ | $3\text{x}10^{-6}$ – $3\text{x}10^{-4}$ |
| Silt | $10^{-8}$ – $10^{-3}$ | $3\text{x}10^{-10}$ – $3\text{x}10^{-5}$ |
| Clay | $10^{-10}$ – $10^{-6}$ | $3\text{x}10^{-12}$ – $3\text{x}10^{-8}$ |

**Table 5-57. Typical values of saturated hydraulic conductivity for highway materials (Carter and Bentley, 1991).**

| Material | Hydraulic Conductivity $k$ (m/s)* |
|---|---|
| Uniformly graded coarse aggregate | $0.4 - 4\text{x}10^{-3}$ |
| Well-graded aggregate without fines | $4\text{x}10^{-3} - 4\text{x}10^{-5}$ |
| Concrete sand, low dust content | $7\text{x}10^{-4} - 7\text{x}10^{-6}$ |
| Concrete sand, high dust content | $7\text{x}10^{-6} - 7\text{x}10^{-8}$ |
| Silty and clayey sands | $10^{-7} - 10^{-9}$ |
| Compacted silt | $7\text{x}10^{-8} - 7\text{x}10^{-10}$ |
| Compacted clay | $< 10^{-9}$ |
| Bituminous concrete** | $4\text{x}10^{-5} - 4\text{x}10^{-8}$ |
| Portland cement concrete | $< 10^{-10}$ |

*1 m/s = 3.25 ft/s

**New pavements; values as low as $10^{-10}$ have been reported for sealed, traffic-compacted highway pavements.

***Thermal Conductivity***

Thermal conductivity $K$ is defined as the ability of a material to conduct heat. Typical units are BTU/ft-hr-$^{o}$F or W/m- $^{o}$K. Thermal conductivity is used in the EICM algorithms for the computation of temperature distributions with depth and time in the NCHRP 1-37A analysis methodology.

Table 5-58 outlines the NCHRP 1-37A recommended approach for characterizing the dry thermal conductivity $K$ for unbound materials. Note that thermal conductivity is not commonly measured for unbound pavement materials, and consequently the level 3 inputs will be used for nearly all designs. The EICM automatically adjusts the dry thermal conductivity for the influence of moisture during the calculations.

***Heat Capacity***

Heat capacity $Q$ is defined as the amount of heat required to raise by one degree the temperature of a unit mass of soil. Typical units are BTU/lb-$^{o}$F or J/kg-$^{o}$K. Heat capacity is used in the EICM algorithms for the computation of temperature distributions with depth and time in the NCHRP 1-37A analysis methodology.

Table 5-58 outlines the NCHRP 1-37A recommended approached for characterizing the dry heat capacity *Q* for unbound materials. Note that heat capacity is not commonly measured for unbound pavement materials, and consequently the level 3 inputs will be used for nearly all designs. The EICM automatically adjusts the dry heat capacity for the influence of moisture content during the calculations.

**Table 5-58. Options for determining the dry thermal conductivity and heat capacity for unbound materials (NCHRP 1-37A, 2004).**

<table>
<tr><th>Material Property</th><th>Input Level</th><th colspan="3">Description</th></tr>
<tr><td rowspan="3">Dry Thermal Conductivity, K</td><td>1</td><td colspan="3">Direct measurement (ASTM E 1952).</td></tr>
<tr><td>2</td><td colspan="3">Not applicable.</td></tr>
<tr><td>3</td><td colspan="3">
<table>
<tr><th><u>Soil Type</u></th><th><u>Range</u></th><th>Recommended<br><u>BTU/ft-hr-°F*</u></th></tr>
<tr><td>A-1-a</td><td>0.22 – 0.44</td><td>0.30</td></tr>
<tr><td>A-1-b</td><td>0.22 – 0.44</td><td>0.27</td></tr>
<tr><td>A-2-4</td><td>0.22 – 0.24</td><td>0.23</td></tr>
<tr><td>A-2-5</td><td>0.22 – 0.24</td><td>0.23</td></tr>
<tr><td>A-2-6</td><td>0.20 – 0.23</td><td>0.22</td></tr>
<tr><td>A-2-7</td><td>0.16 – 0.23</td><td>0.20</td></tr>
<tr><td>A-3</td><td>0.25 – 0.40</td><td>0.30</td></tr>
<tr><td>A-4</td><td>0.17 – 0.23</td><td>0.22</td></tr>
<tr><td>A-5</td><td>0.17 – 0.23</td><td>0.19</td></tr>
<tr><td>A-6</td><td>0.16 – 0.22</td><td>0.18</td></tr>
<tr><td>A-7-5</td><td>0.09 – 0.17</td><td>0.13</td></tr>
<tr><td>A-7-6</td><td>0.09 – 0.17</td><td>0.12</td></tr>
</table>
Additional typical values are given in Table 5-59.</td></tr>
<tr><td rowspan="3">Dry Heat Capacity, Q</td><td>1</td><td colspan="3">Direct measurement (ASTM D 2766).</td></tr>
<tr><td>2</td><td colspan="3">Not applicable.</td></tr>
<tr><td>3</td><td colspan="3">Typical values range from 0.17 to 0.20 BTU/lb-°F. Additional typical values are given in Table 5-59.</td></tr>
</table>

*1 BTU/ft-hr-°F = 1.73 W/m-°K; 1 BTU/lb-°F = 4187 J/kg-°K

**Table 5-59. Typical values for thermal conductivity and heat capacity of unbound materials (adapted from Sundberg, 1988).**

| Soil Type | Thermal Conductivity (W/m-°K)* | Heat Capacity (J/kg-°K)* |
|---|---|---|
| Clay with high clay content | 0.85 - 1.1 | 1700 – 2050 |
| Silty clay/silt | 1.1 - 1.5 | 1650 – 1900 |
| Silt | 1.2 – 2.4 | 1400 – 1900 |
| Sand, gravel below GWT | 1.5 – 2.6<br>(1.6 – 2.0) | 1450 – 1850<br>(1700) |
| Sand, gravel above GWT | 0.4 – 1.1<br>(0.7 – 0.9) | 700 – 1000<br>(800) |
| Till below GWT | 1.5 – 2.5 | 1350 – 1700 |
| Sandy till above GWT | 0.6 – 1.8 | 750 – 1100 |
| Peat below GWT | 0.6 | 2300 |
| Peat above GWT | 0.2 – 0.5 | 400 - 1850 |

*1 W/m-°K = 0.578 BTU/ft-hr-°F; 1 J/kg-°K = 2.388E-4 BTU/lb-°F

## 5.6 ENVIRONMENT/CLIMATE INPUTS

### 5.6.1 1993 AASHTO Guide

There are only four environmental inputs in the 1993 AASHTO Guide:

- Estimated seasonal variation of the subgrade resilient modulus $M_R$ (Section 5.4.3)
- The category for the percentage of time that the unbound pavement materials are exposed to moisture conditions near saturation (Section 5.5.1)
- The qualitative description of moisture supply for expansive subgrades (Section 5.5.1)
- The depth of frost penetration (Section 5.5.1)

These environmental factors are intertwined with their associated material property inputs and have already been described in this chapter in the sections noted above.

### 5.6.2 NCHRP 1-37A Design Guide

Three sets of environmental inputs are required in the NCHRP 1-37A design methodology:

- Climate, defined in terms of histories of key weather parameters
- Groundwater depth

- Surface shortwave absorptivity

These parameters are the inputs/boundary conditions for the calculation of climate-specific temperature and moisture distributions with depth and time in the EICM (see Appendix D). These distributions, in turn, are used to determine seasonal moisture contents and freeze-thaw cycles for the unbound pavement materials.

***Climate Inputs***

The seasonal damage and distress accumulation algorithms in the NCHRP 1-37A design methodology require hourly history data for five weather parameters:

- Air temperature
- Precipitation
- Wind speed
- Percentage sunshine (used to define cloud cover)
- Relative humidity.

The NCHRP 1-37A Design Guide recommends that the weather inputs be obtained from weather stations located near the project site. At least 24 months of actual weather station data are required for the computations. The Design Guide software includes a database of appropriate weather histories from nearly 800 weather stations throughout the United States. This database is accessed by specifying the latitude, longitude, and elevation of the project site. The Design Guide software locates the six closest weather stations to the site; the user selects a subset of these to create a virtual project weather station via interpolation of the climatic data from the selected physical weather stations.

Specification of the weather inputs is identical at all the three hierarchical input levels in the NCHRP 1-37A Design Guide.

***Groundwater Depth***

The groundwater table depth is intended to be the best estimate of the annual average depth. Level 1 inputs are based on soil borings, while level 3 inputs are simple estimates of the annual or seasonal average values. A potential source for level 3 groundwater depth estimates is the county soil reports produced by the National Resources Conservation Service. There is no level 2 approach for this design input.

It is important to recognize that groundwater depth can play a significant role in the overall accuracy of the foundation/pavement moisture contents and, hence, the seasonal modulus values. This is explored further in Chapter 6. Every attempt should be made to characterize groundwater depth as accurately as possible.

***Surface Shortwave Absorptivity***

This last environmental input is a property of the AC or PCC surface layer. The dimensionless surface short wave absorptivity defines the fraction of available solar energy that is absorbed by the pavement surface. It depends on the composition, color, and texture of the surface layer. Generally speaking, lighter and more reflective surfaces tend to have lower short wave absorptivity.

The NCHRP 1-37A recommendations for estimating surface shortwave absorptivity at each hierarchical input level are as follows:

- Level 1—Determined via laboratory testing. However, although laboratory procedures exist for measuring shortwave absorptivity, there currently are no AASHTO protocols for this for paving materials.
- Level 2—Not applicable.
- Level 3—Default values as follows:
  - Weathered asphalt (gray) 0.80 – 0.90
  - Fresh asphalt (black) 0.90 – 0.98
  - Aged PCC layer 0.70 – 0.90

Given the lack of suitable laboratory testing standards, level 3 values will typically be used for this design input.

## 5.7 DEVELOPMENT OF DESIGN INPUTS

Myth has it that an unknown structural engineer offered the following definition of his profession (Coduto, 2001):

> *"Structural engineering is the art and science of molding materials we do not fully understand into shapes we cannot precisely analyze to resist forces we cannot accurately predict, all in such a way that the society at large is given no reason to suspect the extent of our ignorance."*

This definition applies even more emphatically to pavement engineering. In spite of our many technical advances, there are still great gaps in our understanding. Often the greatest uncertainties in an individual project are with site conditions and materials—the types and conditions of materials encountered along the highway alignment, their spatial, temporal, and inherent variability, and their complex behavior under repeated traffic loading and environmental cycles.

Site investigation and testing programs often generate large amounts of data that can be difficult to synthesize. Real soil profiles are nearly always very complex, so borings often do not correlate and results from different tests may differ enormously. The development of a simplified representation of the soils and geotechnical conditions at a project site requires much interpolation and extrapolation of data, combined with sound engineering judgment. But what is engineering judgment? Ralph Peck suggested several alternative definitions (Dunnicliff and Deere, 1984):

> *"To the engineering student, judgment often appears to be an ingredient said to be necessary for the solution of engineering problems, but one that the student can acquire only later in his career by some undefined process of absorption from his experience and his colleagues.*
>
> *"To the engineering scientist, engineering judgment may appear to be a crutch used by practicing engineers as a poor substitute for sophisticated analytical procedures.*
>
> *"To the practicing engineer, engineering judgment may too often be an impressive name for guessing rather than for rational thinking."*

Perhaps *Webster's New Collegiate Dictionary* offers the definitive statement:

> **Judgment:** The operation of the mind, involving comparison and discrimination, by which knowledge of values and relations is mentally formulated.

But when confronted with voluminous quantities of inconsistent—and often contradictory—information, how does the pavement engineer compare and discriminate? What tools (or tricks) of the trade are available? This is a difficult process to describe. However, some common techniques for determining design values from site exploration and other geotechnical data are as follows:

- Find and remove any obvious outliers in the data. Although there are statistical techniques for doing this (*e.g.,* McCuen, 1993), in practice, detailed knowledge of the data plus engineering reasoning is usually sufficient for removing data outliers for cause. Table 5-60 summarizes some typical ranges of variability for pavement design inputs; additional information on measured variability of geotechnical parameters can be found in Baecher and Christian (2003). However, it is important that outliers (*e.g.,* a single low stiffness value) not be arbitrarily removed without fully evaluating the data for an explanation. A local anomaly may exist in the field, for example, that requires remediation.

- Examine spatial (and in some cases, temporal) trends in the data. Look at both the subsurface stratagraphic profiles and plan view "map" of subsurface conditions. Refer to the 1993 AASHTO Design Guide for resolving spatial variations in pavement design data by defining homogeneous analysis units based on a "cumulative difference" approach (Figure 5-40). A separate set of design inputs can then be developed for each homogeneous analysis unit, reducing the variability of measured vs. design input values within each unit.

- Check whether the magnitudes and trends in the data pass the test of "engineering reasonableness" – *e.g.,* are the values of the right order of magnitude? Are the trends in the data in the intuitively correct directions?

- Examine the internal consistency of the data – *e.g.,* are the phase relationships by volume consistent with the phase relationships by weight?

- Use correlations among different types of data to strengthen data interpretation – *e.g.,* statistical correlations between resilient modulus and CBR can be used to supplement a limited set of measured $M_R$ values (although today, laboratory resilient modulus tests can often be performed more quickly and less expensively than laboratory CBR tests—see Table 5-61).

- Be clear on what is needed for a design value. The value of a material property used for specification purposes may be different from the value of that same material property when used for design. For example, a conservative value (mean plus one or two standard deviations) may be specified for the minimum compressive strength of a lime stabilized subgrade for construction quality control specifications; the mean value would be more appropriate for design applications where overall reliability (*e.g.,* factor of safety) is considered explicitly, as is the case in both the AASHTO and NCHRP design procedures.

- Evaluate the sensitivity of the design to the inputs! This is perhaps the most important—and often the most overlooked—aspect of design. Evaluating sensitivity to design inputs can have several benefits. First, it will categorize which inputs are most important and which are less important to the design. There is no need to expend large effort determining the precise design values for inputs that have little impact on the final outcome. More resources can then be allocated to determining the inputs that have significant impact on the outcome once they have been identified. Second, design sensitivity analyses can indicate the potential consequences of incorrect judgments of the design inputs. For example, if the subgrade resilient modulus is underestimated by 50%, will this reduce the expected useful life of the pavement by 1 year or 10 years? How does the increased cost of reduced pavement life compare with the cost of additional exploration in order to establish the subgrade resilient modulus value more robustly?

- When in doubt run more tests (a single test is often worth a thousand guesses).

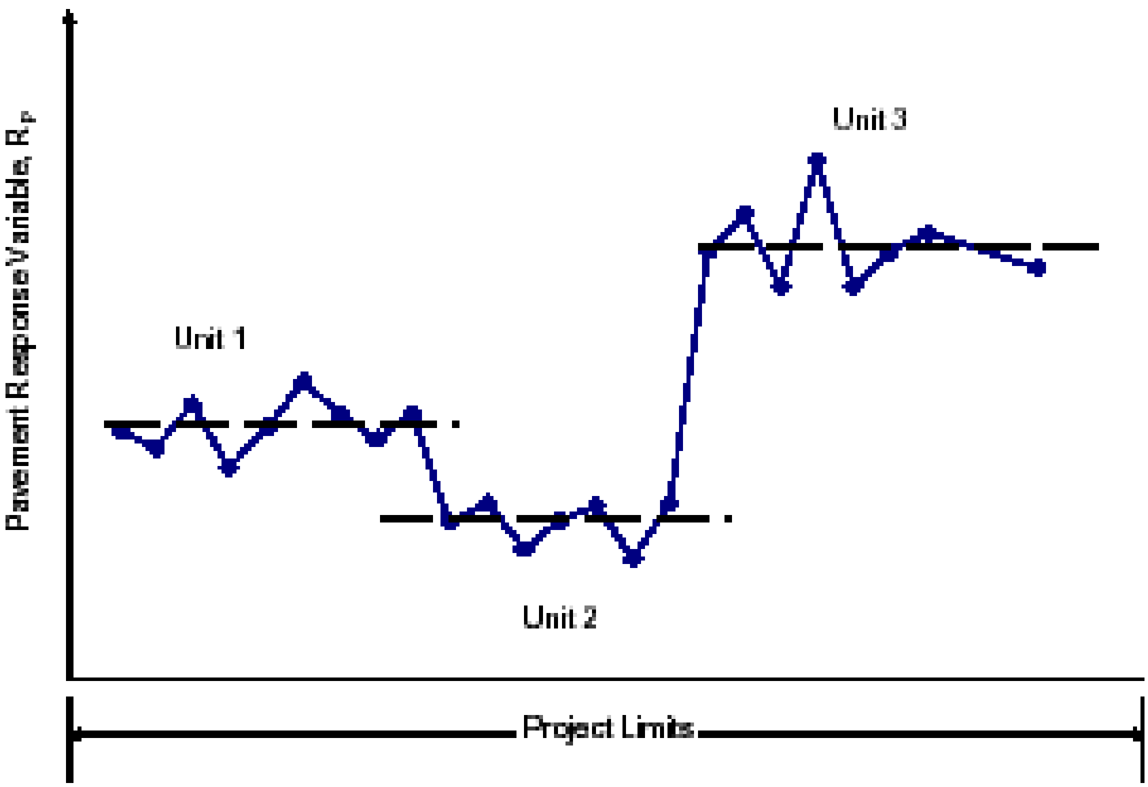



Figure 5-40. Variation of pavement response variable versus distance for given project (NCHRP 1-37A, 2004).

**Table 5-60. Summary of typical pavement parameter variability (AASHTO, 1993).**

| Property | Standard Deviation | | |
|---|---|---|---|
| | Low | Average | High |
| Thickness (inches) | | | |
| Portland cement concrete | 0.1 | 0.3 | 0.5 |
| Asphalt concrete | 0.3 | 0.5 | 0.7 |
| Cement treated base | 0.5 | 0.6 | 0.7 |
| Granular base | 0.6 | 0.8 | 1.0 |
| Granular subbase | 1.0 | 1.2 | 1.5 |
| Strength | | | |
| CBR (%) | | | |
| Subgrade (4 – 7) | 0.5 | 1.0 | 2.0 |
| Subgrade (7 – 13) | 1.0 | 1.5 | 2.5 |
| Subgrade (13 – 20) | 2.5 | 4.0 | 6.0 |
| Granular subbase (20 – 30) | 5.0 | 8.0 | 12.0 |
| Granular base (80+) | 10.0 | 15.0 | 30.0 |
| Percent compaction (%) | | | |
| Embankment/subgrade | 2.0 | 4.5 | 7.0 |
| Subbase/base | 2.0 | 2.8 | 3.5 |
| Portland cement concrete properties | | | |
| Air content (%) | 0.6 | 1.0 | 1.5 |
| Slump (inches) | 0.6 | 1.0 | 1.4 |
| 28-day compressive strength (psi) | 400 | 600 | 800 |
| Asphalt concrete properties | | | |
| Gradation (%) | | | |
| 3/4 or 1/2 inch | 1.5 | 3.0 | 4.5 |
| 3/8 inch | 2.5 | 4.0 | 6.0 |
| No. 4 | 3.2 | 3.8 | 4.2 |
| No. 40 or No. 50 | 1.3 | 1.5 | 1.7 |
| No. 200 | 0.8 | 0.9 | 1.0 |
| Asphalt content (%) | 0.1 | 0.25 | 0.4 |
| Percent compaction (%) | 0.75 | 1.0 | 1.5 |
| Marshall mix properties | | | |
| Stability (lbs) | 200 | 300 | 400 |
| Flow (in./in.) | 1.0 | 1.3 | 2.0 |
| Air voids (%) | 0.8 | 1.0 | 1.4 |
| AC consistency | | | |
| Pen @ 77$^o$F | 2 | 10 | 18 |
| Viscosity @ 149$^o$F (kilopoise) | 2 | 25 | 100 |
| | Coefficient of Variation (%) | | |
| | Low | Average | High |
| Pavement deflection | 15 | 30 | 45 |

**Table 5-61. Resilient modulus versus CBR testing for fine grained subgrade soil (Boudreau Engineering, 2004, personal communication).**

| Property | CBR | Resilient Modulus |
|---|---|---|
| Sample size required | 60 lbs (27 kg) | 5 lbs (2.3 kg) |
| Turnaround time | 10 days | 4 days |
| Data value | Empirical | Mechanistic |
| In-situ testing | Field test | Shelby tube - lab |
| Unit price | \$365 | \$300 |

## 5.8 EXERCISES

Depending upon the number of groups in the class, one or more of the following exercises may be assigned.

### 5.8.1 1993 AASHTO Design Guide—Flexible Pavements

Small group exercise: Given the pavement information for the Main Highway in Appendix B, estimate appropriate material property inputs for the unbound materials in a flexible pavement structure as required for the 1993 AASHTO Design Guide. (A worksheet will be distributed to guide this exercise.)

### 5.8.2 1993 AASHTO Design Guide—Rigid Pavements

Small group exercise: Given the pavement information for the Main Highway in Appendix B, estimate appropriate material property inputs for the unbound materials in a rigid pavement structure as required by the 1993 AASHTO Design Guide. (A worksheet will be distributed to guide this exercise.)

## 5.9 REFERENCES


AASHTO (1972). *AASHTO Interim Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (1986). *Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (1993). *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (2003). *Standard Specifications for Transportation Materials and Methods of Sampling and Testing (23rd ed.)*, American Association of State Highway and Transportation Officials, Washington, D.C.

Andrei, D. (1999). *Development of a Harmonized Test Protocol for the Resilient Modulus of Unbound Materials Used in Pavement Design*, Master of Science thesis, University of Maryland, College Park, MD.

Andrei, D. (2003). *Development of a Predictive Model for the Resilient Modulus of Unbound Materials*, Ph.D. dissertation, Arizona State University, Tempe, AZ.

Asphalt Institute (1982). "Research and Development of the Asphalt Institute's Thickness Design Manual, Ninth Edition," *Research Report No. 82-2*.

Baecher, G.B., and Christian, J.T. (2003). *Reliability and Statistics in Geotechnical Engineering*, John Wiley and Sons, New York, NY.

Barksdale, R.D., Alba, J., Khosla, P.N., Kim, R., Lambe, P.C. and Rahman, M.S. (1996). *Laboratory Determination of Resilient Modulus for Flexible Pavement Design*, Final Report, NCHRP Project 1-28, National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, D.C.

Barksdale, R.D. (ed.) (2000). *The Aggregate Handbook Plus Supplement*, National Stone Association, Washington, D.C.

Boudreau Engineering (2004) personal communication.

Burmister, D.M. (1943). "The Theory of Stresses and Displacements in Layered Systems and Applications to the Design of Airport Runways," *Highway Research Board*, Vol. 23, pp. 126-144.

Carter, M., and Bentley, S.P. (1991). *Correlations of Soil Properties*, Pentech Press, London.

Cheney, R.S., and Chassie, R.G. (1993). *Soils and Foundations Workshop Manual.* Federal Highway Administration, Washington, D.C.

Coduto, D.P. (2001). *Foundation Design Principles and Practices (2nd ed.)*, Prentice-Hall, Englewood Cliffs, NJ.

Coduto, D.P. (1999). *Geotechnical Engineering Principles and Practices*, Prentice-Hall, Englewood Cliffs, NJ.

Dunnicliff, D., and Deere, D.U. (1984). *Judgment in Geotechnical Engineering: The Professional Legacy of Ralph B. Peck*, John Wiley and Sons, New York, NY.

Fredlund, D.G, and Rahardjo, H. (1993). *Soil Mechanics for Unsaturated Soils*, John Wiley and Sons, New York, NY.

Fredlund, D.G., and Xing, A. (1994). "Equations for the Soil Water Characteristic Curve," *Canadian Geotechnical Journal*, Vol. 31, No. 4, pp. 521-532.

Fredlund, D.G., Xing, A. and Huang, S. (1994). "Predicting the Permeability Function for Unsaturated Soils Using the Soil-Water Characteristic Curve," *Canadian Geotechnical Journal*, Vol 3., No. 4, pp. 533 - 546.

Heukelom, W., and Klomp, A.J.G. (1962). "Dynamic Testing as a Means of Controlling Pavements During and After Construction," *Proceedings*, First International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, MI.

Holtz, W.G., and Gibbs, H.J. (1956). "Engineering Properties of Expansive Clays," *Transactions ASCE*, Vol. 121, pp. 641-677.

Huang, Y.H. (1993). *Pavement Design and Analysis*, Prentice-Hall, Englewood Cliffs, NJ.

LTTP (2003). *Guide for Determining Design Resilient Modulus Values for Unbound Materials*, interactive CD ROM (Version 1.1), Long Term Pavement Performance Program, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.

Mallela, J., Larson, G.E., Wyatt, T., Hall, J.P. and Barker W. (2002). *User's Guide for Drainage Requirements in Pavements – DRIP 2.0 Microcomputer Program*, FHWA Contract No. DTFH61-00-F-00199, July.

Mayne, P.W. and Kulhawy, F.H. (1982). "$K_o$-OCR Relationships in Soil," *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 108, GT6, pp. 851-872.

McCarthy, D.F. (2002). *Essentials of Soil Mechanics and Foundations*, 6th Edition, Prentice-Hall, Englewood Cliffs, NJ.

McCuen, R.H. (1993). *Microcomputer Applications in Statistical Hydrology*, Prentice-Hall, Inc., Englewood Cliffs, NJ.

Mohammad, L.N., Titi, H.H. and Herath, A. (2002). *Effect of Moisture Content and Dry Unit Weight on the Resilient Modulus of Subgrade Soils Predicted by Cone Penetration Test*, Report No. FHWA/LA.00/355, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.

NAPA (1994). "Guidelines for Use of HMA Overlays to Rehabilitate PCC Pavements," *Publication IS 117*, National Asphalt Paving Association, Lanham, MD.

NCHRP 1-37A (2004). *Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*, Final Report, NCHRP Project 1-37A, Transportation Research Board, National Research Council, Washington, D.C.

Peck, R.B., Hansen, W.E. and Thornburn, T.H. (1974). *Foundation Engineering*, Wiley.

Saeed, A., Hall, J.W. and Barker, W. (2001). *Performance-Related Tests of Aggregates for Use in Unbound Pavement Layers*, NCHRP Report 453, Transportation Research Board, National Research Council, Washington, D.C.

Seed, H.B., Chan, C.K. and Lee, C.E. (1963). "Resilience Characteristics of Subgrade Soils and Their Relation to Fatigue Failures in Asphalt Pavements," *1st International Conference on the Structural Design of Asphalt Pavements*, Ann Arbor, MI.

Seed, H.B., Chan, C.K. and Monismith, C.L. (1955). "Effects of Repeated Loading on the Strength and Deformation of Compacted Clay," *HRB Proceedings*, No. 34, pp. 541-558.

Seed, H.B., and McNeill, R.L. (1956). "Soils Deformation in Normal Compression and Repeated Loading Tests," *HRB Bulletin 141*.

Sowers, G.F. (1979). *Introductory Soil Mechanics and Foundations*, Macmillan, New York, NY.

Sundberg, J. (1988) *Thermal Properties of Soils and Rocks*, Report No. 35, Swedish Geotechnical Institute, Linkoping, Sweden.

Tseng, K. and Lytton, R. (1989). "Prediction of Permanent Deformation in Flexible Pavement Material," *Implication of Aggregates in the Design, Construction, and Performance of Flexible Pavements*, ASTM STP 1016, ASTM, pp. 154-172.

U.S. Army Corps of Engineers (1953). "The Unified Soil Classification System," *Technical Memorandum 3-357*, Waterways Experiment Station, Vicksburg, MS.

Van Til, C.J., McCullough, B.F., Vallerga, B.A. and Hicks, R.G. (1972). *Evaluation of AASHO Interim Guides for Design of Pavement Structures*, NCHRP 128, Highway Research Board, Washington, D.C.

Von Quintus, H.L. and Killingsworth, B. (1997a). *Design Pamphlet for the Determination of Design Subgrade in Support of the 1993 AASHTO Guide for the Design of Pavement Structures*, Report No. FHWA-RD-97-083, FHWA, Washington, D.C.

Von Quintus, H.L. and Killingsworth, B. (1997b). *Design Pamphlet for the Determination of Layered Elastic Moduli for Flexible Pavement Design in Support of the 1993 AASHTO Guide for the Design of Pavement Structures*, Report No. FHWA-RD-97-077, Federal Highway Administration, Department of Transportation, Washington, DC.

Von Quintus, H.L. and Killingsworth, B. (1998). *Analysis Relating to Pavement Material Characterizations and Their Effects of Pavement Performance*, Report No. FHWA-RD-97-085, Federal Highway Administration, Department of Transportation, Washington, DC.

Witczak, M.W. (2004). *Harmonized Test Methods for Laboratory Determination of Resilient Modulus for Flexible Pavement Design*, NCHRP Project 1-28A Final Report, Transportation Research Board, National Research Council, Washington, D.C.

Yoder, E.J., and Witczak, M.W. (1975). *Principles of Pavement Design*, John Wiley and Sons, New York, NY.

# CHAPTER 6.0
# PAVEMENT STRUCTURAL DESIGN AND PERFORMANCE

## 6.1 INTRODUCTION

Previous chapters have described in qualitative terms the many geotechnical factors influencing pavement design and performance, the wide range of geotechnical properties required as input to the design procedures, and the various methods for determining the values of these geotechnical inputs. Now it is time to evaluate quantitatively the importance of these factors and properties. These are the primary objectives of the present chapter:

1. To illustrate via examples how the geotechnical properties described in Chapter 5 are incorporated in the pavement design calculations; and
2. To highlight the effects of the geotechnical factors and inputs on pavement design and performance.

These objectives will be met through a series of design scenarios. First, a set of reference or baseline flexible and rigid pavement designs are developed for a hypothetical and simple project scenario. Then, the effects of various deviations from the baseline conditions will be investigated and quantified. These include

- Soft/weak subgrade conditions
- Subgrade stabilization
- Low quality base/subbase material
- Drainage and water conditions
- Shallow bedrock conditions

The design scenarios are intentionally highly idealized and simplified. Their point is to emphasize in quantitative terms how changes in geotechnical inputs affect the overall pavement design and performance. In a sense, these design scenarios are examples of the types of sensitivity studies one should perform during design to evaluate the importance of the various design inputs, especially with reference to the quality of the information used in their estimation.

All of the design scenarios described in this chapter are for new construction (or reconstruction). This is not to minimize the importance of rehabilitation design; as described in Chapter 1, most pavement design today is in fact for rehabilitation and not new construction. However, most structural rehabilitation designs focus on restoration of the surface layer, either through asphalt concrete overlays, concrete pavement restoration, or a combination of the two. For these types of scenarios, the geotechnical inputs are essentially the same as for new construction design – *e.g.,* a subgrade is a subgrade whether it is beneath a new or an existing pavement. The principal things that change are the methods by which the geotechnical inputs are determined – *e.g.,* $M_R$ backcalculated from FWD tests instead of measured in the laboratory. The ways that these geotechnical inputs are used in the design calculations and the effects that they have on the design pavement structure are similar for new construction, rehabilitation, and reconstruction designs.

Both the current 1993 AASHTO Design Guide and the forthcoming new design guide from NCHRP Project 1-37A are applied to the scenarios in this chapter. Summaries of each of these design procedures are provided in Appendices C and D, respectively. Calculations for the 1993 AASHTO Guide designs are based on simple spreadsheet evaluation of the flexible and rigid pavement design equations. The calculations for the mechanistic-empirical designs were performed using Final Report Release version 0.700 (4/7/2004) of the NCHRP 1-37A software. This is the final version of the software as submitted to NCHRP at the conclusion of Project 1-37A.

It is important to keep in mind the significant differences between the two design procedures. The 1993 AASHTO Design Guide is an empirical methodology in which typically the design traffic, environmental conditions, and maximum serviceability (performance) loss are specified, and the corresponding required pavement structure—typically described just in terms of layer thicknesses—is determined. The NCHRP 1-37A procedure is a mechanistic-empirical methodology in which the design traffic, environmental conditions, and pavement structure are specified, and the corresponding pavement performance vs. time is predicted. In the NCHRP 1-37A procedure, several trial designs generally need to be evaluated in an iterative fashion in order to find the one (or more than one) that meets the design performance requirements.

Typically, there are multiple pavement designs that can provide the required performance for any scenario. This is true for both the 1993 AASHTO and NCHRP 1-37A design methodologies. The final selection of the "best" design should be based upon life-cycle costs, constructability, and other issues. Crude economic evaluations can be made in terms of initial construction costs, although even this is difficult because of large region-to-region variations in unit costs.

One final note regarding units of measure in this chapter: FHWA policy is to report values in SI units in all reports, with the corresponding U.S. Customary equivalent in parentheses. This is not done here. All values for the design examples in this chapter are reported in U.S. Customary units. There are two important practical reasons that dictate this choice. First, the structural layer coefficients, empirical correlations, and other data in the 1993 AASHTO Design Guide were developed and are presented in U.S. Customary units only. Although many of these could be converted to SI units, the consequences would be confusing. For example, expressing flexible pavement layer thicknesses in millimeters would require changing the values of the structural layer coefficients to quantities that would be unfamiliar to pavement engineers (*e.g.,* the $a_1$ value for asphalt concrete would change from 0.44 to 0.017 if asphalt thickness were expressed in millimeters). Second, although an SI version of the NCHRP 1-37A software is planned, at the time of this writing, only the U.S. Customary version is available. All of the outputs from the software are expressed in U.S. Customary units, and, consequently, the inputs described in this report are left in U.S. Customary units as well for consistency. The general SI-U.S. Customary conversion table included at the beginning of this reference manual can be used, if necessary, for converting units in this chapter.

## 6.2 BASELINE DESIGNS

The baseline designs for flexible and rigid pavements are intended to provide very simple and ordinary reference cases that can be used as the basis for subsequent exploration of the effects of various geotechnical inputs. The design scenario is based on the following assumptions:

- New construction
- Simple pavement structure
    - Hot mix asphalt concrete (HMA) over crushed stone graded aggregate base (GAB) over subgrade (SG) for flexible pavements
    - Jointed plain concrete pavement (JPCP) over graded aggregate base (GAB) over subgrade (SG) for rigid pavements
- Excellent, non-erodable base material (AASHTO A-1-a crushed stone)
- Nonexpansive subgrade
- Benign environmental conditions – *e.g.,* no frost heave/thaw or expansive soils
- Good drainage
- Simple traffic conditions – *e.g.,* no traffic growth over design period

Reference pavement designs consistent with these assumptions have been developed for a hypothetical new arterial highway outside of College Park, MD. The roadway is assumed to have two lanes in each direction and significant truck traffic consistenting primarily of Class 9 5-axle tractor-trailer units. The subgrade conditions are a non-expansive silty clay subgrade (AASHTO A-7-5/USCS MH material), a deep groundwater table, and no shallow bedrock. Environmental conditions in the Mid-Atlantic region are mild, so frost heave/thaw is not a design issue, and seasonal variations of the unbound material properties are expected to be minor.

Table 6-1 provides some typical in-place initial construction unit costs for paving materials in Maryland. These costs will be used for rough economic evaluations of the designs developed in this chapter.

**Table 6-1. Typical in-place unit material costs for use in example design problems (MDSHA, 2002).**

| Material | Reasonable Range | Typical Unit Price | Typical Unit Price |
|---|---|---|---|
| Hot mix asphalt concrete (12.5 mm PG 64-22) | \$30-\$50/ton | \$36/ton | \$14,250/lane-mi-in |
| Portland cement concrete (PCC) without steel | \$110-\$180/cy | \$144/cy | \$28,200/lane-mi-in |
| Graded aggregate base | \$24-\$60/cy | \$42/cy | \$8,200/lane-mi-in |

### 6.2.1 1993 AASHTO Design

***Flexible Pavement***

The baseline flexible pavement design is a three-layer system consisting of an asphalt concrete (AC) surface layer over a nonstabilized graded aggregate base (GAB) layer over subgrade (SG). The input parameters for the baseline design using the 1993 AASHTO flexible procedure for new pavements are summarized in Table 6-2. Refer to Chapter 5 for detailed explanations of all input parameters and the methods available for their determination.

The methodology by which the input parameters in Table 6-2 are used to determine the final structural design in the 1993 AASHTO Guide is described in Appendix C. The calculations are sufficiently straightforward that they can be easily performed using a spreadsheet. The key output from the 1993 AASHTO design methodology is the required pavement structure, which is determined as follows:

- Required overall structural number $SN = 4.61$
- Required structural number for asphalt concrete surface layer $SN_1 = 2.35$

- Required minimum thickness of asphalt $D_1 = \frac{SN_1}{a_1} = 5.3$ inches[1]
- Remaining structural number required for granular base layer $SN_2 = SN - D_1 a_1 = 2.28$
- Required thickness of granular base $D_2 = \frac{SN_2}{m_2 a_2} = 12.7$ inches[1]

Since the ratio of the layer coefficients $\left(a_1 / a_2 = 0.44 / 0.18 = 2.44\right)$ is greater than the ratio of the associated in-place unit costs per lane-mile-inch of thickness in Table 6-1 $\left(\$14,250 / \$8,200 = 1.74\right)$, there is an economic benefit from substituting granular base thickness with additional asphalt in this design – *i.e.,* replacing 2.4 inches of granular base with an additional 1.0 inch of asphalt concrete is both structurally feasible (at least in terms of the 1993 AASHTO Guide) and economically beneficial (at least in terms of initial construction costs) since it would result in a savings of about $5400 per lane mile at the same *SN* value. However, in order to avoid complicating comparisons between the various design scenarios later in this chapter, the baseline flexible pavement structure will be kept at <u>5.3 inches of AC over 12.7 inches of GAB</u> (or 5.5 inches of AC over 13 inches of GAB after rounding).

Although the 15-year initial service life specified for this scenario is typical for flexible pavements and equal to the values used in the design examples in the 1993 AASHTO Guide, current trends are toward longer life or "perpetual" pavement designs. The required pavement section for a 30-year initial service life based on the 1993 AASHTO Guide is 6.0 inches of AC over 13.7 inches of GAB. The "premium" for an additional 15 years of pavement life is thus only about three-quarters of an inch of asphalt and one inch of crushed stone base.

[1]The 1993 AASHTO Design Guide recommends rounding the asphalt layer thickness to the nearest half inch and unbound layer thicknesses to the nearest inch. However, all layer thicknesses are rounded to the nearest 0.1 inch in this chapter to make the comparisons between the various scenarios more meaningful.

**Table 6-2. Input parameters for 1993 AASHTO flexible pavement baseline design.**

| Input Parameter | Design Value | Notes |
|---|---|---|
| Initial service life | 15 years | 1 |
| Traffic ($W_{18}$) | $6.1 \times 10^6$ ESALs | 2 |
| Reliability | 90% | 3 |
| Reliability factor ($Z_R$) | -1.282 | |
| Overall standard error ($S_o$) | 0.45 | 1 |
| Allowable serviceability deterioration ($\Delta PSI$) | 1.7 | 4 |
| Subgrade resilient modulus ($M_R$) | 7,500 psi | 5 |
| Granular base type | AASHTO A-1-a | 1 |
| Granular base layer coefficient ($a_2$) | 0.18 | 6 |
| Granular base drainage coefficient ($m_2$) | 1.0 | 7 |
| Asphalt concrete layer coefficient ($a_1$) | 0.44 | 1 |

Notes:

1. Typical value for flexible pavement design.
2. Consistent with more detailed traffic input in the NCHRP 1-37A design (Section 6.2.2).
3. Typical value for a principal arterial (AASHTO, 1993).
4. Typical value for flexible pavements. No serviceability reduction for swelling or frost heave.
5. Consistent with the Level 3 default input for this soil class in the NCHRP 1-37A design after adjustment for seasonal effects (Section 6.2.2).
6. Corresponds to an $M_R$ value of 40,000 psi, which is consistent with the Level 3 default input for this soil class in the NCHRP 1-37A design (Section 6.2.2).
7. Representative of good drainage and moderate (5-25%) saturation conditions; matches value typically used by the Maryland State Highway Administration for design.

***Rigid Pavement***

The baseline rigid pavement design is a three layer JPCP system consisting of a Portland cement concrete (PCC) slab over a nonstabilized graded aggregate base (GAB) layer over subgrade (SG). The input parameters for the baseline design using the 1993 AASHTO rigid procedure for new pavements are summarized in Table 6-3. Refer to Chapter 5 for detailed explanations of all input parameters and the methods available for their determination. The rigid pavement design inputs are consistent with those used for the baseline flexible pavement design.

The methodology by which the input parameters in Table 6-3 are used to determine the final structural design in the 1993 AASHTO Guide is described in Appendix C. The calculations are sufficiently straightforward that they can be easily performed using a spreadsheet. The key output from the 1993 AASHTO design methodology is the required pavement structure.

This is determined from the input parameters in Table 6-3 and the following additional intermediate steps:

- Assume a 6-inch design thickness for the granular subbase. This value is typical for rigid pavements on reasonably competent subgrades.
- Determine the composite modulus of subgrade reaction $k_\infty$ representing the combined stiffness of the subgrade and the subgrade layer. For a 6-inch subbase thickness and unbound moduli as given in Table 6-3, $k_\infty = 423$ pci.
- Correct $k_\infty$ for loss of support to determine the design modulus of subgrade reaction $k_{eff}$. (NOTE: No shallow bedrock correction is required for the assumed subgrade conditions.) For $k_\infty = 423$ pci and LS = 2, $k_{eff} = 38$ pci.
- Determine the required slab thickness $D = 10.4$ inches from the rigid pavement design equation.[2]

The final design selection based upon the design inputs in Table 6-3 is therefore a <u>10.4 inch PCC slab over 6 inches of GAB</u> (or 10.5 inches of PCC over 6 inches of GAB after rounding).

[2] The 1993 AASHTO Design Guide recommends rounding the slab thickness to the nearest inch (nearest half inch if controlled grade slip form pavers are used). However, all slab thicknesses are rounded to the nearest 0.1 inch in this chapter to make the comparisons between the various scenarios more meaningful.

**Table 6-3. Input parameters for 1993 AASHTO rigid pavement baseline design.**

| Input Parameter | Design Value | Notes |
|---|---|---|
| Initial service life | 25 years | 1 |
| Traffic ($W_{18}$) | 16.4x10$^6$ ESALs | 2 |
| Reliability | 90% | 3 |
| Reliability factor ($Z_R$) | -1.282 | |
| Overall standard error ($S_o$) | 0.35 | 1 |
| Allowable serviceability deterioration ($\Delta PSI$) | 1.9 | 4 |
| Terminal serviceability level ($p_t$) | 2.5 | 1 |
| Subgrade resilient modulus ($M_R$) | 7,500 psi | 5 |
| Granular subbase type | AASHTO A-1-a | 1 |
| Granular subbase resilient modulus ($E_{SB}$) | 40,000 psi | 6 |
| Drainage coefficient ($C_d$) | 1.0 | 7 |
| Loss of Support ($LS$) | 2.0 | 8 |
| PCC modulus of rupture ($S_c'$) | 690 psi | 1 |
| PCC modulus of elasticity ($E_c$) | 4.4x10$^6$ psi | 1 |
| Joint load transfer coefficient ($J$) | 2.8 | 9 |

Notes:

1. Typical value for rigid pavement design.
2. Consistent with more detailed traffic input in the NCHRP 1-37A design (Section 6.2.2).
3. Typical value for a principal arterial (AASHTO, 1993).
4. Typical value for rigid pavements. No serviceability reduction for swelling or frost heave.
5. Consistent with the Level 3 default input for this soil class in the NCHRP 1-37A design after adjustment for seasonal effects (Section 6.2.2).
6. Consistent with the Level 3 default input for this soil class in the NCHRP 1-37A design (Section 6.2.2).
7. Representative of good drainage and moderate (5-25%) saturation conditions.
8. Within AASHTO-recommended range for unbound granular materials.
9. Typical value for JPCP with tied PCC shoulders and dowelled joints.

### 6.2.2 NCHRP 1-37A Design

***Flexible Pavement***

Consistent with the 1993 AASHTO design, the baseline flexible pavement structure for the NCHRP 1-37A design methodology is a three-layer new construction consisting of an asphalt concrete (AC) surface layer over a nonstabilized graded aggregate base (GAB) layer over subgrade (SG). However, the input parameters required for the NCHRP 1-37A methodology are considerably more extensive than those for the 1993 AASHTO Design Guide. The NCHRP 1-37A design inputs are summarized in Table 6-4. Refer to Chapter 5 for detailed explanations of all input parameters and to Appendix D for a summary of the NCHRP 1-37A design procedure. All of the inputs correspond to Level 3 quality, and the default values provided within the NCHRP 1-37A software are used wherever appropriate.

The NCHRP 1-37A procedure requires the evaluation of several trial pavement sections in order to find the design that best meets the performance requirements. The baseline pavement structure from the 1993 AASHTO design procedure can be conveniently taken as the initial trial section. The predicted rutting performance for a trial section corresponding to the 1993 AASHTO design of 5.3 inches of AC over 12.7 inches of GAB is shown in Figure 6-1; total rutting (after adjustment for reliability) at the end of the 15-year initial service life is 0.646 inches. Fatigue and thermal cracking are negligible for this design scenario, and rutting is the controlling distress type. The design limit for predicted total rutting is an explicit input in the NCHRP 1-37A procedure that would, in general, be set by individual agency policy. For the examples in this chapter, however, the design limit for total rutting is taken as the predicted rutting for the 1993 AASHTO design section in order to make the 1993 AASHTO and NCHRP 1-37A designs equivalent for the baseline conditions. The design limit for total rutting (after adjustment for reliability and rounding) is thus 0.65 inches, which is slightly less than the 0.75 inch default value in the NCHRP 1-37A software.

**Table 6-4. Input parameters for NCHRP 1-37A flexible pavement baseline design.**

| Input Parameter | Design Value | Notes |
|---|---|---|
| *General Information* | | |
| Design life | 15 years | 1 |
| Base/subbase construction month | September | 2 |
| Pavement construction month | September | 2 |
| Traffic open month | October | 2 |
| *Site/Project Identification* | | |
| Functional class | Principal Arterials - Others | |
| *Analysis Parameters* | | |
| Initial IRI | 63 in./mi | 2 |
| Terminal IRI | 172 in./mi | 2 |
| Alligator cracking limit | 25% | 2 |
| Total rutting limit | 0.65 in | See text |
| Reliability | 90% | |
| *Traffic* | | |
| Initial two-way AADTT | 2000 | |
| Number of lanes in design direction | 2 | |
| Percent of trucks in design direction | 50% | 2 |
| Percent of trucks in design lane | 95% | 2 |
| Operational speed | 55 mph | 2 |
| Monthly adjustment | 1.0 throughout | 2 |
| Vehicle class distribution | Level 3 defaults | Table 6-5 |
| Hourly distribution | Level 3 default | Table 6-6 |
| Traffic growth factor | 0% | |
| Axle load distribution factors | Level 3 defaults | Table 6-7 |
| Mean wheel location from edge | 18 in | 2 |
| Traffic wander standard deviation | 10 in | 2 |
| Design lane width | 12 ft | 2 |
| Number of axles per truck | Level 3 defaults | Table 6-8 |
| Average axle outside width | 8.5 ft | 2 |
| Dual tire spacing | 12 in | 2 |
| Tire pressure | 120 psi | 2 |
| Tandem axle spacing | 51.6 in | 2 |
| Tridem axle spacing | 49.2 in | 2 |
| Quad axle spacing | 49.2 | 2 |
| *Climate* | | |
| Latitude | 38.98$^{o}$ | |
| Longitude | -76.94$^{o}$ | |
| Elevation | 48 ft | |
| Depth of water table | 20 ft | |

| Input Parameter | Design Value | Notes |
|---|---|---|
| College Park, MD climate data | Generated | 3 |
| *Thermal Cracking* | | |
| Average AC tensile strength at 14$^o$F | 366.5 psi | 4 |
| Creep test duration | 100 sec | 4 |
| Creep compliance | Level 3 defaults | 4 |
| Mixture VMA | 14.1% | 5 |
| Aggregate coefficient of thermal contraction | 5x10$^{-6}$/$^o$F | 4 |
| *Drainage and Surface Properties* | | |
| Surface shortwave absorptivity | 0.85 | 2 |
| Infiltration | n/a | 2 |
| Drainage path length | n/a | 2 |
| Pavement cross slope | n/a | 2 |
| *AC Surface Layer* | | |
| Cumulative % retained on 3/4 inch sieve | 4 | 5 |
| Cumulative % retained on 3/8 inch sieve | 39 | 5 |
| Cumulative % retained on #4 sieve | 59 | 5 |
| % passing #200 sieve | 3 | 5 |
| Asphalt binder grade | PG 64-22 | 5 |
| Reference temperature | 70$^o$F | 2 |
| Effective binder content | 10.1% | 5 |
| Air voids | 4.0% | 5 |
| Total unit weight | 151 pcf | 5 |
| Poisson's ratio | 0.35 | 2 |
| Thermal conductivity | 0.67 BTU/hr-ft-$^o$F | 2 |
| Heat capacity | 0.23 BUT/lb-$^o$F | 2 |
| *Granular Base Layer* | | |
| Unbound material type | AASHTO A-1-a | |
| Analysis type | ICM Inputs | |
| Poisson's ratio | 0.35 | 2 |
| Coefficient of lateral pressure $K_0$ | 0.5 | 2 |
| Modulus | 40,000 psi | 2,6 |
| Plasticity index | 1% | |
| % passing #200 sieve | 3 | |
| % passing #4 sieve | 20 | |
| $D_{60}$ | 8 mm | |
| Compaction state | Compacted | 2 |
| Maximum dry unit weight | 122.2 pcf | 2 |
| Specific gravity of solids | 2.66 | 2 |
| Saturated hydraulic conductivity | 263 ft/hr | 2 |
| Optimum gravimetric water content | 11.1% | 2 |
| Calculated degree of saturation | 82% | 2 |

| Input Parameter | Design Value | Notes |
|---|---|---|
| SWCC parameter $a_f$ | 11.1 psi | 2 |
| SWCC parameter $b_f$ | 1.83 | 2 |
| SWCC parameter $c_f$ | 0.51 | 2 |
| SWCC parameter $h_r$ | 361 psi | 2 |
| *Compacted Subgrade (top 6 inches)* | | |
| Unbound material type | AASHTO A-7-5 | |
| Analysis type | ICM Inputs | |
| Poisson's ratio | 0.35 | 2 |
| Coefficient of lateral pressure $K_0$ | 0.5 | 2 |
| Modulus | 12,000 psi | 2,6 |
| Plasticity index | 30% | 2 |
| % passing #200 sieve | 85 | 2 |
| % passing #4 sieve | 99 | 2 |
| $D_{60}$ | 0.01 mm | 2 |
| Compaction state | Compacted | |
| Maximum dry unit weight | 97.1 pcf | 2 |
| Specific gravity of solids | 2.75 | 2 |
| Saturated hydraulic conductivity | $3.25 \times 10^{-5}$ ft/hr | 2 |
| Optimum gravimetric water content | 24.8% | 2 |
| Calculated degree of saturation | 88.9% | 2 |
| SWCC parameter $a_f$ | 301 psi | 2 |
| SWCC parameter $b_f$ | 0.995 | 2 |
| SWCC parameter $c_f$ | 0.732 | 2 |
| SWCC parameter $h_r$ | $1.57 \times 10^{4}$ psi | 2 |
| *Natural Subgrade (beneath top 6 inches)* | | |
| Unbound material type | AASHTO A-7-5 | |
| Compaction state | Uncompacted | |
| Maximum dry unit weight | 87.4 pcf | 2 |
| (other properties same as for compacted subgrade) | | |
| *Distress Potential* | | |
| Block cracking | None | 2 |
| Sealed longitudinal cracks outside wheel path | None | 2 |

Notes:
1. Typical initial service life for flexible pavement design.
2. Level 3 default/calculated/derived value from NCHRP 1-37A software.
3. Based on interpolated climate histories at IAD, DCA, and BWI airports.
4. Level 3 default/calculated/derived values from NCHRP 1-37A software for baseline AC mixture properties. Thermal cracking is not expected for the baseline design. However, these values are included here because they will be used in subsequent design scenarios.
5. Based on a Maryland State Highway Administration 19.0mm Superpave mix design.
6. Default input value at optimum moisture and density conditions before adjustment for seasonal effects (adjustment performed internally within the NCHRP 1-37A software).

**Table 6-5. AADTT distribution by truck class**
**(Level 3 defaults for Principal Arterials – Others).**

| | |
|---|---|
| Class 4 | 1.3% |
| Class 5 | 8.5% |
| Class 6 | 2.8% |
| Class 7 | 0.3% |
| Class 8 | 7.6% |
| Class 9 | 74.0% |
| Class 10 | 1.2% |
| Class 11 | 3.4% |
| Class 12 | 0.6% |
| Class 13 | 0.3% |

**Table 6-6. Hourly truck traffic distribution**
**(Level 3 defaults for Principal Arterials – Others).**

By period beginning:

| | | | |
|---|---|---|---|
| Midnight | 2.3% | Noon | 5.9% |
| 1:00 am | 2.3% | 1:00 pm | 5.9% |
| 2:00 am | 2.3% | 2:00 pm | 5.9% |
| 3:00 am | 2.3% | 3:00 pm | 5.9% |
| 4:00 am | 2.3% | 4:00 pm | 4.6% |
| 5:00 am | 2.3% | 5:00 pm | 4.6% |
| 6:00 am | 5.0% | 6:00 pm | 4.6% |
| 7:00 am | 5.0% | 7:00 pm | 4.6% |
| 8:00 am | 5.0% | 8:00 pm | 3.1% |
| 9:00 am | 5.0% | 9:00 pm | 3.1% |
| 10:00 am | 5.9% | 10:00 pm | 3.1% |
| 11:00 am | 5.9% | 11:00 pm | 3.1% |

**Table 6-7. Truck axle load distributions: Percentage of axle loads by truck class for single axle configurations (Level 3 defaults).**

| | Truck Class | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Axle Weight (lbs) | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 |
| 3000 | 1.80 | 10.05 | 2.47 | 2.14 | 11.65 | 1.74 | 3.64 | 3.55 | 6.68 | 8.88 |
| 4000 | 0.96 | 13.21 | 1.78 | 0.55 | 5.37 | 1.37 | 1.24 | 2.91 | 2.29 | 2.67 |
| 5000 | 2.91 | 16.42 | 3.45 | 2.42 | 7.84 | 2.84 | 2.36 | 5.19 | 4.87 | 3.81 |
| 6000 | 3.99 | 10.61 | 3.95 | 2.70 | 6.99 | 3.53 | 3.38 | 5.27 | 5.86 | 5.23 |
| 7000 | 6.80 | 9.22 | 6.70 | 3.21 | 7.99 | 4.93 | 5.18 | 6.32 | 5.97 | 6.03 |
| 8000 | 11.47 | 8.27 | 8.45 | 5.81 | 9.63 | 8.43 | 8.35 | 6.98 | 8.86 | 8.10 |
| 9000 | 11.30 | 7.12 | 11.85 | 5.26 | 9.93 | 13.67 | 13.85 | 8.08 | 9.58 | 8.35 |
| 10000 | 10.97 | 5.85 | 13.57 | 7.39 | 8.51 | 17.68 | 17.35 | 9.68 | 9.94 | 10.69 |
| 11000 | 9.88 | 4.53 | 12.13 | 6.85 | 6.47 | 16.71 | 16.21 | 8.55 | 8.59 | 10.69 |
| 12000 | 8.54 | 3.46 | 9.48 | 7.42 | 5.19 | 11.57 | 10.27 | 7.29 | 7.11 | 11.11 |
| 13000 | 7.33 | 2.56 | 6.83 | 8.99 | 3.99 | 6.09 | 6.52 | 7.16 | 5.87 | 7.32 |
| 14000 | 5.55 | 1.92 | 5.05 | 8.15 | 3.38 | 3.52 | 3.94 | 5.65 | 6.61 | 3.78 |
| 15000 | 4.23 | 1.54 | 3.74 | 7.77 | 2.73 | 1.91 | 2.33 | 4.77 | 4.55 | 3.10 |
| 16000 | 3.11 | 1.19 | 2.66 | 6.84 | 2.19 | 1.55 | 1.57 | 4.35 | 3.63 | 2.58 |
| 17000 | 2.54 | 0.90 | 1.92 | 5.67 | 1.83 | 1.10 | 1.07 | 3.56 | 2.56 | 1.52 |
| 18000 | 1.98 | 0.68 | 1.43 | 4.63 | 1.53 | 0.88 | 0.71 | 3.02 | 2.00 | 1.32 |
| 19000 | 1.53 | 0.52 | 1.07 | 3.50 | 1.16 | 0.73 | 0.53 | 2.06 | 1.54 | 1.00 |
| 20000 | 1.19 | 0.40 | 0.82 | 2.64 | 0.97 | 0.53 | 0.32 | 1.63 | 0.98 | 0.83 |
| 21000 | 1.16 | 0.31 | 0.64 | 1.90 | 0.61 | 0.38 | 0.29 | 1.27 | 0.71 | 0.64 |
| 22000 | 0.66 | 0.31 | 0.49 | 1.31 | 0.55 | 0.25 | 0.19 | 0.76 | 0.51 | 0.38 |
| 23000 | 0.56 | 0.18 | 0.38 | 0.97 | 0.36 | 0.17 | 0.15 | 0.59 | 0.29 | 0.52 |
| 24000 | 0.37 | 0.14 | 0.26 | 0.67 | 0.26 | 0.13 | 0.17 | 0.41 | 0.27 | 0.22 |
| 25000 | 0.31 | 0.15 | 0.24 | 0.43 | 0.19 | 0.08 | 0.09 | 0.25 | 0.19 | 0.13 |
| 26000 | 0.18 | 0.12 | 0.13 | 1.18 | 0.16 | 0.06 | 0.05 | 0.14 | 0.15 | 0.26 |
| 27000 | 0.18 | 0.08 | 0.13 | 0.26 | 0.11 | 0.04 | 0.03 | 0.21 | 0.12 | 0.28 |
| 28000 | 0.14 | 0.05 | 0.08 | 0.17 | 0.08 | 0.03 | 0.02 | 0.07 | 0.08 | 0.12 |
| 29000 | 0.08 | 0.05 | 0.08 | 0.17 | 0.05 | 0.02 | 0.03 | 0.09 | 0.09 | 0.13 |
| 30000 | 0.05 | 0.02 | 0.05 | 0.08 | 0.04 | 0.01 | 0.02 | 0.06 | 0.02 | 0.05 |
| 31000 | 0.04 | 0.02 | 0.03 | 0.72 | 0.04 | 0.01 | 0.03 | 0.03 | 0.03 | 0.05 |
| 32000 | 0.04 | 0.02 | 0.03 | 0.06 | 0.12 | 0.01 | 0.01 | 0.04 | 0.01 | 0.08 |
| 33000 | 0.04 | 0.02 | 0.03 | 0.03 | 0.01 | 0.01 | 0.02 | 0.01 | 0.01 | 0.06 |
| 34000 | 0.03 | 0.02 | 0.02 | 0.03 | 0.02 | 0.01 | 0.01 | 0.00 | 0.01 | 0.02 |
| 35000 | 0.02 | 0.02 | 0.01 | 0.02 | 0.02 | 0.00 | 0.01 | 0.00 | 0.00 | 0.01 |
| 36000 | 0.02 | 0.02 | 0.01 | 0.02 | 0.01 | 0.01 | 0.00 | 0.00 | 0.00 | 0.01 |
| 37000 | 0.01 | 0.01 | 0.01 | 0.01 | 0.01 | 0.00 | 0.01 | 0.00 | 0.01 | 0.01 |
| 38000 | 0.01 | 0.01 | 0.01 | 0.01 | 0.00 | 0.00 | 0.00 | 0.02 | 0.01 | 0.01 |
| 39000 | 0.01 | 0.00 | 0.01 | 0.01 | 0.01 | 0.00 | 0.01 | 0.01 | 0.00 | 0.01 |
| 40000 | 0.01 | 0.00 | 0.01 | 0.01 | 0.00 | 0.00 | 0.04 | 0.02 | 0.00 | 0.00 |
| 41000 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Total % | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 |

**Table 6-8. Truck axle distribution (Level 3 defaults).**

| Vehicle Class | Single Axle | Tandem Axle | Tridem Axle | Quad Axle |
|---|---|---|---|---|
| Class 4 | 1.62 | 0.39 | 0.00 | 0.00 |
| Class 5 | 2.00 | 0.00 | 0.00 | 0.00 |
| Class 6 | 1.02 | 0.99 | 0.00 | 0.00 |
| Class 7 | 1.00 | 0.26 | 0.83 | 0.00 |
| Class 8 | 2.38 | 0.67 | 0.00 | 0.00 |
| Class 9 | 1.13 | 1.93 | 0.00 | 0.00 |
| Class 10 | 1.19 | 1.09 | 0.89 | 0.00 |
| Class 11 | 4.29 | 0.26 | 0.06 | 0.00 |
| Class 12 | 3.52 | 1.14 | 0.06 | 0.00 |
| Class 13 | 2.15 | 2.13 | 0.35 | 0.00 |

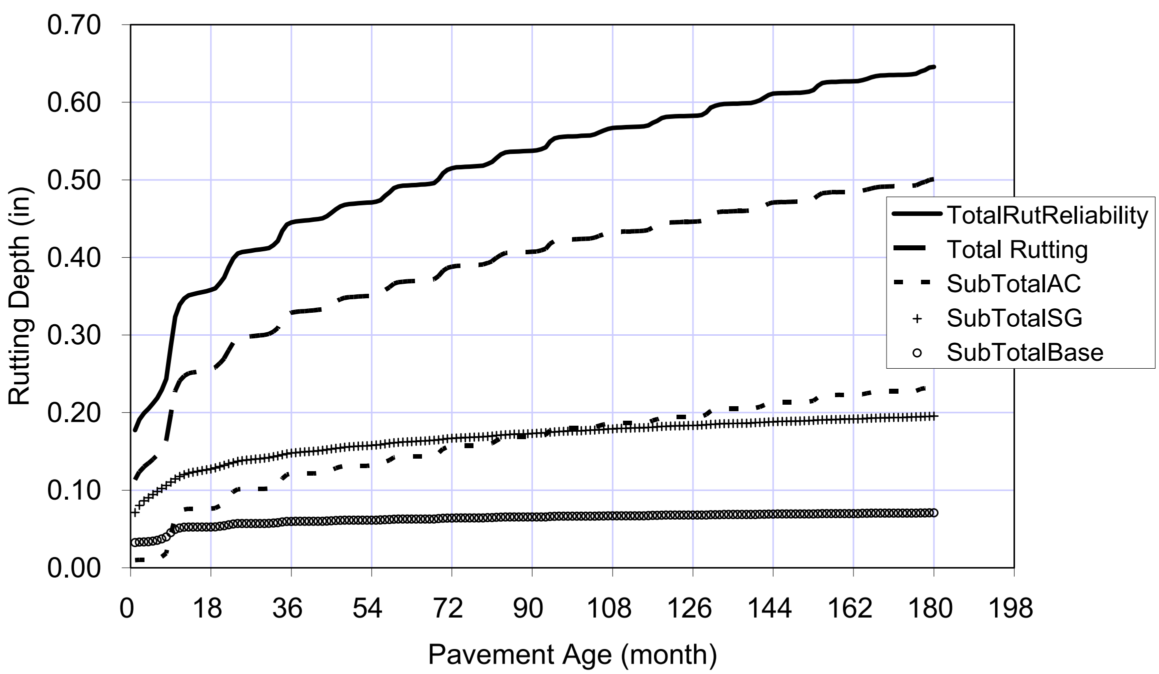



Figure 6-1. Predicted rutting performance for NCHRP 1-37A baseline flexible pavement design.

***Rigid Pavement***

Consistent with the 1993 AASHTO design, the baseline rigid pavement structure for the NCHRP 1-37A design methodology is a three-layer JPCP construction consisting of a Portland cement concrete (PCC) slab over a nonstabilized graded aggregate base (GAB) over subgrade (SG). However, the input parameters required for the NCHRP 1-37A methodology are considerably more extensive than for the 1993 AASHTO Design Guide. The NCHRP 1-37A design inputs are summarized in Table 6-9. Refer to Chapter 5 for detailed explanations of all input parameters and to Appendix D for a summary of the NCHRP 1-37A design methodology. All of the inputs correspond to Level 3 quality, and the default values provided within the NCHRP 1-37A software are used wherever appropriate.

The NCHRP 1-37A procedure requires the evaluation of several trial pavement sections in order to find the design that best meets the performance requirements. The baseline pavement structure from the 1993 AASHTO design procedure can be conveniently taken as the initial trial section. The predicted faulting performance for a trial section corresponding to the 1993 AASHTO Design section of 10.4 inches of PCC over 6.0 inches of GAB is shown in Figure 6-2; total faulting (after adjustment for reliability) at the end of the 25-year initial service life is 0.117 inches. Transverse fatigue cracking is negligible for this design scenario, and faulting is the controlling distress type. The design limit for predicted faulting, which is an explicit input in the NCHRP 1-37A procedure, would in general be set by individual agency policy. For the examples in this chapter, however, the design faulting limit is taken as the predicted faulting for the 1993 AASHTO design section. This is done in order to make the 1993 AASHTO and NCHRP 1-37A designs equivalent for the baseline conditions. The design limit for faulting (after adjustment for reliability and rounding) is thus 0.12 inches, which coincidentally equals the default value in the NCHRP 1-37A software. Note that initial service life for the baseline rigid pavement is 25.5 years after rounding of the faulting limit.

**Table 6-9. Input parameters for NCHRP 1-37A rigid pavement baseline design.**

| Input Parameter | Design Value | Notes |
|---|---|---|
| *General Information* | | |
| Initial service life | 25 years | 1 |
| Pavement construction month | September | 2 |
| Traffic open month | October | 2 |
| *Site/Project Identification* | | |
| Functional class | Principal Arterials - Others | |
| *Analysis Parameters* | | |
| Initial IRI | 63 in./mi | 2 |
| Terminal IRI | 172 in./mi | 2 |
| Transverse cracking (% slabs cracked) | 15% | 2 |
| Mean joint faulting | 0.12 in | See text |
| Reliability | 90% | |
| *Traffic* | | |
| Initial two-way AADTT | 2000 | |
| Number of lanes in design direction | 2 | |
| Percent of trucks in design direction | 50% | 2 |
| Percent of trucks in design lane | 95% | 2 |
| Operational speed | 55 mph | 2 |
| Monthly adjustment | 1.0 throughout | 2 |
| Vehicle class distribution | Level 3 defaults | Table 6-5 |
| Hourly distribution | Level 3 defaults | Table 6-6 |
| Traffic growth factor | 0% | |
| Axle load distribution factors | Level 3 defaults | Table 6-7 |
| Mean wheel location from edge | 18 in | 2 |
| Traffic wander standard deviation | 10 in | 2 |
| Design lane width | 12 ft | 2 |
| Number of axles per truck | Level 3 defaults | Table 6-8Table 6-8 |
| Average axle outside width | 8.5 ft | 2 |
| Dual tire spacing | 12 in | 2 |
| Tire pressure | 120 psi | 2 |
| Tandem axle spacing | 51.6 in | 2 |
| Tridem axle spacing | 49.2 in | 2 |
| Quad axle spacing | 49.2 | 2 |
| Wheelbase spacing | Level 3 defaults | Table 6-10 |
| *Climate* | | |
| Latitude | 38.98$^{o}$ | |
| Longitude | -76.94$^{o}$ | |
| Elevation | 48 ft | |
| Depth of water table | 20 ft | |
| College Park, MD climate data | Generated | 3 |
| *Design Features* | | |
| Permanent curl/warp effective | -10$^{o}$F | 2 |

| Input Parameter | Design Value | Notes |
|---|---|---|
| temperature difference | | |
| Joint spacing | 15 ft | 2 |
| Dowel bar diameter | 1 in | 2 |
| Dowel bar spacing | 12 in | 2 |
| Edge support | Widened slab | |
| Slab width | 14 ft | |
| Bond at PCC-base interface | Unbonded | |
| Base erodibility index | 4 (Fairly Erodable) | |
| *Drainage and Surface Properties* | | |
| Surface shortwave absorptivity | 0.85 | 2 |
| Infiltration | Minor (10%) | 2 |
| Drainage path length | 12 ft | 2 |
| Pavement cross slope | 2% | 2 |
| *PCC Surface Layer* | | |
| Unit weight | 150 pcf | 2 |
| Poisson's ratio | 0.2 | 2 |
| Coefficient of thermal expansion | $5.5 \times 10^{-6}/^{o}F$ | 2 |
| Thermal conductivity | 1.25 BTU/hr-ft-$^{o}$F | 2 |
| Heat capacity | 0.28 BTU/lb-$^{o}$F | 2 |
| Cement type | Type 1 | 2 |
| Cement content | 600 lb/yd$^3$ | 2 |
| Water/cement ratio | 0.42 | 2 |
| Aggregate type | Limestone | 2 |
| PCC zero-stress temperature | 120 $^{o}$F | 4 |
| Ultimate shrinkage at 40% relative humidity | 632 $\mu\varepsilon$ | 4 |
| Reversible shrinkage | 50% | 2 |
| Time to develop 50% of ultimate shrinkage | 35 days | 2 |
| 28-day PCC modulus of rupture | 690 psi | 2 |
| 28-day PCC elastic modulus | $4.4 \times 10^6$ psi | 4 |
| *Granular Base Layer* | | |
| Unbound material type | AASHTO A-1-a | |
| Analysis type | ICM Inputs | |
| Poisson's ratio | 0.35 | 2 |
| Coefficient of lateral pressure $K_0$ | 0.5 | 2 |
| Modulus | 40,000 psi | 2,5 |
| Plasticity index | 1% | |
| % passing #200 sieve | 3 | |
| % passing #4 sieve | 20 | |
| $D_{60}$ | 8 mm | |
| Compaction state | Compacted | 2 |
| Maximum dry unit weight | 122.2 pcf | 2 |
| Specific gravity of solids | 2.66 | 2 |
| Saturated hydraulic conductivity | 263 ft/hr | 2 |

| Input Parameter | Design Value | Notes |
|---|---|---|
| Optimum gravimetric water content | 11.1% | 2 |
| Calculated degree of saturation | 82% | 2 |
| SWCC parameter $a_f$ | 11.1 psi | 2 |
| SWCC parameter $b_f$ | 1.83 | 2 |
| SWCC parameter $c_f$ | 0.51 | 2 |
| SWCC parameter $h_r$ | 361 psi | 2 |
| *Compacted Subgrade (top 6 inches)* | | |
| Unbound material type | AASHTO A-7-5 | |
| Analysis type | ICM Inputs | |
| Poisson's ratio | 0.35 | 2 |
| Coefficient of lateral pressure $K_0$ | 0.5 | 2 |
| Modulus | 12,000 psi | 2,5 |
| Plasticity index | 30% | 2 |
| % passing #200 sieve | 85 | 2 |
| % passing #4 sieve | 99 | 2 |
| $D_{60}$ | 0.01 mm | 2 |
| Compaction state | Compacted | |
| Maximum dry unit weight | 97.1 pcf | 2 |
| Specific gravity of solids | 2.75 | 2 |
| Saturated hydraulic conductivity | $3.25 \times 10^{-5}$ ft/hr | 2 |
| Optimum gravimetric water content | 24.8% | 2 |
| Calculated degree of saturation | 88.9% | 2 |
| SWCC parameter $a_f$ | 301 psi | 2 |
| SWCC parameter $b_f$ | 0.995 | 2 |
| SWCC parameter $c_f$ | 0.732 | 2 |
| SWCC parameter $h_r$ | $1.57 \times 10^{4}$ psi | 2 |
| *Natural Subgrade (beneath top 6 inches)* | | |
| Unbound material type | AASHTO A-7-5 | |
| Compaction state | Uncompacted | |
| Maximum dry unit weight | 87.4 pcf | 2 |
| (Other properties same as for compacted subgrade) | | |

Notes:

1. Typical initial service life for rigid pavement design.
2. Level 3 default/calculated/derived value from NCHRP 1-37A software.
3. Based on interpolated climate histories at IAD, DCA, and BWI airports.
4. Level 3 default/calculated/derived values from NCHRP 1-37A software for baseline PCC mixture properties.
5. Default input value at optimum moisture and density conditions before adjustment for seasonal effects (adjustment performed internally within the NCHRP 1-37A software).

**Table 6-10. Wheelbase spacing distribution (Level 3 defaults).**

| | Short | Medium | Long |
|---|---|---|---|
| **Average Axle Spacing (ft)** | 12 | 15 | 18 |
| **Percent of trucks** | 33% | 33% | 34% |


Figure 6-2. Predicted faulting performance for NCHRP 1-37A baseline rigid pavement design.

### 6.2.3 Summary

Baseline flexible and rigid pavement designs were developed using the 1993 AASHTO Design Guide and the input parameters in Table 6-2 and Table 6-3. The final designs are 5.3 inches of AC over 12.7 inches of GAB (5.5”/13” after rounding) and 10.4 inches of PCC over 6 inches of GAB (10.5”/6” after rounding), respectively. Initial construction costs for these designs, based on the unit cost data in Table 6-1, are $180,000 and $342,000 per line-mile, respectively ($185,000 and $345,000 per lane-mile using rounded layer thicknesses).

These baseline designs were then analyzed using the NCHRP 1-37A procedures and the input parameters in Table 6-4 and Table 6-9 to determine the corresponding distress levels at the end of initial service life. Permanent deformations are the controlling distress type for the flexible pavement scenario; the predicted total rutting (after adjustment for reliability) for the baseline flexible pavement section is 0.65 inches, as compared to the 0.75-inch default value in the NCHRP 1-37A software. Joint faulting is the controlling distress type for the rigid pavement scenario; the predicted joint faulting (after adjustment for reliability) for the baseline rigid pavement design is 0.12 inches, identical to the default value in the NCHRP 1-37A software. Note that the baseline design scenarios are not greatly different from the pavement conditions at the AASHO Road Test (except perhaps for climate), and therefore the 1993 AASHTO designs should be in general agreement with those from the more sophisticated NCHRP 1-37A methodology. Discrepancies between the design procedures should become more pronounced as conditions increasingly deviate from the AASHO Road Test conditions.

## 6.3 SOFT SUBGRADE

The design scenario for this case is identical to the baseline conditions in Section 6.2, except for a much softer and weaker subgrade. The subgrade is now postulated to be a very soft high plasticity clay (AASHTO A-7-6, USCS CH) with $M_R$ = 6000 psi at optimum moisture and density before adjustment for seasonal effects. Note that this $M_R$ value is even lower than the NCHRP 1-37A default values for an A-7-6/CH material in order to accentuate the effects of low subgrade stiffness. The groundwater depth is left unchanged from the baseline scenario in order to focus on the subgrade stiffness effect. Intuitively, more substantial pavement sections are expected for this scenario as compared to the baseline conditions to achieve the same level of pavement performance. This can be achieved by increasing the thicknesses of the AC/PCC/granular base layers, increasing the quality of the AC/PCC/granular base materials, stabilizing the granular base layer, treating the soft subgrade soil, or some combination of these design modifications. In order to keep the comparisons among scenarios simple, only increases in AC or PCC thickness will be considered here.

### 6.3.1 1993 AASHTO Design

***Flexible Pavement***

The only modification to the 1993 AASHTO flexible pavement baseline inputs (Table 6-2) required to simulate the soft subgrade condition is a reduction of the seasonally-adjusted subgrade resilient modulus $M_R$ from 7,500 to 3,800 psi.[3] The $W_{18}$ traffic capacity for baseline flexible pavement section under the soft subgrade conditions is only $1.25 \times 10^6$ ESALs, corresponding to an 80% decrease in initial service life.

The required pavement structure for the soft subgrade condition is determined from the 1993 AASHTO design procedure, as follows (assuming changes only in the AC layer thickness):

- Required overall structural number $SN$ = 5.76 (compared to 4.61 for the baseline conditions)
- Structural number provided by granular base (same thickness $D_2$ as in baseline design) $SN_2 = m_2 a_2 D_2 = (1.0)(0.18)(12.7) = 2.28$
- Required asphalt structural number $SN_1 = SN - SN_2 = 5.76 - 2.28 = 3.48$
- Required asphalt layer thickness $D_1 = \dfrac{SN_1}{a_1} = 7.9$ inches

The design for the soft subgrade condition is thus <u>7.9 inches of AC over 12.7 inches of GAB</u> (before rounding). This represents a 50% increase in AC thickness as compared to the 5.3 inches in the baseline design, which, in turn, translates to a 20% initial construction cost increase of about $37K per lane-mile (using the typical unit cost data in Table 6-1).

***Rigid Pavement***

The only modification to the 1993 AASHTO rigid pavement baseline inputs (Table 6-3) required to simulate the soft subgrade condition is a reduction of the seasonally-adjusted subgrade resilient modulus $M_R$ from 7,500 to 3,800 psi. The $W_{18}$ traffic capacity for the baseline rigid pavement section under the soft subgrade conditions is reduced to $15.5 \times 10^6$ ESALs, corresponding to a 6% decrease in initial service life.

The reduction in foundation stiffness in this scenario has a direct effect on the design modulus of subgrade reaction $k_{eff}$, which decreases from its original value of 38 pci for baseline conditions to a value of 27 pci for the soft subgrade case. However, the required slab thickness is relatively insensitive to this reduction in foundation stiffness, increasing only 0.1 inches for a final design of <u>10.5 inches of PCC over 6 inches of GAB</u> for the soft subgrade

[3] Based on the results from the NCHRP 1-37A analyses for these conditions.

condition. Note that after rounding to the nearest half-inch, this design is identical to the rigid pavement design for the baseline conditions.

A common constructability concern under these soft *in-situ* soil conditions is the requirement for a stable working platform. The 6-inch granular subbase is unlikely to provide adequate stability. Consequently, a realistic final design would require either a thicker granular subbase, a separate granular working platform (not included in the structural design calculations), and/or subgrade improvement (see Chapter 7) for constructability.

### 6.3.2 NCHRP 1-37A Design

***Flexible Pavement***

Changing the subgrade soil type to A-7-6 changes many of the other Level 3 default inputs for the subgrade in the NCHRP 1-37A design methodology. The altered input parameters for the soft subgrade condition are summarized in Table 6-11. Figure 6-3 summarizes the predicted rutting vs. time for the baseline flexible pavement section (5.3″ AC over 12.7″ granular base); the time to the 0.65 inch total rutting design limit is only 93 months (7.75 years), corresponding to a 48% decrease in initial service life due to the soft subgrade conditions.

The trial designs (assuming only increases in AC thickness) and their corresponding predicted performance at end of the initial service life are listed in Table 6-12. Rutting is again the critical distress mode controlling the design in all cases; the design limit of 0.65 inches for total rutting is based on the performance of the baseline pavement section, as described previously in Section 6.2.2. Interpolating among the results in Table 6-12, the final flexible pavement design section for the soft subgrade conditions consists of <u>7.9 inches of AC over 12.7 inches of GAB</u>. This design section is identical to that obtained from the 1993 AASHTO Design Guide for this scenario.

**Table 6-11. Modified input parameters for NCHRP 1-37A flexible pavement design: soft subgrade scenario.**

| Input Parameter | Design Value | Notes |
|---|---|---|
| *Compacted Subgrade (top 6 inches)* | | |
| Unbound material type | AASHTO A-7-6 | |
| Analysis type | ICM Inputs | |
| Poisson's ratio | 0.35 | 1 |
| Coefficient of lateral pressure $K_0$ | 0.5 | 1 |
| Modulus | 6,000 psi | 2,3 |
| Plasticity index | 40% | 1 |
| % passing #200 sieve | 90 | 1 |
| % passing #4 sieve | 99 | 1 |
| $D_{60}$ | 0.01 mm | 1 |
| Compaction state | Compacted | |
| Maximum dry unit weight | 91.3 | 1 |
| Specific gravity of solids | 2.77 | 1 |
| Saturated hydraulic conductivity | $3.25 \times 10^{-5}$ ft/hr | 1 |
| Optimum gravimetric water content | 28.8% | 1 |
| Calculated degree of saturation | 89.4% | 1 |
| SWCC parameter $a_f$ | 750 psi | 1 |
| SWCC parameter $b_f$ | 0.911 | 1 |
| SWCC parameter $c_f$ | 0.772 | 1 |
| SWCC parameter $h_r$ | $4.75 \times 10^{4}$ psi | 1 |
| *Natural Subgrade (beneath top 6 inches)* | | |
| Unbound material type | AASHTO A-7-6 | |
| Compaction state | Uncompacted | |
| Maximum dry unit weight | 82.2 pcf | 2 |
| (Other properties same as for compacted subgrade) | | |

Notes:

1. Level 3 default/calculated/derived value from NCHRP 1-37A software.
2. Set artificially low to simulate a soft subgrade condition.
3. Input value at optimum moisture and density conditions before adjustment for seasonal effects (adjustment performed internally within the NCHRP 1-37A software).

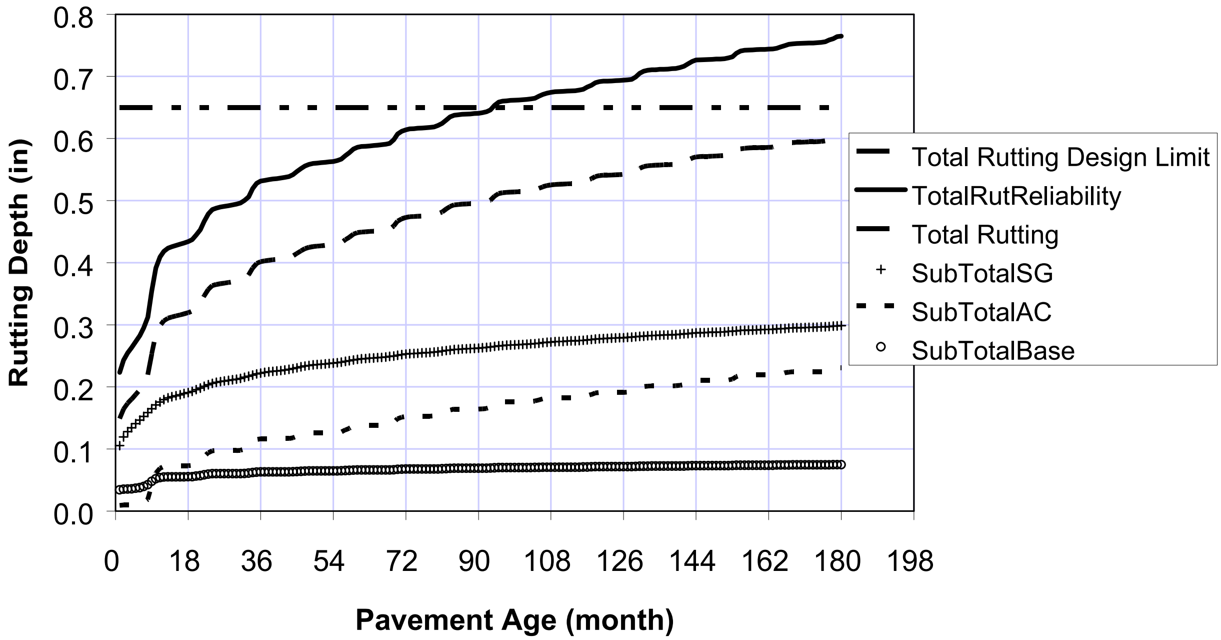



Figure 6-3. Predicted rutting performance for soft subgrade scenario.

**Table 6-12. Trial cross sections for NCHRP 1-37A flexible pavement design: soft subgrade scenario.**

| AC Thickness (in.) | Base Thickness (in.) | Total Rutting (in.) |
|---|---|---|
| 5.3 | 12.7 | 0.765 |
| 6.0 | 12.7 | 0.730 |
| 8.0 | 12.7 | 0.643 |
| 10.0 | 12.7 | 0.572 |
| | Design Limit: | 0.65 |

***Rigid Pavement***

Changing the subgrade soil type to A-7-6 changes many of the other Level 3 default inputs for the subgrade in the NCHRP 1-37A design methodology. The altered input parameters for the rigid pavement soft subgrade condition are the same as those summarized earlier in Table 6-11 for the corresponding flexible design condition. Figure 6-4 summarizes the predicted faulting vs. time for the baseline rigid pavement section (10.4″ PCC over 6.0″ granular base); the time to the 0.12 inch faulting design limit is only 22.2 years, corresponding to a 13% decrease in initial service life due to the soft subgrade conditions.

The trial designs (assuming only increases in PCC slab thickness) and their corresponding predicted performance at end of design life are listed in Table 6-13. Faulting is again the critical distress mode controlling the design in all cases; the design limit of 0.12 inches for faulting is based on the performance of the baseline pavement section, as described previously in Section 6.2.2. Interpolating among the results in Table 6-13, the final rigid pavement design section for the soft subgrade conditions consists of a 10.9 inch PCC slab over 6.0 inches of GAB. This slab thickness is 0.5 inch (5%) greater than that obtained from the 1993 AASHTO Design Guide for this scenario; this corresponds to an initial construction cost increase of $14K (8%) per lane-mile.

Again, the subgrade soil is so soft and weak in this scenario that some additional design features may be required to provide a stable working platform during construction. The 6 inch granular subbase is unlikely by itself to provide an adequate working platform.

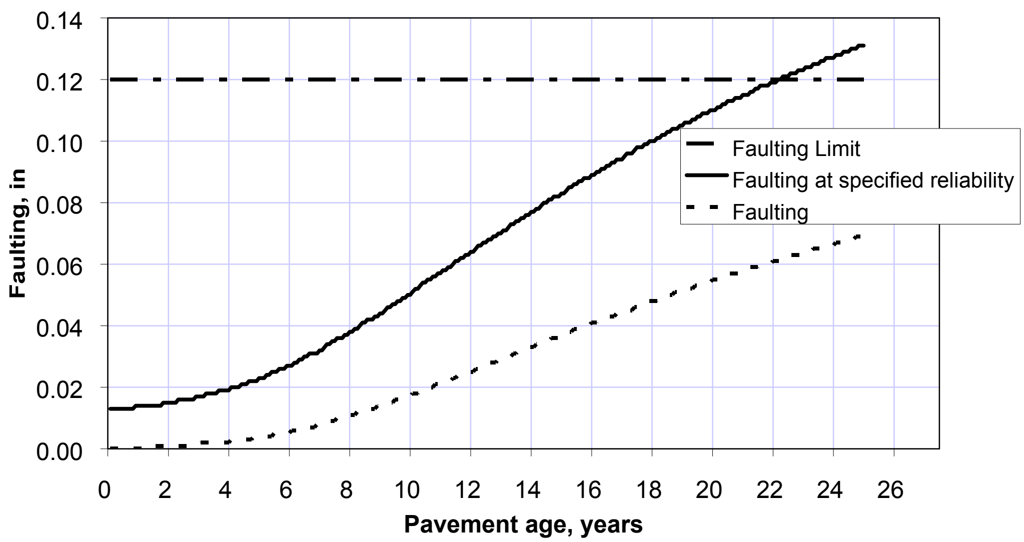



Figure 6-4. Predicted faulting performance for soft subgrade scenario.

**Table 6-13. Trial cross sections for NCHRP 1-37A rigid pavement design: soft subgrade scenario.**

| PCC Thickness (in.) | Base Thickness (in.) | Faulting (in.) |
|---|---|---|
| 10.4 | 6 | 0.131 |
| 10.7 | 6 | 0.125 |
| 11.0 | 6 | 0.118 |
| | Design Limit: | 0.12 |

### 6.3.3 Summary

The design flexible pavement sections for the baseline and soft subgrade scenarios are summarized in Figure 6-5. As expected, the soft subgrade condition mandates a thicker pavement cross section. For the both the 1993 AASHTO and NCHRP 1-37A designs, the required AC thickness increases from 5.3 – 7.9 inches (before rounding and based on the simplest assumption of constant GAB thickness). The GAB thickness remains a constant 12.7 inches for all designs, although as described previously, the granular base thickness would probably be increased for constructability purposes in order to provide a stable working platform over the soft subgrade.

As mentioned previously, increased asphalt thickness could economically be used with a thinner aggregate base layer for the same structural capacity in the 1993 AASHTO procedure, but this adjustment has not been made here. The NCHRP 1-37A methodology does not allocate the increased overall section thickness to the individual layers in the same way as the 1993 AASHTO procedure, and in practice one should examine multiple thickness combinations to find the minimum cost design that meets the specified performance limits.

Initial construction cost ranges (based on Table 6-1) for the flexible pavement designs are summarized in Figure 6-6. The average initial construction costs increase from about $180K to $217K (~20%) per lane-mile due to the soft subgrade for both the 1993 AASHTO and NCHRP 1-37A designs.

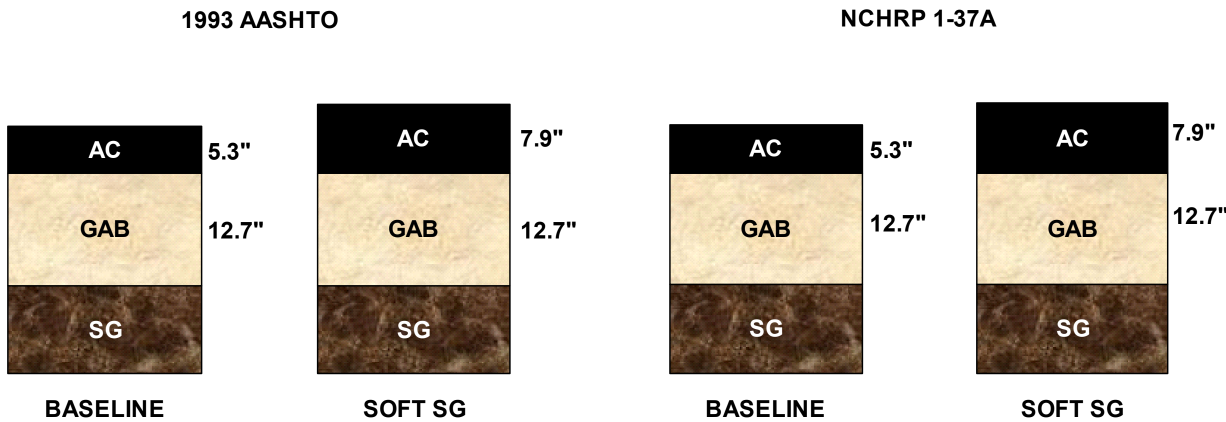



Figure 6-5. Summary of flexible pavement sections: soft subgrade scenario.

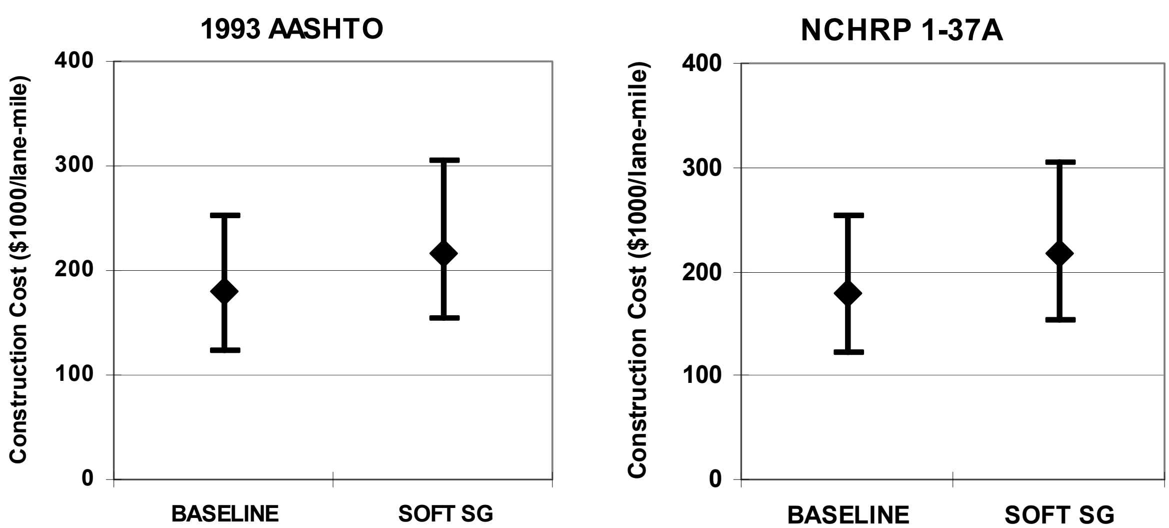



Figure 6-6. Example construction costs for flexible pavement sections: soft subgrade scenario.

The design rigid pavement sections for the baseline and soft subgrade scenarios are summarized in Figure 6-7. The increase in slab thickness required for the soft subgrade conditions is slight, from 10.4 to 10.5 inches for the 1993 AASHTO designs and from 10.4 to 10.9 inches for the NCHRP 1-37A sections (before rounding). The GAB thickness remains a constant 6 inches for all designs, although as described previously, this would probably be increased for constructability purposes.

Initial construction cost ranges (based on Table 6-1) for the rigid pavement designs are summarized in Figure 6-8. The average increase in initial construction cost due to the soft subgrade are quite small, ranging from about \$2K (~1%) for the 1993 AASHTO design to about \$14K (~8%) for the NCHRP 1-37A sections. Note that these initial construction costs for rigid pavements (Figure 6-8) cannot be fairly compared to the initial construction costs for flexible pavements (Figure 6-6) because of the different assumptions for the initial service life and the different maintenance and repair costs that will be required over their design lives. A fair comparison would require evaluation of life-cycle costs, including maintenance, repair, rehabilitation, and perhaps user costs, in addition to the initial construction expense.

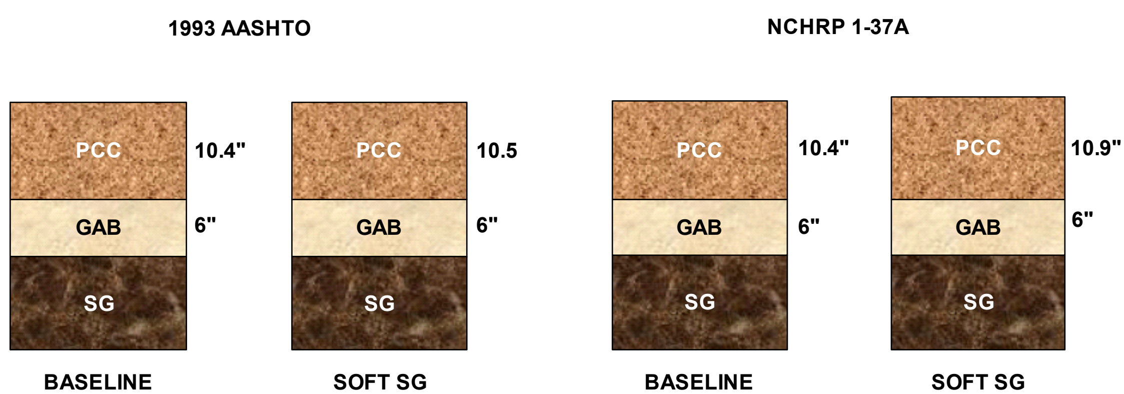



Figure 6-7. Summary of rigid pavement sections: soft subgrade scenario.

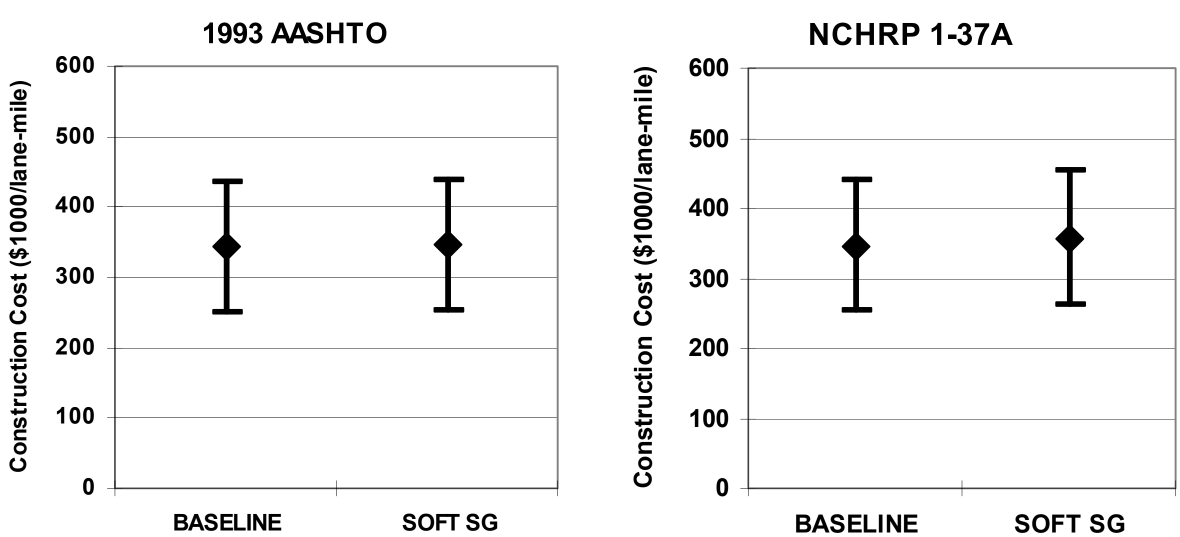



Figure 6-8. Example construction costs for rigid pavement sections: soft subgrade scenario.

Given the significant increases in flexible pavement section required to deal with the weak subgrade condition, other design approaches should be considered, such as subgrade stabilization, geosynthetic reinforcement of the base layer, or switching to a rigid pavement system that is more tolerant of poor foundation conditions. The benefits of subgrade stabilization for the flexible pavement designs are explored in the next section.

## 6.4 SUBGRADE STABILIZATION

As described in the preceding section, a very soft and weak subgrade requires a substantially thicker section for flexible pavements. The effect on required slab thickness for rigid pavements is slight; this confirms the conventional wisdom that rigid pavements are particularly advantageous for very poor subgrade support conditions.

Lime stabilization is a common technique for improving soft and weak subgrades beneath flexible pavements. A primary benefit of lime stabilization is a greatly increased stiffness within the stabilized zone as a function of the lime content. Consequently, the lime content—or more specifically, the effect of lime content on subgrade properties—and the thickness of the stabilized zone are the primary variables for the stabilization design.

The effect of lime content on the engineering properties of stabilized subgrades will depend greatly on the specific subgrade being stabilized. As a simple illustration of the benefits of lime stabilization, the sensitivity of predicted rutting to the thickness and resilient modulus of the stabilized zone can be examined using the NCHRP 1-37A methodology.[4] For the purpose of this illustration, all design inputs are kept the same as for the soft subgrade scenario (Table 6-4 and Table 6-11) except that the thickness and resilient modulus of the compacted upper layer of the subgrade are adjusted to simulate the lime stabilized zone.

Total rutting (after adjustment for reliability) at the end of the 15-year initial service life as predicted by the NCHRP 1-37A design methodology for various thicknesses and stiffnesses of the lime stabilized zone are summarized in Figure 6-9. Based on the data in Chapter 7, each 1% of lime in the stabilized zone corresponds very roughly to an increase in resilient modulus $M_R$ of about 10,000 psi. The data in Figure 6-9 suggest that a lime content corresponding to an $M_R$ value of 60,000 psi over a depth of 18 inches is one of several combinations that will meet the 0.65 inch design limit for total rutting. A pavement section consisting of 5.3 inches of AC over 12.7 inches of GAB over 18 inches of lime-stabilized subgrade ($M_R$=60,000 psi) should provide sufficient performance. This will also be the more

[4] Note: The NCHRP 1-37A design methodology has not been calibrated for lime stabilization because of an insufficient numbers of appropriate field sections in the LTPP database.

economically feasible design if the cost of the lime stabilization is less than the $37K/lane-mile cost of the 2.6 inches of asphalt concrete saved as a consequence of the subgrade improvement.

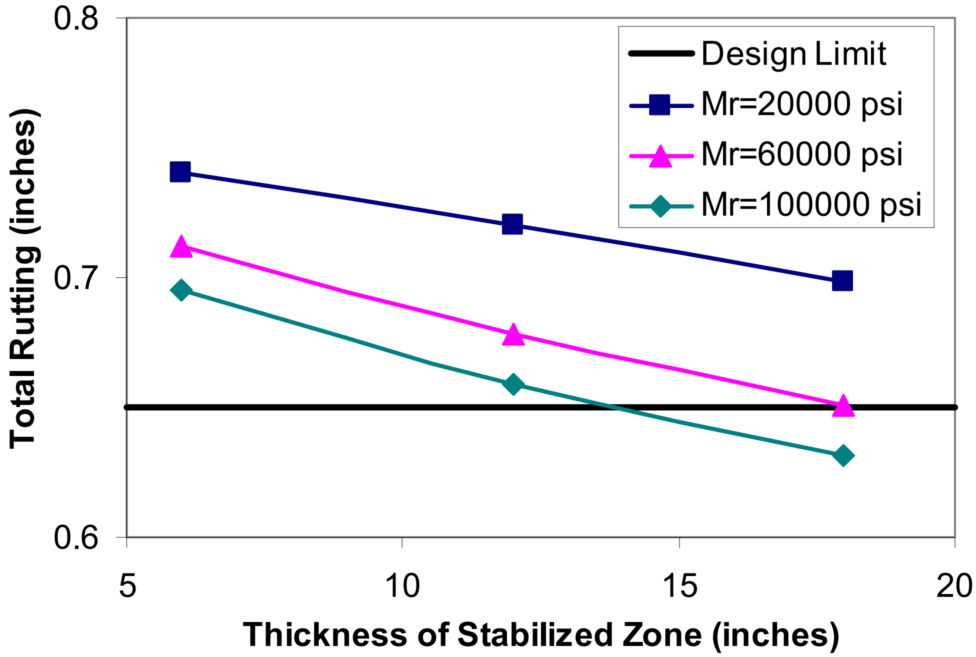



Figure 6-9. Effect of lime stabilization on predicted total rutting.

## 6.5 LOW QUALITY BASE/SUBBASE

The design scenario for this case is identical to the baseline conditions in Section 6.2, except that a lower quality granular base material is specified. The granular base is now postulated to be a clayey sand gravel (AASHTO A-2-6, USCS GC or SC) with $M_R$ = 26,000 psi at optimum moisture and density. Intuitively, significantly thicker design pavement sections are expected for this scenario, as compared to the baseline conditions to achieve the same level of pavement performance.

### 6.5.1 1993 AASHTO Design

***Flexible Pavement***

Three modifications to the 1993 AASHTO flexible pavement baseline inputs (Table 6-2) are made to simulate the low quality base condition:

1. The seasonally-adjusted resilient modulus $M_R$ for the granular base layer is reduced from 40,000 to 26,000 psi.

2. The structural layer coefficient $a_2$ is reduced from 0.18 to 0.12, consistent with the reduction in $M_R$ for the granular base (see correlations in Section 5.4.5).
3. The drainage coefficient $m_2$ is reduced from 1.0 to 0.70 to reflect a reduction in drainage quality from good to poor, due to the substantially increased fines content.
4. This yields an $SN$ value of only 3.4 for the baseline design layer thicknesses. The $W_{18}$ traffic capacity for this $SN$ value is only $0.88 \times 10^6$ ESALs, corresponding to an 86% decrease in initial service life.

The required structure is then determined from the design equations, as follows (again assuming changes only in the AC layer thickness):

- Required overall structural number $SN = 4.6$ (unchanged from baseline conditions)
- Structural number provided by granular base (same thickness $D_2$ as in baseline design) $SN_2 = m_2 a_2 D_2 = (0.7)(0.12)(12.7) = 1.07$
- Required asphalt structural number $SN_1 = SN - SN_2 = 4.61 - 1.07 = 3.54$
- Required asphalt layer thickness $D_1 = \dfrac{SN_1}{a_1} = 8.0$ inches

The design for the low quality base condition is thus <u>8.0 inches of AC over 12.7 inches of GAB</u> (before rounding). This represents a significant increase in AC thickness as compared to the baseline AC design thickness of 5.3 inches and is even slightly thicker than the 7.9 inches of AC required for the soft subgrade scenario (Section 6.3.1). This translates to a 21% initial construction cost increase of about $38K per lane-mile (using the typical unit cost data in Table 6-1 and assuming that the unit cost for the low quality base is the same as for high quality crushed stone).

***Rigid Pavement***

Three modifications to the 1993 AASHTO rigid pavement baseline inputs (Table 6-3) are made to simulate the low quality subbase condition:

1. The resilient modulus $M_R$ for the granular subbase is reduced from 40,000 to 26,000 psi.
2. The Loss of Support LS coefficient is increased from 2 to 3 to reflect the increased erosion potential of the fines in the granular subbase.
3. The drainage coefficient $C_d$ is reduced from 1.0 to 0.85 to reflect a reduction in drainage quality from good to poor, due to the substantially increased fines content.

The $W_{18}$ traffic capacity for the baseline rigid pavement section under the low quality subbase condition is reduced to $8.3 \times 10^6$ ESALs, corresponding to a 50% decrease in initial service life.

The first two changes listed above will have a direct effect on the design modulus of subgrade reaction $k_{eff}$, which reduces from its baseline value of 38 pci to a value of 17 pci for the low quality subbase condition. The 1993 AASHTO design corresponding to this $k_{eff}$ and $C_d$ of 0.85 consists of an <u>11.5 inch PCC slab over 6 inches of GAB</u>. Note that this design is substantially thicker than the 10.4" PCC over 6" subbase section for the 1993 AASHTO baseline scenario; this corresponds to an initial construction cost increase of $31K (9%) per lane-mile. The increase in slab thickness is due primarily to the reduction in the drainage coefficient $C_d$; only about 0.2 inches of the slab thickness increase is attributable to the increased erodibility of the subbase, as reflected in the larger *LS* value and the consequently reduced value for $k_{eff}$.

### 6.5.2 NCHRP 1-37A Design

***Flexible Pavement***

Changing the granular base soil type from A-1-a to A-2-6 changes many of the other Level 3 default inputs for this layer in the NCHRP 1-37A design methodology. The altered input parameters for the low quality base condition are summarized in Table 6-14. Figure 6-10 summarizes the predicted rutting vs. time for the baseline flexible pavement section (5.3″ AC over 12.7″ GAB); the time to the 0.65 inch total rutting design limit is only 153 months (12.75 years), corresponding to a 15% decrease in initial service life due to the low quality base. Although cracking is not the controlling distress for this scenario, reducing the quality of the base material does have a significant effect on cracking, more so than on rutting: predicted alligator cracking increases by a factor of 3 and longitudinal cracking by a factor of over 10 in the low quality base scenario, as compared to baseline conditions.

The trial designs and their corresponding predicted performance at end of the initial service life are listed in Table 6-15. Rutting is again the critical distress mode controlling the design in all cases; the design limit of 0.65 inches for total rutting is based on the performance of the baseline pavement section, as described previously in Section 6.2.2. Interpolating among the results in Table 6-15, the final flexible pavement design section for the low quality base condition consists of <u>5.8 inches of AC over 12.7 inches of GAB</u>. This is significantly thinner than the 8 inches of asphalt required by the 1993 AASHTO design for this scenario, but only 0.5 inches (9%) thicker than in the baseline design, corresponding to a $7.1K/lane-mile (4%) increase in initial construction costs.

**Table 6-14. Modified input parameters for NCHRP 1-37A flexible pavement design: low quality base scenario.**

| Input Parameter | Design Value | Notes |
|---|---|---|
| *Granular Base Layer* | | |
| Unbound material type | AASHTO A-2-6 | |
| Analysis type | ICM Inputs | |
| Poisson's ratio | 0.35 | 1 |
| Coefficient of lateral pressure $K_0$ | 0.5 | 1 |
| Modulus | 26,000 psi | 2 |
| Plasticity index | 15% | 1 |
| % passing #200 sieve | 20 | 1 |
| % passing #4 sieve | 95 | 1 |
| $D_{60}$ | 0.1 mm | 1 |
| Compaction state | Compacted | |
| Maximum dry unit weight | 117.5 pcf | 1 |
| Specific gravity of solids | 2.71 | 1 |
| Saturated hydraulic conductivity | $1.73 \times 10^{-5}$ ft/hr | 1 |
| Optimum gravimetric water content | 13.9% | 1 |
| Calculated degree of saturation | 85.9% | 1 |
| SWCC parameter $a_f$ | 23.1 psi | 1 |
| SWCC parameter $b_f$ | 1.35 | 1 |
| SWCC parameter $c_f$ | 0.586 | 1 |
| SWCC parameter $h_r$ | 794 psi | 1 |

Notes:

1. Level 3 default/calculated/derived value from NCHRP 1-37A software.
2. Level 3 default value for this soil class. Input value is at optimum moisture and density conditions before adjustment for seasonal effects (adjustment performed internally within the NCHRP 1-37A software).

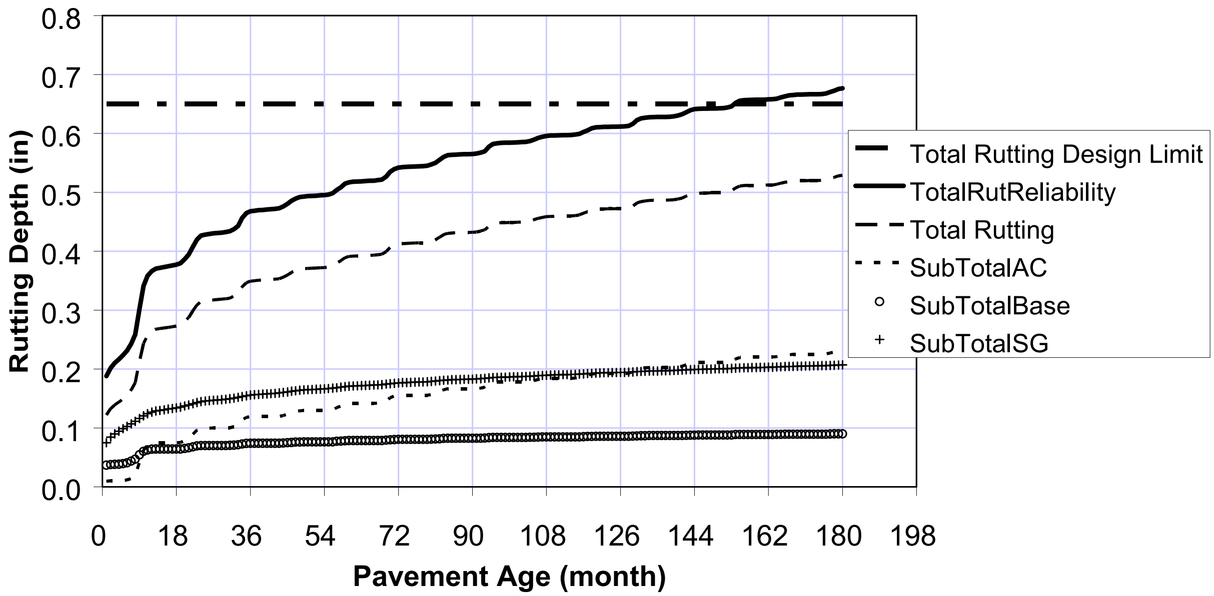



Figure 6-10. Predicted rutting performance for low quality base scenario.

**Table 6-15. Trial cross sections for NCHRP 1-37A flexible pavement design: low quality base scenario.**

| AC Thickness (in.) | Base Thickness (in.) | Total Rutting (in.) |
|---|---|---|
| 5.3 | 12.7 | 0.677 |
| 5.5 | 12.7 | 0.664 |
| 6.0 | 12.7 | 0.635 |
| | Design Limit: | 0.65 |

***Rigid Pavement***

Changing the granular base soil type from A-1-a to A-2-6 changes many of the other Level 3 default inputs for this layer in the NCHRP 1-37A design methodology. The altered input parameters for the base layer in the rigid pavement design are the same as those summarized earlier in Table 6-14 for the corresponding flexible design condition. In addition, the Erodibility Index for the base layer is changed from "Fairly Erodible (4)" to "Very Erodible (5)" (see Table 6-16) to reflect the increased fines content of the A-2-6 base material. In reality, the erodibility is probably somewhere in between these categories, but the NCHRP 1-37A does not permit input of intermediate erodibility conditions.

Figure 6-11 summarizes the predicted faulting vs. time for the baseline rigid pavement section (10.4″ PCC over 6.0″ GAB); the time to the 0.12 inch faulting design limit is only 24 years, corresponding to a 6% decrease in initial service life due to the low quality base condition.

The trial designs and their corresponding predicted performance at end of design life are listed in Table 6-17. Also shown in the table is the design limit for predicted faulting, which is the controlling distress for this scenario. A pavement section consisting of a 10.7 inch PCC slab over 6 inches of GAB (before rounding) meets the design faulting limit for the low quality base scenario. This pavement section is 0.3 inches thicker than the baseline design, but significantly thinner than the 11.5″ slab from the 1993 AASHTO design for this scenario. Increased initial construction cost for the additional 0.3 inches of PCC slab is $8.5K (2.5%) per lane-mile.

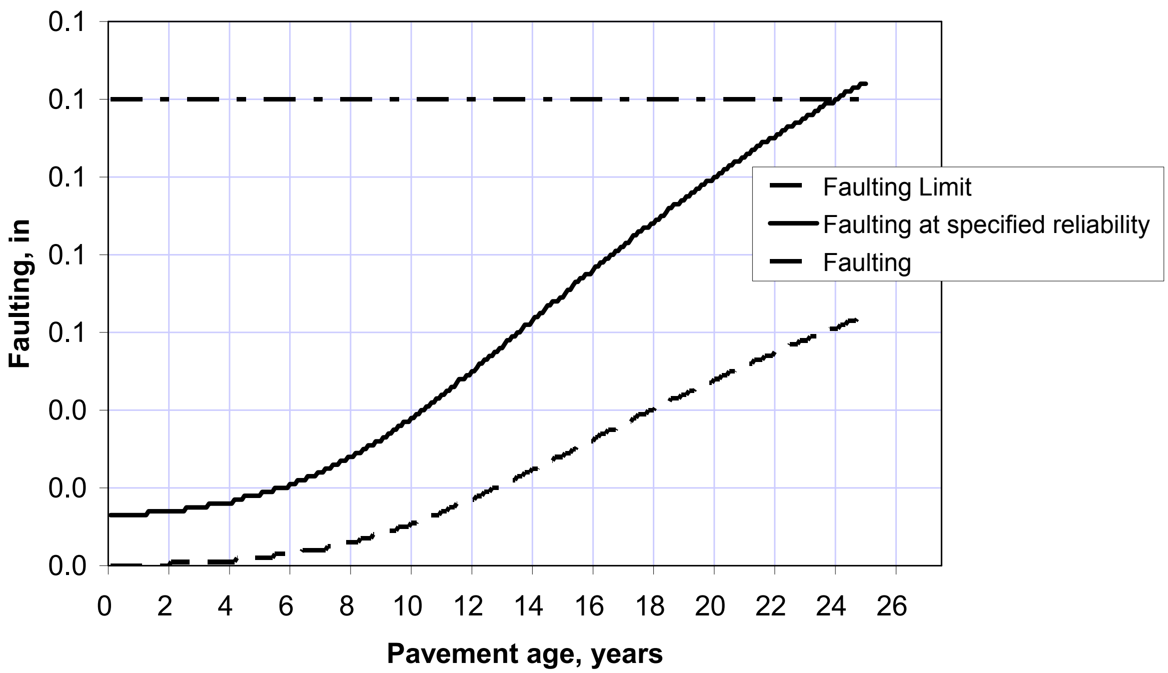



Figure 6-11. Predicted faulting performance for low quality base scenario.

**Table 6-16. NCHRP 1-37A recommendations for assessing erosion potential of base material (adapted after PIARC, 1987; Christory, 1990).**

| Erodibility Class | Material Description and Testing |
|---|---|
| 1 | (1) Lean concrete with previous outstanding past performance and a granular subbase layer or a stabilized soil layer or a geotextile fabric layer is placed between the treated base and subgrade, otherwise class 2.<br>(2) Hot mixed asphalt concrete with previous outstanding past performance and a granular subbase layer or a stabilized soil layer is placed between the treated base and subgrade, otherwise class 2.<br>(3) Permeable drainage layer (asphalt or cement treated aggregate) and a granular or a geotextile separation layer between the treated permeable base and subgrade.<br>(4) Unbonded PCC Overlays: HMAC separation layer (either dense or permeable graded) is specified. |
| 2 | (1) Cement treated granular material with good past performance and a granular subbase layer or a stabilized soil or a geotextile fabric layer is placed between the treated base and subgrade, otherwise class 3.<br>(2) Asphalt treated granular material with good past performance and a granular subbase layer or a stabilized soil layer or a geotextile soil layer is placed between the treated base and subgrade, otherwise class 3. |
| 3 | (1) Cement-treated granular material that has exhibited some erosion and pumping in the past.<br>(2) Asphalt treated granular material that has exhibited some erosion and pumping in the past.<br>(3) Unbonded PCC Overlays: Surface treatment or sand asphalt is used. |
| 4 | Unbound crushed granular material having dense gradation and high quality aggregates. |
| 5 | Untreated subgrade soils (compacted). |

**Table 6-17. Trial cross sections for NCHRP 1-37A rigid pavement design: low quality base scenario.**

| PCC Thickness (in.) | Base Thickness (in.) | Faulting (in.) |
|---|---|---|
| 10.4 | 6 | 0.124 |
| 10.6 | 6 | 0.121 |
| 10.7 | 6 | 0.119 |
| 11.0 | 6 | 0.113 |
| | Design Limit: | 0.12 |

### 6.5.3 Summary

The design flexible pavement sections for the baseline and low quality base scenarios are summarized in Figure 6-12. As expected, the low quality base condition mandates a thicker pavement cross section in both design methodologies. For the 1993 AASHTO design, the required asphalt thickness increases from 5.3 to 8.0 inches (assuming constant granular base thickness); the NCHRP 1-37A design requires a significantly smaller increase of only an additional half-inch of asphalt thickness. Overall, these results suggest that the 1993 AASHTO Guide assigns more weight—at least with regard to rutting, the controlling distress in these scenarios—to the structural contributions from the unbound layers than does the NCHRP 1-37A methodology; this trend has been observed in other comparison studies between the 1993 AASHTO Guide and the new NCHRP 1-37A procedure.

Initial construction cost ranges (based on Table 6-1) for the flexible pavement designs are summarized in Figure 6-12. Note that these costs are very approximate; in particular, the same unit cost has been used for both the high quality (baseline) and low quality granular base materials, but in reality these would likely be different. The initial construction costs for the low quality base scenario (based on the typical unit costs in Table 6-1) increase by about 38K or 21% per lane-mile in the 1993 AASHTO designs and only by about $7.1K or 4% in the NCHRP 1-37A designs.

The design rigid pavement sections for the baseline and low quality base scenarios are summarized in Figure 6-14. The low quality base condition necessitates a significantly thicker pavement cross section in the 1993 AASHTO design, with the slab thickness increasing by 1.1 inches. The required slab thickness increased a much smaller 0.3 inches in the NCHRP 1-37A design. As for flexible pavements, the 1993 AASHTO Guide appears to attach more weight to the structural contributions of the unbound layers than does the NCHRP 1-37A procedure.

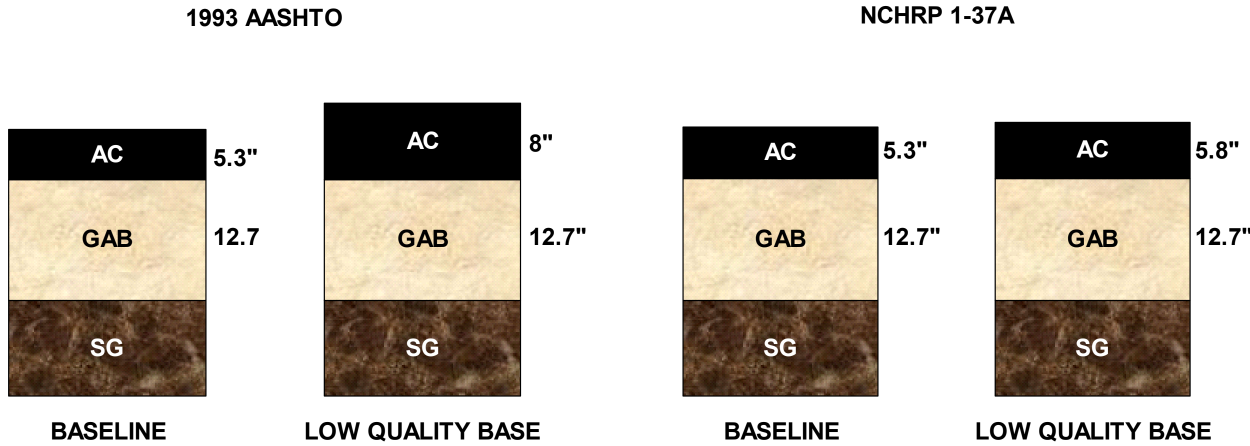



Figure 6-12. Summary of flexible pavement sections: low quality base scenario.

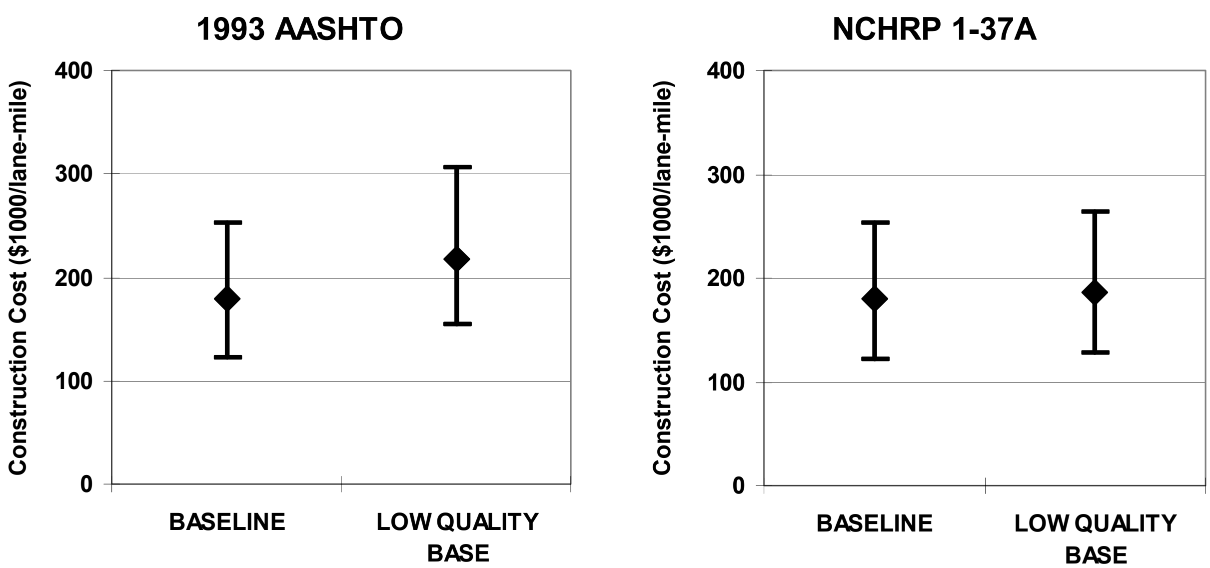



Figure 6-13. Example construction costs for flexible pavement sections: low quality base scenario.

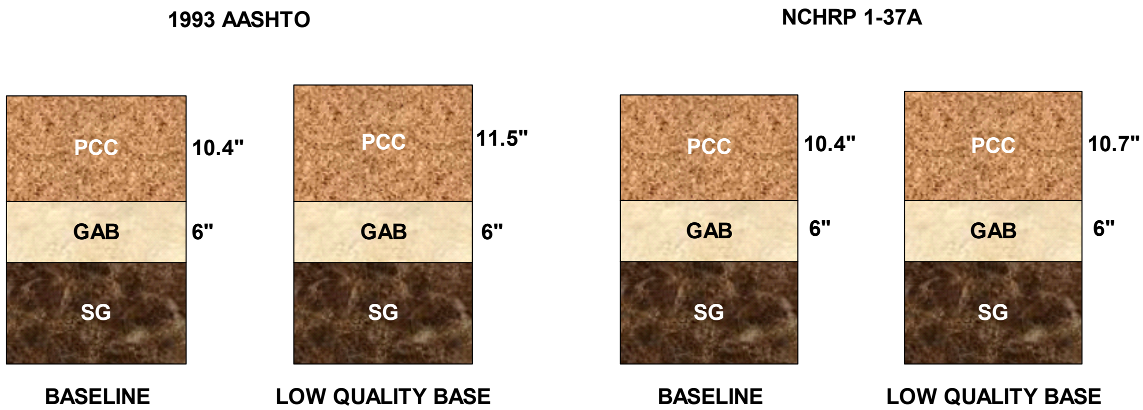



Figure 6-14. Summary of rigid pavement sections: low quality base scenario.

Initial construction cost ranges (based on Table 6-1) for the rigid pavement designs are summarized in Figure 6-15. Note again that these costs are very approximate; in particular, the same unit cost has been used for both the high quality (baseline) and low quality granular base materials, but in reality these would likely be different. The initial construction costs increase by about $31K (9%) per lane-mile for the 1993 AASHTO designs and by $8.4K (2.5%) per lane-mile for the NCHRP 1-37A pavement sections. Note again that the overall magnitudes of these initial construction costs for rigid pavements (Figure 6-15) cannot be fairly compared to the initial construction costs for flexible pavements (Figure 6-13) because of the different assumptions regarding initial service life and different maintenance and repair expenses over the design lives of these different pavement classes. A fair comparison would require evaluation of life-cycle costs, including maintenance, repair, rehabilitation, and perhaps user costs, in addition to the initial construction expense.

The base quality in this scenario is probably not sufficiently low to present a serious problem in design. However, if low base quality does become a critical issue and no high quality crushed material is available, cement or bituminous stabilization could be employed to improve the base quality substantially. Geosynthetics can also be employed for drainage and separation. These techniques are described more fully in Chapter 7.

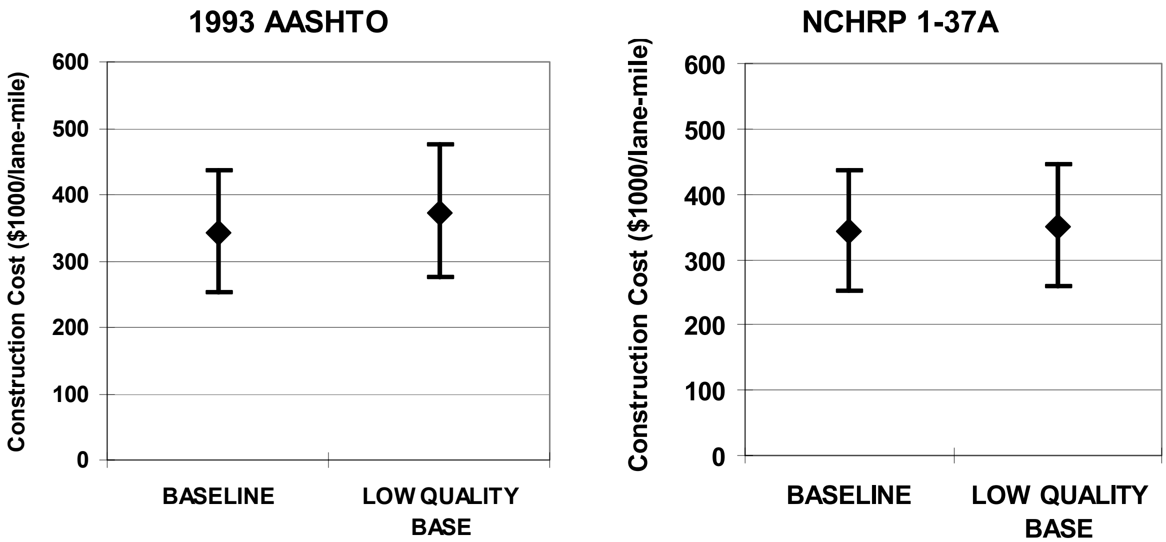



Figure 6-15. Example construction costs for rigid pavement sections: low quality base scenario.

## 6.6 POOR DRAINAGE

Good drainage conditions were explicitly assumed in the baseline design scenario. The values for the drainage coefficients in the 1993 AASHTO designs corresponded to a "good" drainage quality rating, defined as "water removed within 2 hours." This implies a high-permeability base layer (like the A-1-a material assumed in the baseline design scenario) and functioning edge drains. The material properties for the granular base and the drainage length specified in the NCHRP 1-37A design procedure were also consistent with a high-permeability base layer and functioning edge drainage.

In this poor drainage scenario, the material characteristics for the granular base layer remain the same as for the baseline conditions. However, it is now assumed that the edge drains are clogged (or perhaps nonexistent), and the consequences of the ineffective drainage are evaluated. Conceptually, these expected consequences are

- a decrease in the stiffness (and strength) of the granular base layer because of higher average moisture content;
- a decrease in the stiffness (and strength) of the subgrade because of higher average moisture content;
- an increase in other moisture-related distresses (*e.g.,* freeze/thaw cycles, mixing of granular base with subgrade, etc.).

### 6.6.1 1993 AASHTO Design

***Flexible Pavement***

Three modifications to the 1993 AASHTO flexible pavement baseline inputs (Table 6-2) are appropriate for simulating the poor drainage condition:

1. The seasonally averaged resilient modulus and corresponding structural layer coefficient for the granular base material is reduced to reflect the higher average moisture content due to the ineffective drainage. From Table 5-39, the granular base resilient modulus value for wet conditions may be as low as about 40% of that for dry conditions. The $E_{BS}$ = 40,000 psi value for excellent drainage conditions in the baseline design scenario is therefore reduced 35% to 26,000 psi for the poor drainage condition. This corresponds to a structural layer coefficient $a_2$ = 0.12 using the AASHTO correlation in Eq. (5.16). Note that these values coincide with those for the low quality granular base scenario in Section 6.5.
2. The base layer drainage coefficient $m_2$ is reduced from 1.0 to 0.6. This corresponds to a reduction in drainage quality rating from "good" to "very poor" under the assumption that the pavement is exposed to moisture conditions approaching saturation for 5-25% of the time.
3. The seasonally averaged resilient modulus for the subgrade is reduced to reflect the higher average moisture content due to ineffective drainage. From Table 5-39, wet resilient modulus values for granular base and subbase materials may be as low as about 40% of those for dry conditions. It is likely that the stiffness decrease in moisture-sensitive fine-grained subgrade soils will be even greater. Consequently, it is assumed here that the $M_R$ = 7,500 psi value for good drainage conditions in the baseline design scenario is reduced by about 50% to 3,800 psi for the poor drainage condition (all $M_R$ values are after seasonal adjustment). Note that this value coincides with that for the weak subgrade scenario in Section 6.3.

The $W_{18}$ traffic capacity for baseline flexible pavement section under these poor drainage conditions is only $0.67 \times 10^6$ ESALs, corresponding to an 89% decrease in initial service life.

The required structure is then determined from the design equations, as follows (again assuming changes only in the AC layer thickness):

- Required overall structural number $SN$ = 5.76 (unchanged from baseline conditions)
- Structural number provided by granular base (same thickness $D_2$ as in baseline design) $SN_2 = m_2 a_2 D_2 = (0.6)(0.12)(12.7) = 0.91$
- Required asphalt structural number $SN_1 = SN - SN_2 = 5.67 - 0.91 = 4.76$
- Required asphalt layer thickness $D_1 = \frac{SN_1}{a_1} = 10.8$ inches

The design for the poor drainage condition is thus <u>10.8 inches of AC over 12.7 inches of GAB</u> (before rounding). This represents a significant increase in AC thickness, as compared to the baseline AC design thickness of 5.3 inches. The additional asphalt thickness required for the poor drainage condition is slightly more than the additional asphalt thicknesses for the soft subgrade and low quality base scenarios combined. This increase in asphalt thickness translates to a 43% initial construction cost increase of about $78K per lane-mile (using the typical unit cost data in Table 6-1).

***Rigid Pavement***

Four modifications to the 1993 AASHTO rigid pavement baseline inputs (Table 6-2) are appropriate for simulating the poor drainage condition:

1. The resilient modulus for the granular subbase material is reduced to reflect the higher average moisture content due to the ineffective drainage. Consistent with the calculations for the 1993 AASHTO flexible design, the $E_{SB}$ = 40,000 psi value for excellent drainage conditions in the baseline design scenario is reduced to 26,000 psi for the poor drainage condition.
2. The Loss of Support LS coefficient is increased from 2 to 3 to reflect the increased erosion potential in the granular subbase due to the increased moisture levels.
3. The resilient modulus for the subgrade is reduced to reflect the higher average moisture content due to the ineffective drainage. Consistent with the calculations for the 1993 AASHTO flexible design in the preceding subsection, the $M_R$ = 7,500 psi value for good drainage conditions in the baseline design scenario is reduced to 3,800 psi for the poor drainage condition (all $M_R$ values are after seasonal adjustment).
4. The drainage coefficient $C_d$ is reduced from 1.0 to 0.75. This corresponds to a reduction in drainage quality rating from "good" to "very poor" under the assumption that the pavement is exposed to moisture conditions approaching saturation for 5-25% of the time.

The $W_{18}$ traffic capacity for the baseline rigid pavement section under these postulated poor drainage conditions is reduced to 5.2x$10^6$ ESALs, corresponding to a 68% decrease in initial service life.

The first three changes listed above will have a direct effect on the design modulus of subgrade reaction $k_{eff}$, which reduces from its baseline value of 38 pci to a value of 12 pci for the poor drainage conditions. The 1993 AASHTO design corresponding to this $k_{eff}$ and $C_d$ of 0.75 consists of a <u>12.3 inch PCC slab over 6 inches of GAB</u>. Note that this design is substantially thicker than the 10.4" PCC over 6" subbase section for the 1993 AASHTO baseline scenario; this corresponds to an initial construction cost increase of $53K (16%) per lane-mile. As was the case for the low quality subbase scenario, the increase in slab thickness is due primarily to the reduction in the drainage coefficient $C_d$ attributable to the subbase; only about 0.25 inches of the slab thickness increase is due to the increased erodibility of the subbase as reflected in the larger *LS* value and the consequently reduced value for $k_{eff}$.

### 6.6.2 NCHRP 1-37A Design

***Flexible Pavement***

The current version of the NCHRP 1-37A design software does not yet include the capability for directly modeling drainage influences. This in part is due to the paucity of field data available for calibrating the empirical distress models for drainage effects. As stated in the NCHRP 1-37A final report (NCHRP 1-37A, 2004):

> "The calibration of the flexible pavement distress models assumed that no infiltration of moisture occurred throughout the design period. Thus, the flexible pavement design procedure does not allow the designer to choose any level of infiltration at this time. However, adequate consideration to subdrainage should be given when designing flexible pavements."

Although it would be possible to run the NCHRP 1-37A analyses using the moduli values assumed for the 1993 AASHTO designs (Section 6.6.1), this defeats the purpose of including the analysis of seasonal moisture variations on material properties in the mechanistic-empirical methodology. It is also probable that this approximate approach would underestimate the detrimental effects of excess moisture in the pavement structure.

***Rigid Pavement***

Three modifications to the NCHRP 1-37A rigid pavement baseline inputs (Table 6-9) are appropriate for simulating the poor drainage condition:

1. The surface infiltration condition is changed from "Minor (10%)" to "Extreme (100%)".
2. The granular base layer is changed from a free-draining A-1-a to a much less permeable A-2-6 material. This is the same material as for the low quality base scenario in Section 6.5; the modified NCHRP 1-37A design inputs for this material are given in Table 6-14.
3. The Erodibility Index for the base layer is changed from "Fairly Erodible (4)" to "Very Erodible (5)" to reflect the increased fines content of the A-2-6 base material (see Table 6-16).

Figure 6-16 summarizes the predicted faulting vs. time for the baseline rigid pavement section (10.4″ PCC over 6.0″ granular base); the time to the 0.12 inch faulting design limit is only 24 years, corresponding to a 6% decrease in initial service life due to the low poor drainage conditions.

The trial designs and their corresponding predicted performance at end of design life are listed in Table 6-18. Also shown in the table is the design limit for predicted faulting, which is the controlling distress for this scenario. A pavement section consisting of a 10.7 inch PCC slab over 6 inches of GAB meets the design faulting limit for the poor drainage scenario. This pavement section is significantly thinner than the 12.3 inch slab required by 1993 AASHTO design for this scenario, but only 0.3 inches thicker than the design for the baseline conditions, corresponding to an $8.5K per lane-mile (2.5%) increase in initial construction costs.

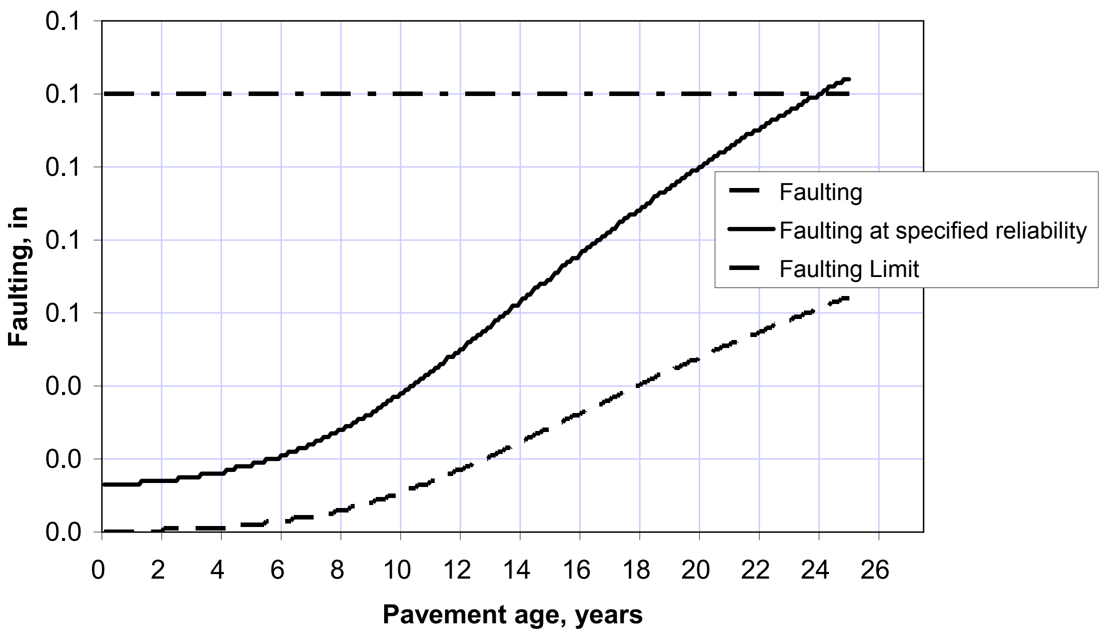



Figure 6-16. Predicted faulting performance for poor drainage scenario.

**Table 6-18. Trial cross sections for NCHRP 1-37A rigid pavement design: poor drainage scenario.**

| PCC Thickness (in.) | Base Thickness (in.) | Faulting (in.) |
|---|---|---|
| 10.4 | 6 | 0.124 |
| 10.7 | 6 | 0.119 |
| 11.0 | 6 | 0.113 |
| | Design Limit: | 0.12 |

### 6.6.3 Summary

The design flexible pavement sections for the baseline and poor drainage scenarios are summarized in Figure 6-17. Only designs from the 1993 AASHTO procedure are included here; as described previously, the NCHRP 1-37A procedure in its present form does not have the capability to analyze this scenario. As expected, the poor drainage condition mandates a substantially thicker pavement cross section, with the required thickness of the asphalt increasing from 5.3 to 10.8 inches (for the simple assumption of constant graded aggregate base thickness).

Initial construction cost ranges (based on Table 6-1) for the flexible pavement designs are summarized in Figure 6-18. The initial construction costs based on the typical unit costs in Table 6-1 increase by about $78K or 44% per lane-mile as a consequence of the poor drainage conditions for the 1993 AASHTO designs.

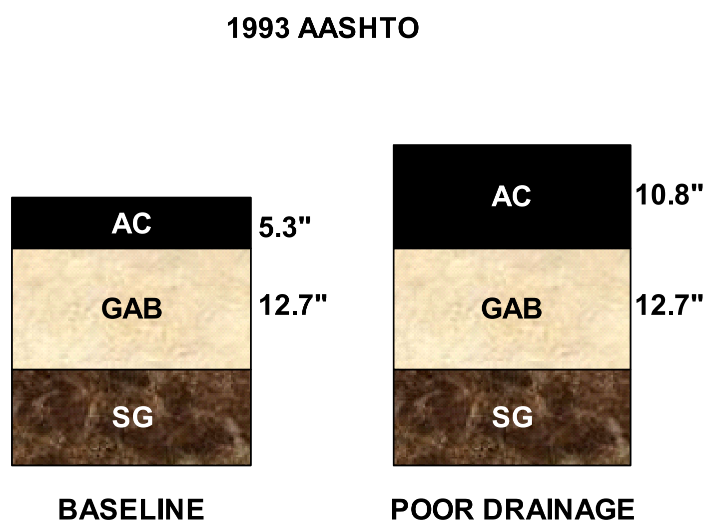



Figure 6-17. Summary of flexible pavement sections: poor drainage scenario.

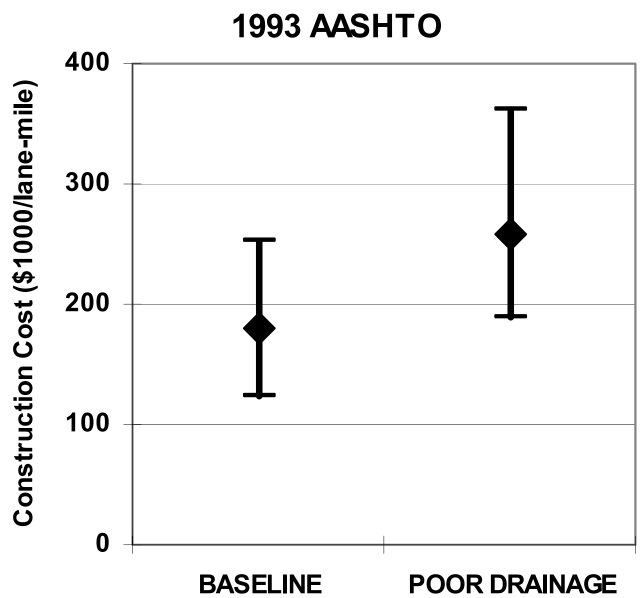



Figure 6-18. Example construction costs for flexible pavement sections: poor drainage scenario.

The design rigid pavement sections for the baseline and poor drainage scenarios are summarized in Figure 6-19. Again, the poor drainage necessitates a thicker pavement cross section. For the 1993 AASHTO design, the required thickness for the PCC slab increases substantially from 10.4 to 12.3 inches (before rounding). For the NCHRP 1-37A designs, the PCC slab thickness increases only from 10.4 to 10.7 inches. These disparities suggest that the 1993 AASHTO procedure overestimates and/or the NCHRP 1-37A methodology underestimates the impact of poor drainage on pavement performance and design requirements. Poor drainage impacts the unbound layers most significantly, and so these disparities are consistent with the trends observed in the low quality base scenario in Section 6.5.

Initial construction cost ranges (based on Table 6-1) for the rigid pavement designs are summarized in Figure 6-20. The initial construction costs increase by about $54K (16%) per lane-mile due to the poor drainage for the 1993 AASHTO designs and by about $8.5K (2.5%) per lane-mile in the NCHRP 1-37A designs. Note again that these construction costs for rigid pavements (Figure 6-20) cannot be fairly compared to the construction costs for flexible pavements (Figure 6-18) because of the different initial service lives and the different maintenance and repair expenses incurred over their design lives. A fair comparison would require evaluation of life-cycle costs, including maintenance, repair, rehabilitation, and perhaps user costs, in addition to the initial construction expense.

These examples clearly show the very substantial effect that ineffective or nonexistent drainage can have on the pavement design thickness. Methods for ensuring adequate drainage in pavements are described in Chapter 7.

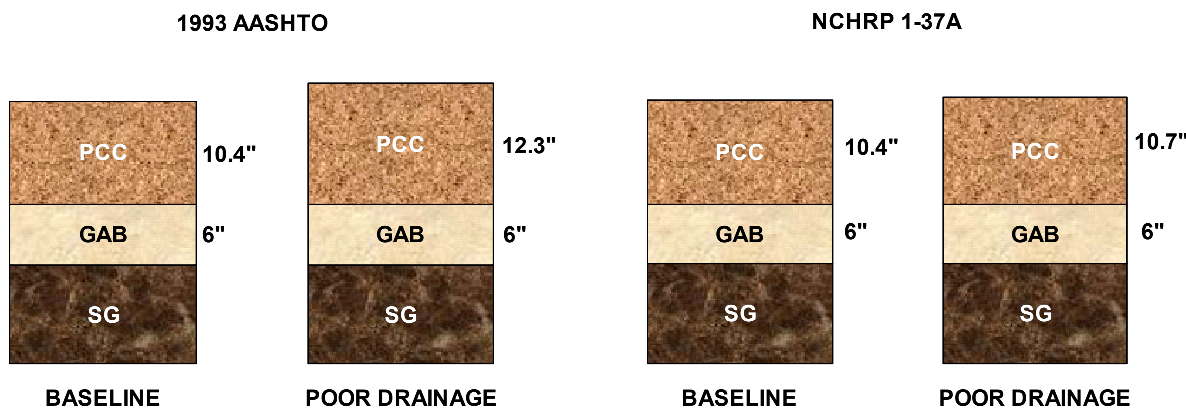



Figure 6-19. Summary of rigid pavement sections: poor drainage scenario.

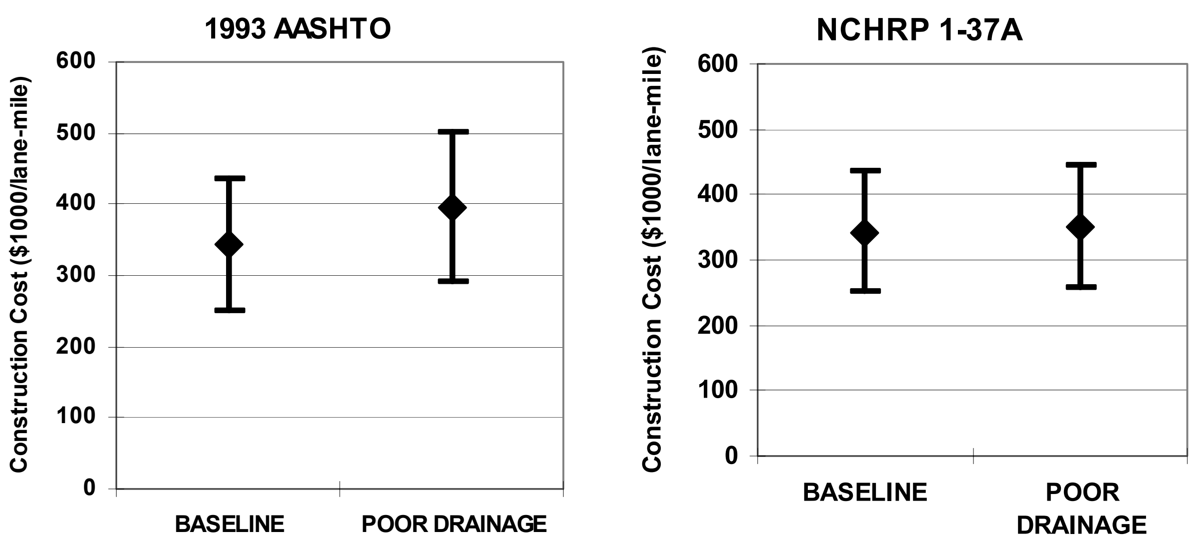



Figure 6-20. Example construction costs for rigid pavement sections: poor drainage scenario.

## 6.7 SHALLOW BEDROCK

### 6.7.1 1993 AASHTO Design

***Flexible Pavement***

Although the presence of shallow bedrock will clearly have a beneficial stiffening effect on the pavement foundation, the 1993 AASHTO Guide does not include any provision for including this benefit in the flexible design procedure. Consequently, the design pavement section for the shallow bedrock scenario will be identical to that for the baseline conditions

***Rigid Pavement***

The presence of shallow bedrock will increase the design modulus of subgrade reaction $k_{eff}$ via the shallow bedrock correction factor (Figure 5.25). Values of $k_{eff}$ for several bedrock depths are summarized in Table 6-19, along with the corresponding values of required slab thickness. All design inputs other than depth to bedrock are equal to the baseline conditions (Table 6-3), and the granular subbase thickness is held constant at 6 inches. The results in Table 6-19 indicate that, for the design conditions in this example, the depth to bedrock has a moderate effect on the design modulus of subgrade reaction $k_{eff}$, but that this has a nearly negligible effect on the required slab thickness. The design pavement section for the case of a 4 foot depth to bedrock is a slightly thinner <u>10.3 inch PCC slab over 6 inches of GAB</u>. Viewed alternatively, bedrock at a 4 foot depth will increase the allowable traffic $W_{18}$ to $17.0x10^6$, increasing the initial service life of the baseline rigid pavement section by approximately 1 year (4%).

**Table 6-19. Design pavement sections for 1993 AASHTO rigid pavement design: shallow bedrock effects.**

| Depth to bedrock (ft) | ∞ | 8 | 4 | 2 |
|---|---|---|---|---|
| Composite subgrade modulus $k_\infty$ (pci) | 423 | 474 | 577 | 704 |
| Design subgrade modulus $k_{eff}$ (pci) | 38 | 41 | 46 | 52 |
| Slab thickness $D$ (in.) | 10.4 | 10.4 | 10.3 | 10.3 |

### 6.7.2 NCHRP 1-37A Design

***Flexible Pavement***

Depth to bedrock is an explicit design input in the NCHRP 1-37A procedure. Table 6-20 summarizes the predicted total rutting (adjusted for reliability) for the baseline flexible pavement conditions (Table 6-4) as a function of depth to bedrock, as predicted by the NCHRP 1-37A methodology. For the conditions in this example, a 4 foot depth to bedrock will reduce the predicted rutting by about 17%, or increase the initial service life by about 18 years (120%). This translates directly to a thinner required pavement section. Trial designs and their corresponding predicted performance at end of design life for the case of a 3 foot depth to bedrock are summarized in Table 6-21. A pavement section consisting of <u>3 inches of AC over 12.7 inches of GAB</u> is sufficient for the shallow bedrock scenario. This corresponds to an initial construction cost reduction of about $38K (18%).

**Table 6-20. Influence of bedrock depth on predicted total rutting: NCHRP 1-37A design methodology.**

| Depth to Bedrock (ft) | Total Rutting (in.) |
|---|---|
| ∞ | 0.646 |
| 10 | 0.578 |
| 8 | 0.544 |
| 4 | 0.533 |

**Table 6-21. Trial cross sections for NCHRP 1-37A flexible pavement design: baseline conditions with shallow bedrock at 4 ft depth.**

| AC Thickness (in.) | Base Thickness (in.) | Total Rutting (in.) |
|---|---|---|
| 5.3 | 12.7 | 0.533 |
| 4.0 | 12.7 | 0.593 |
| 3.0 | 12.7 | 0.651 |
| | Design Limit: | 0.65 |

***Rigid Pavement***

Depth to bedrock is an explicit design input in the NCHRP 1-37A procedure. Table 6-22 summarizes the predicted joint faulting (adjusted for reliability) for the baseline rigid pavement conditions (Table 6-9) as a function of depth to bedrock, as predicted by the NCHRP 1-37A methodology. For the conditions in this example, a 4 foot depth to bedrock will reduce the predicted faulting by about 75%. This translates directly to a thinner required pavement section. Trial designs and their corresponding predicted performance at end of design life for the case of a 4 foot depth to bedrock are summarized in Table 6-23. The NCHRP 1-37A software cannot model PCC thicknesses less than 7 inches. Even at this reduced thickness, however, the predicted faulting for the shallow bedrock conditions is well below the design limit.

**Table 6-22. Influence of bedrock depth on predicted joint faulting: NCHRP 1-37A design methodology.**

| Depth to Bedrock (ft) | Faulting (in.) |
|---|---|
| ∞ | 0.117 |
| 20 | 0.093 |
| 10 | 0.040 |
| 8 | 0.033 |
| 4 | 0.028 |

**Table 6-23. Trial cross sections for NCHRP 1-37A rigid pavement design: baseline conditions with shallow bedrock at 3 ft depth.**

| PCC Thickness (in.) | Base Thickness (in.) | Faulting (in.) |
|---|---|---|
| 10.4 | 6.0 | 0.028 |
| 9.0 | 6.0 | 0.041 |
| 7.0 | 6.0 | 0.079 |
| | Design Limit: | 0.12 |

## 6.8 CONCLUDING COMMENTS

It is worthwhile to review the primary objectives of the design studies presented in this chapter:

1. to illustrate via examples how the geotechnical properties described in Chapter 5 are incorporated in the pavement design calculations in the 1993 AASHTO and NCHRP 1-37A procedures; and
2. to highlight the effects of the geotechnical factors and inputs on pavement design performance.

The design scenarios are intentionally highly idealized and simplified. Their point is to emphasize in quantitative terms how changes in geotechnical inputs affect the overall pavement design and performance. These design scenarios are also good examples of the types of sensitivity studies one should perform during design to evaluate the importance of the various design inputs, especially with reference to the quality of the information used to determine these inputs. As succinctly stated by Hamming (1973) in the frontispiece of his pioneering book on engineering computation: "The purpose of computing is insight, not numbers."

The ultimate measure for comparing the designs for the various scenarios is cost. Life-cycle costs are the best measure, but calculation of life-cycle costs is beyond the scope of these exercises. As a fallback, the various design scenarios can be compared in terms of their initial construction cost ratios using the example unit cost data from Table 6-1 and the baseline design conditions as the reference (*i.e.,* cost index=1). These comparisons are presented in Figure 6-21 and Figure 6-22 for flexible and rigid pavement designs, respectively. Remember that it is inappropriate to compare flexible versus rigid pavements based only on initial construction costs because of the different assumptions regarding initial service life and different maintenance and repair expenses that would be incurred over the design lives of these different pavement classes.

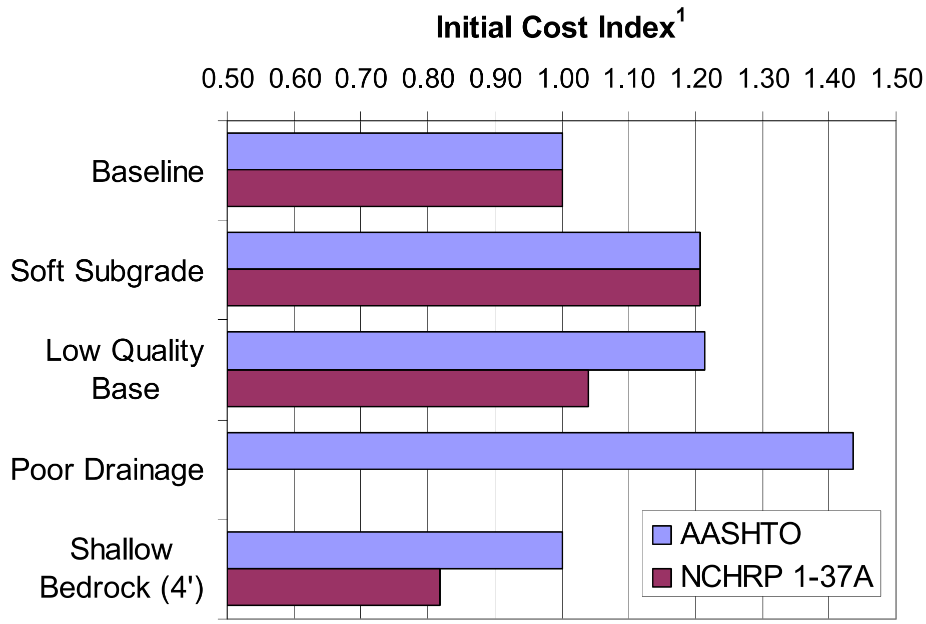
Some key observations to be drawn from Figure 6-21 and Figure 6-22 include the following:

- Poor drainage is by far the most detrimental geotechnical factor for flexible pavements. For the conditions in these examples, poor drainage drives up the initial cost of the flexible pavement design by nearly 50%.
- For the flexible pavement scenarios considered here, a very soft subgrade is the second most detrimental geotechnical factor, driving initial costs up by about 20%. However, the soft subgrade condition can be mitigated by lime stabilization or other remedial measures, as described in more detail in Chapter 7.

- More thickness may be required if lower quality materials are used for the granular base layers in flexible pavements, but the thickness increase may be partially or fully compensated by lower unit costs for the lower quality materials. This will be very sensitive to the specific unit costs of competing materials in the project location.
- Overall, the rigid pavement designs were much less sensitive to geotechnical factors than were the flexible designs. The range and variations in the cost index among the various design scenarios were much less for the rigid pavement designs (Figure 6-22) than for the flexible pavements (Figure 6-21).

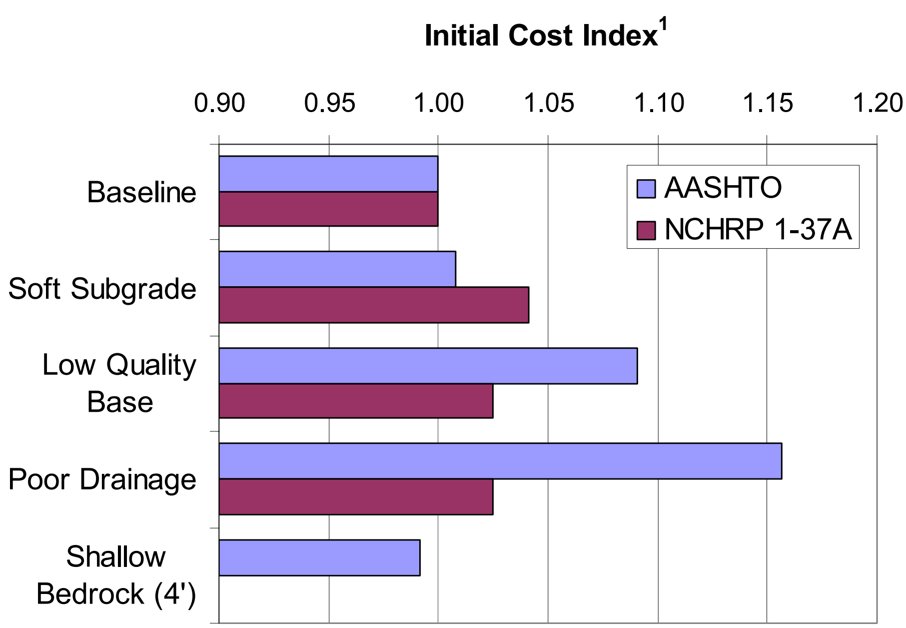
An alternate way of looking at the impact of the various geotechnical design parameters is in terms of their effect on initial service life for the baseline pavement section. Initial service lives for the various design scenarios are summarized in Figure 6-23 and Figure 6-24 for the flexible and rigid pavement sections, respectively. The observations in terms of service life are similar to those from costs. For the flexible pavements, poor drainage, soft subgrade, and low quality base had the most impact on service life (especially for the AASHTO designs), reducing the initial service life from 15 to as little as 2 years in the most extreme case. For the rigid pavements, poor drainage and low quality base again had the most significant impact (again, especially for the AASHTO designs), reducing the initial service life from 25 to as little as 8 years. The soft subgrade conditions had comparatively less effect on the rigid pavement service life, as compared to the flexible pavements, confirming the advantages of rigid pavements for very poor foundation conditions.

Of course, all of the specific observations from these results apply only to the particular design scenarios considered in these illustrative studies. Pavement design conditions and in-place unit costs will vary considerably across agencies and regions. Nevertheless, the simple design scenarios presented here demonstrate quite convincingly the important effects that geotechnical factors can have on design pavement sections and costs.




[1]Based on typical unit cost data from Table 6-1. For low quality base scenario, unit cost for granular base is assumed to be the same as for high quality base; in reality, this unit cost will likely be lower.

Figure 6-21. Summary of costs for example design scenarios: flexible pavement designs.




[1]Based on typical unit cost data from Table 6-1. For low quality base scenario, unit cost for granular base is assumed to be the same as for high quality base; in reality, this unit cost will likely be lower.

Figure 6-22. Summary of costs for example design scenarios: rigid pavement designs.

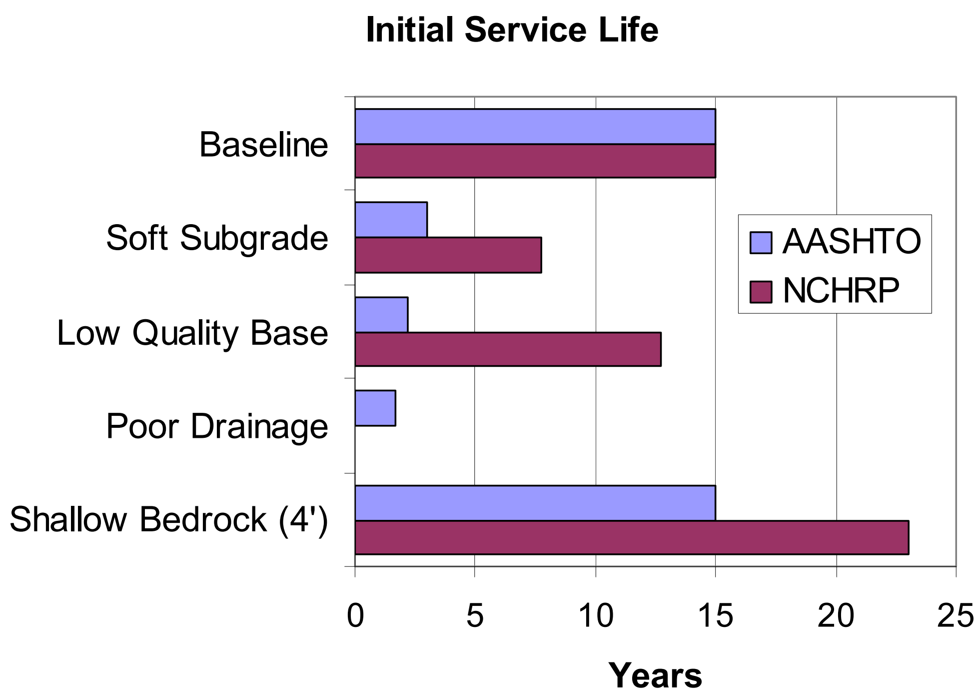



Figure 6-23. Initial service lives for example design scenarios: flexible pavement designs.

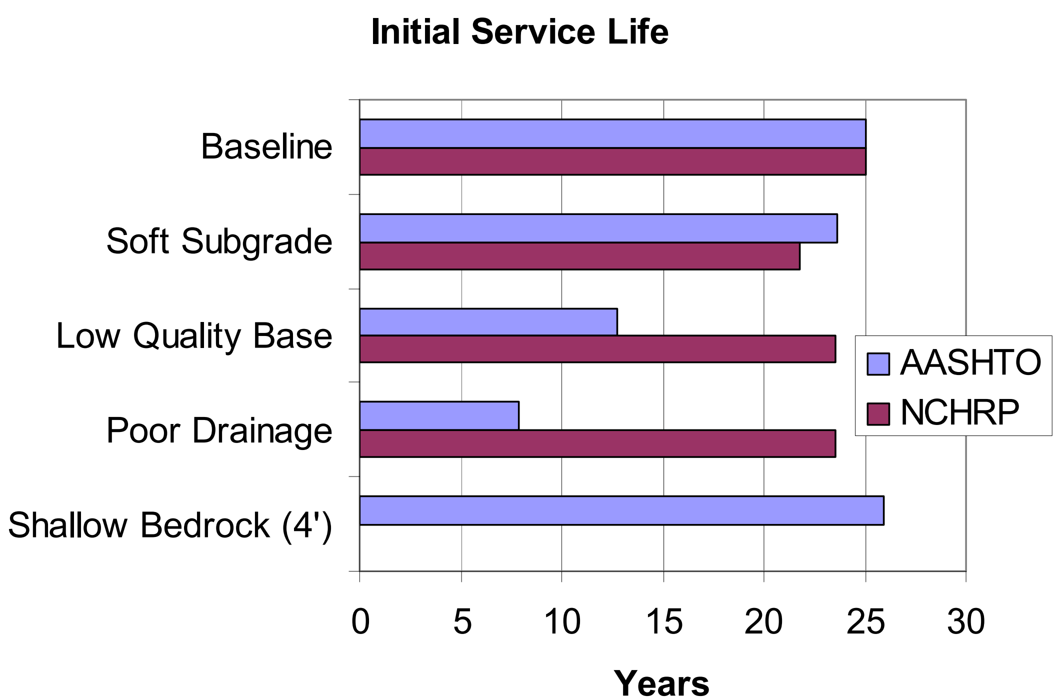



Figure 6-24. Initial service lives for example design scenarios: rigid pavement designs.

## 6.9 EXERCISES

Students will divide into groups to develop designs for the Main Highway project. Each group will focus on either a flexible or a rigid pavement design. Specific tasks for each group are as follows:

- Develop a pavement design based on the most-likely conditions expected at the site.
- Identify the short list of critical geotechnical inputs to which the design is expected to be most sensitive.
- Perform some initial sensitivity evaluation for the critical geotechnical inputs.
- Complete computations using a simple Excel spreadsheet provided by the instructors.

## 6.10 REFERENCES


AASHTO (1993). *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C.

Christory, J.P. (1990). "Assessment of PIARC Recommendations on the Combating of Pumping in Concrete Pavements," Proceedings of the Second International Workshop on the Design and the Evaluation of Concrete Pavements, Siguenza, Spain.

Hamming, R.W. (1973). *Numerical Methods for Scientists and Engineers*, McGraw-Hill, New York, NY.

MDSHA (2002). *Pavement Design Guide*, Pavement Division, Office of Materials and Technology, Maryland State Highway Administration, Baltimore, MD.

NCHRP 1-37A (2004). *Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*, Draft Report, Transportation Research Board, National Research Council, Washington, D.C.

PIARC (1987). "Combating Concrete Pavement Slab Pumping," PIARC Technical Committee on Concrete Roads.